Structural Design in Metals

CLIFFORD D. WILLIAMS
CHIEF ENGINEER
PATCHEN & ZIMMERMAN, ENGINEERS
AUGUSTA, GEORGIA

ERNEST C. HARRIS
CHAIRMAN, DEPARTMENT OF CIVIL ENGINEERING
FENN COLLEGE

SECOND EDITION

THE RONALD PRESS COMPANY · NEW YORK

PREFACE

This second edition of *Structural Design in Metals* continues to provide the material for a coordinated first course in design at the junior-senior level, following the mechanics and strength of materials courses. It has been revised extensively to incorporate new materials and new illustrative examples that reflect the latest specification and design methods. As a result of suggestions made by users of the first edition, for which the authors are indebted, some clarifications have also been added.

The treatment continues to place emphasis on basic training in the application of the statics of simple structures, and the strength of materials, to details of design. It is thought that the design of a complicated structure in whole adds little to the knowledge obtained from the complete analysis of the details and that, in the time which may be spent on a first course in design, much would be lost by the design of only one type of complete structure. Details, if properly designed, should be as applicable to one type of structure as another. Hence, throughout this text, emphasis is placed on the design of details rather than on the complete structure. It is assumed that special applications of the principles of design may be made in later courses if those applications seem desirable. The complete design of buildings, bridges, aircraft, etc., falls in this classification—each requiring long hours of application, but each being combinations of details and sections which are, in principle, applicable to all.

In each phase of work, both welded and riveted details are analyzed as being of equal importance. Recognition is also given to the increasing use of other metals besides steel, such as aluminum and its alloys, and to the concept of structural design in aircraft work. No attempt is made toward complete analysis of aircraft or toward the specific separation of aircraft design from other structural design because the principles of structural design are the same wherever applied. Much of the advance in structural design methods in the aircraft field is applicable to other fields of structural design.

The authors feel that the student should be given food for thought. The most harmful course is one which makes design procedure a matter of routine. The chapter on "Fatigue of Structural Members" is included to bring the student up to date on this important subject and, at the same time, cause him to appreciate the mass of information which remains for the engineer to learn.

The chapter on "The Rigid Frame" provides material for those schools that have included analysis of statically indeterminate structures in the course of study. Somewhat more emphasis is placed on the analysis in this case because of the great variation existing in courses now given in this field. The method used is considered to be the simplest and best understood by those not having considerable grounding in the subject. For the same reasons, the design has been carried through rather completely. In those schools where the material on rigid frames does not fit into the undergraduate curriculum, it may be valuable for graduate work.

The authors again express their gratitude to the organizations that have permitted the use of standards and specifications reprinted as appendixes. These organizations include the American Association of State Highway Officials, the American Railway Engineering Association, the American Welding Society, the American Iron and Steel Institute, and the American Society of Civil Engineers. Special recognition is also given to the following companies, who generously furnished photographs for use in this book: Chicago Bridge and Iron Company, Bethlehem Steel Company, Boeing Aircraft Company, American Bridge Company, and the Stran Steel Division of the Great Lakes Steel Corporation.

CLIFFORD D. WILLIAMS
ERNEST C. HARRIS

June, 1957

CONTENTS

APPENDIX

Structural Design
in
Metals

CHAPTER 1

INTRODUCTION

1–1. The Structural Engineers' Field. The structural engineer is concerned with the conception, design, construction, economics, and maintenance of many types of structures for a great variety of purposes.

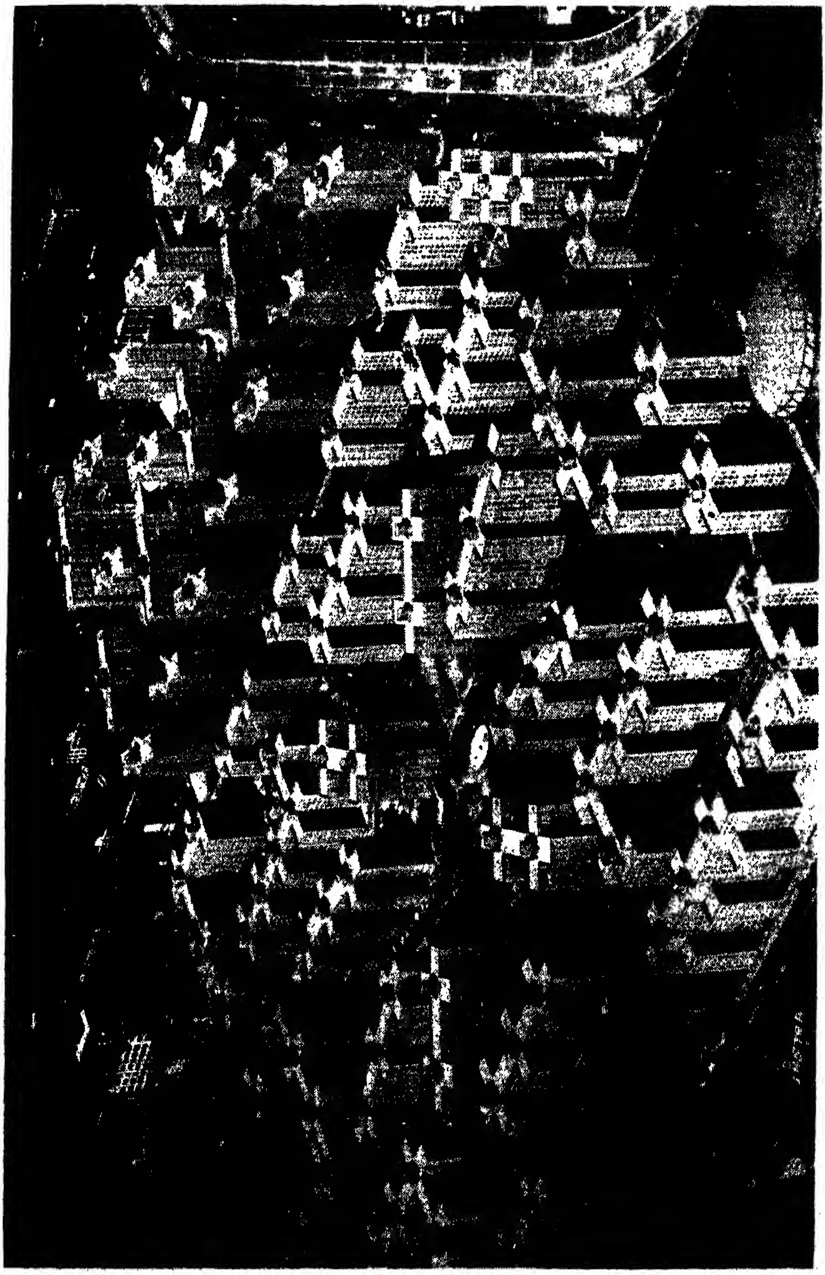
Typical among those structures requiring the knowledge and experience of structural engineers are railway and highway bridges, buildings of every type, industrial equipment, hydraulic and drainage structures, dams, flood-control structures, aircraft, tanks, and ships. Agencies representing the owners and responsible for the design and construction of structures include railroads and other public utilities, the federal government, states, counties, cities, governmental authorities, manufacturers, and private individuals.

The structural engineers taking direct responsibility for the engineering of structures are a part of either the owner's organization or the organization of a consulting office retained for the particular work. The responsibility of the engineer is to provide a structure fitting into its proper place in the economics involved, designed and constructed according to accepted principles of good engineering practice and function, and resulting in an appearance that will be of credit to the profession. To accept such responsibility, the engineer must be a continuous student, must have a thorough understanding of mechanics and materials, must have a knowledge of design procedures and construction processes, and must keep himself informed of the latest developments. He should be an individual capable of making personal contributions to engineering knowledge.

1–2. Design Principles. The engineer will recommend the use of materials and types of structures best suited to the conditions under consideration. Whether the structure be of wood, steel, concrete, aluminum alloys, plastics, brick, or other materials will usually be determined by the construction cost, maintenance cost, availability of materials, and appearance. In general, these considerations are best satisfied when the structure is functional, with each member and each detail serving a direct purpose and made from the material and by the process best adapted to that purpose. Originality of design is to be commended, but never at the cost of violating sound engineering principles.

1–3. Scope of the Study. The study with which this treatise is concerned is the design of structures of metal. Naturally, only a few structures can be studied within the limitations reasonable to the devoted space.

The study of current specifications and their applications to detail design will be applicable to almost all structures, whether or not of the type illus-



Bethlehem Steel Co.

STUYVESANT TOWN AND PETER COOPER VILLAGE, NEW YORK CITY.

trated. The choice of type of structure requires engineering experience and knowledge of the various types. Such engineering knowledge must include the details involved in each case.

Examples of applications of structural details to buildings, railroad and highway bridges, and aircraft structures using riveted and welded construction as applied to currently available metals are included in the study which follows.

It is assumed that the student has a working knowledge of statics and the strength of materials. Discussions of these areas of study are kept to a minimum to provide space for their applications.

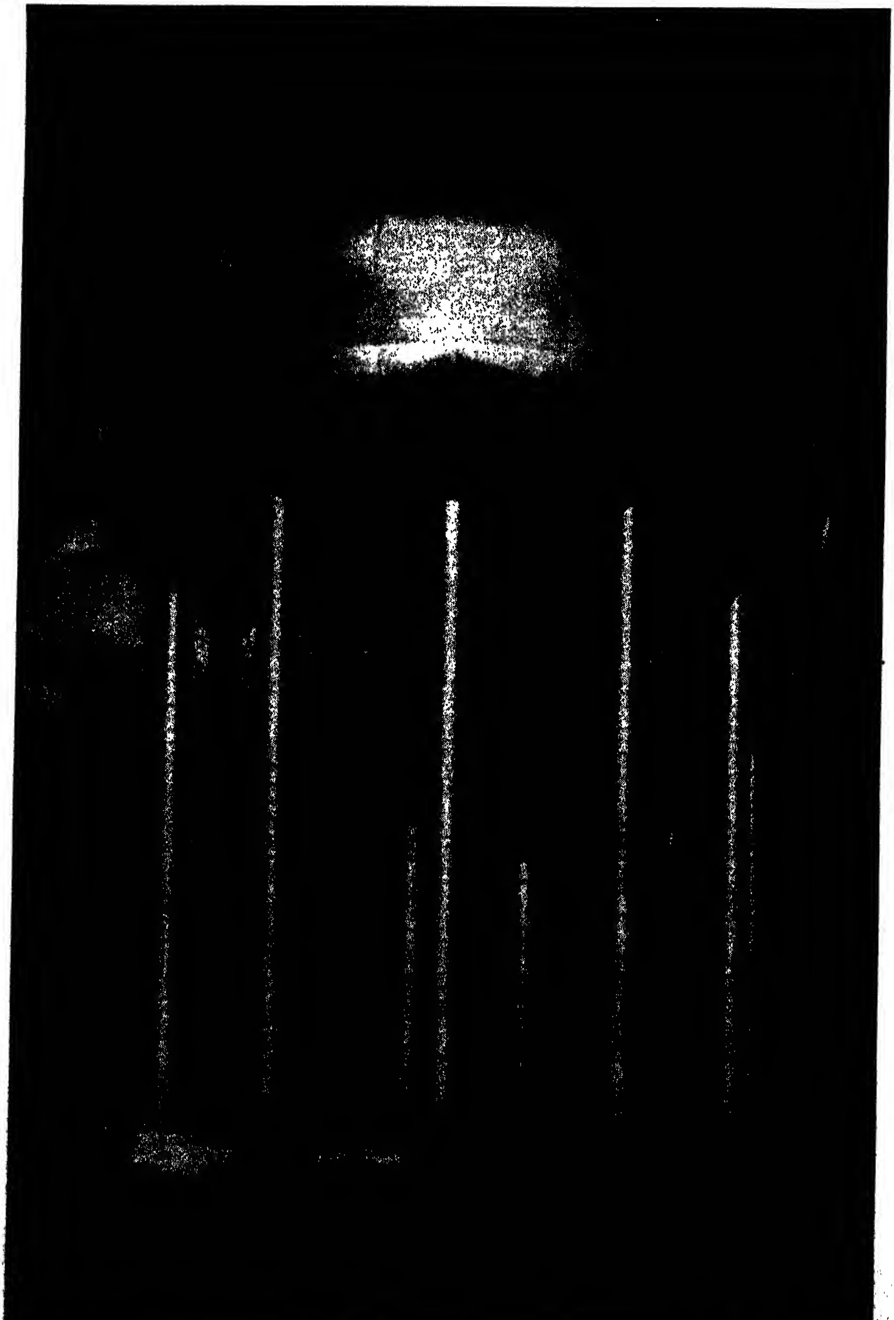
1-4. Specifications. General engineering specifications are developed by engineering organizations, usually by means of committees appointed for that purpose. Any specification represents a collective opinion of best engineering practice. Such specifications are continuously being changed because of newly developed knowledge brought about by tests, theoretical developments, and trial. It should be kept in mind that many of the clauses of specifications are the result of arbitration of opinions, with the purpose of providing a safe structure within reasonable limitations of economy. Engineers should not hesitate to improve upon the design required by the minimum specification limitations and to furnish suggestions and information to the specification writers.

The American Standards Association (ASA)* is a federation of 77 national organizations maintained by business to provide the development and use of standards and to serve as the national clearing house for standards. Its membership includes 47 trade associations, 20 technical societies, 10 major branches of the federal government, and 2,000 companies. A large number of standards has been adopted which pertain directly to the various phases of structural engineering.

The American Society for Testing Materials (ASTM) is a membership organization concerned primarily with the development of standards for testing of materials and for the materials themselves. The ASTM specifications for steel and other metals used in structures are referred to by practically all design and workmanship specifications dealing with these materials.

The American Institute of Steel Construction is an organization of structural steel fabricators and rolling mills. It is engaged in research and the dissemination of information pertaining to the use of steel products. About 20 prominent structural engineers serve as a Technical Advisory Board. The *AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings,* revised 1949, will be cited throughout the text as the AISC specification. This specification has a quite general adoption for building structural design.

* The information about the American Standards Association as given here is taken from their publication No. 4401, dated February 15, 1944.



Chicago Bridge and Iron Co.

1,500,000-Gal Radial-Cone-Bottom Elevated Tank, 75 Ft to Bottom.

The *Standard Specifications for Highway Bridges*, as developed by the American Association of State Highway Officials, is used by a large percentage of state highway departments and other agencies concerned with the design of highway bridges. This specification will be cited in the text as the AASHO specification. Sections of the specification that pertain to structural steel design are reproduced in Appendix A.

The American Railway Engineering Association is a membership society composed of railway engineers, representatives of industries serving the railways, and engineers and educators interested in railway problems. Committee 15 of the AREA, the Iron and Steel Structures Committee, is responsible for the *Specifications for Steel Railway Bridges*, which is included as Section 15 of the AREA Manual. This specification is a standard for railway bridge design in the United States and many foreign countries. Parts of Section 15 of the AREA Manual pertaining to steel railway bridge design are included as Appendix B. Reference will be made throughout the text to this specification as the AREA specification.

Military aircraft design requirements are largely determined by Army and Navy Specifications. Commercial aircraft are designed under recommendations made by the National Advisory Committee for Aeronautics and the Army-Navy-Civil (ANC) specifications. *Strength of Aircraft Members*, ANC-5, covers requirements for detail structural design. Design methods and applications of engineering principles to the structural details of aircraft are published in several books dealing with that subject.

A committee of the American Society of Civil Engineers on Design in Lightweight Structural Alloys has prepared *Specifications for Heavy Duty Structures of High-Strength Aluminum Alloy*. This specification is included as Appendix E and will be referred to throughout the text as the aluminum specification.

The American Welding Society is a membership organization interested in research and development of standards for welded construction. The specifications of the AWS are considered authoritative and are generally supplementary to the general specifications for design. The *AWS Standard Code for Arc and Gas Welding in Building Construction*, 1946, and the AWS *Standard Specifications for Welded Highway and Railway Bridges*, 1947, are included as Appendix C. Appendix D contains information developed by the AWS pertaining to standard symbols for use in designating welding on drawings.

The American Iron and Steel Institute is a trade organization that has developed much useful technical information for the use of structural designers. Appendix F contains the *AISI Specification for the Design of Light Gage Cold-Formed Steel Structural Members*.

1–5. Materials. Structural steel is the chief construction metal in terms of quantity. A majority of structural steel is covered by ASTM designation A7.

Steel might be called a refined product of iron ore found in natural deposits. Iron oxidizes very easily; hence the iron in the natural deposits occurs principally in the form of magnetite (Fe_3O_4) or hematite (Fe_2O_3). Natural deposits of iron ore contain relatively high percentages of oxides of other metals, which must be separated to make the iron usable.

The common method* of production of steel starts with the blast-furnace process and is followed by the open-hearth or electric furnace. The blast-furnace process reduces the ore to pig iron and has slag as a by-product. Iron ore, coke, and limestone are fed into the stack of the blast furnace in charges. The stack of the modern blast furnace is approximately 100 ft high. As the charges of materials settle down in the stack, the temperature is increased to a maximum of about 3000F. A majority of the materials other than iron are driven off as gases, or combine to form a slag. At the bottom of the furnace the molten slag floats on the iron and is drawn off and cooled on beds. Slag has important uses as media for trickling filter beds in sewage plants, as an aggregate for the manufacture of concrete, as ballast for railroads, and for the manufacture of cement.

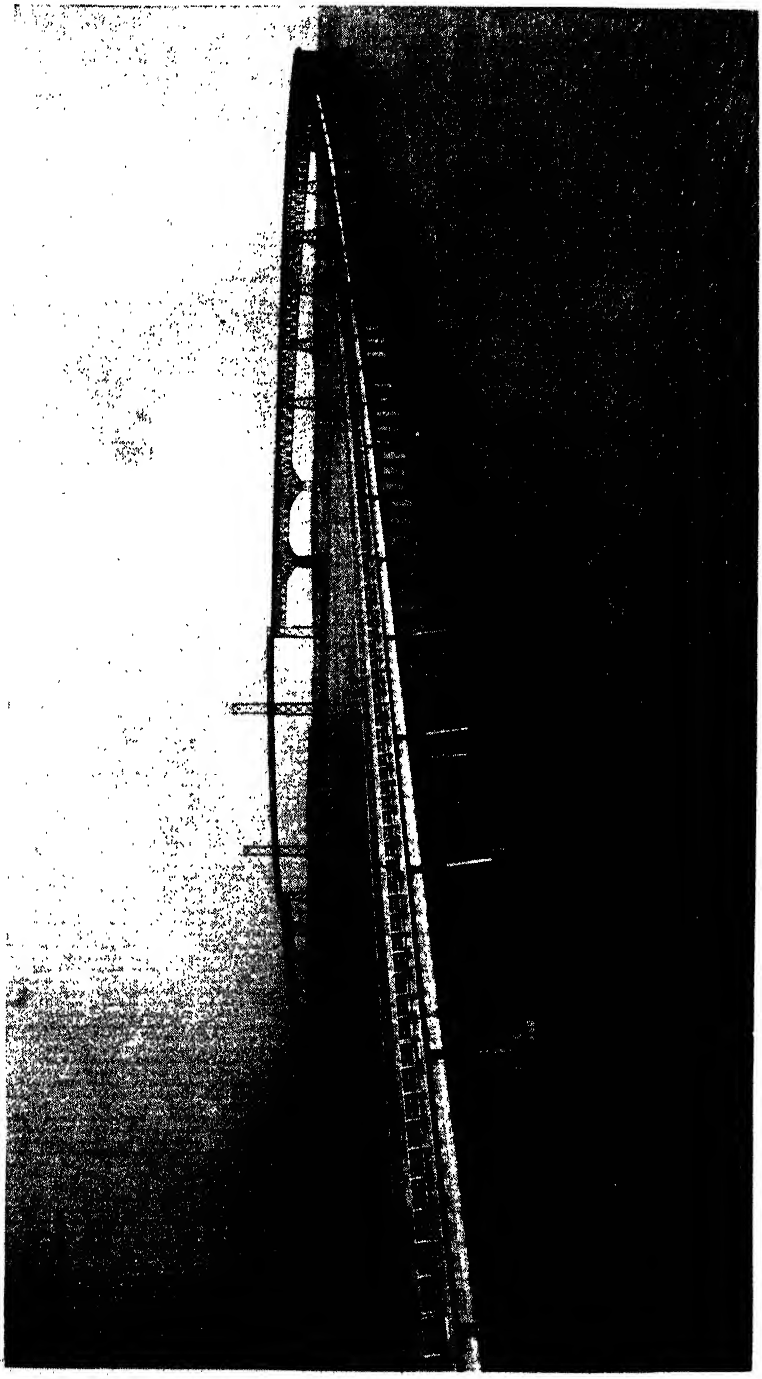
The molten iron is drawn off from the bottom of the blast furnace into ladles and may be transported directly to steel mills or cast into molds in the form of "pigs."

Pig iron contains relatively small percentages of silicon, sulfur, phosphorus, and manganese and from 3.50 to 4.25 per cent of carbon. This percentage of carbon is too great to permit its use as a construction metal subjected to tension. Iron is used, however, for cheap forms of castings, the primary functions of which are to carry compressive loads. Cast iron is a relatively brittle product because of the high percentage of carbon, and it is a material which cannot be successfully shaped except by casting.

Wrought iron is manufactured from pig iron in the puddling furnace. The iron is melted, and specially prepared slag is added. A large part of the carbon, silicon, manganese, and phosphorus of the pig iron is taken up by the slag. The resulting material consists of relatively pure metallic iron particles incorporated in slag coatings. Wrought iron is also made by the newer Aston process, which uses material from the Bessemer furnace rather than pig iron. Wrought iron develops a tensile strength from 25,000 to 45,000 psi and is used where corrosive influences require a material which will not oxidize readily.

A majority of steel is produced by basic open-hearth furnaces. The raw materials are principally pig iron, iron ore, and limestone. The process is principally one of converting pig iron into a product having smaller, controlled quantities of carbon and having the injurious quantities of oxides and phosphorus removed. The electric furnace is used to a limited extent in the manufacture of steel. Because of the high cost of operation, its uses

* A. H. White, *Engineering Materials* (New York: McGraw-Hill Book Co., 1939).



Bethlehem Steel Co.

CHESAPEAKE BAY BRIDGE.

are restricted principally to making alloy steels and steels in which the control must be very close.

The product of the open-hearth furnace is molded into ingots where the steel may either be cooled for future use or be transmitted in its hot state to the rolling mill. In the rolling mill the ingot is placed in a soaking pit, where the steel obtains a uniform temperature at the correct value for

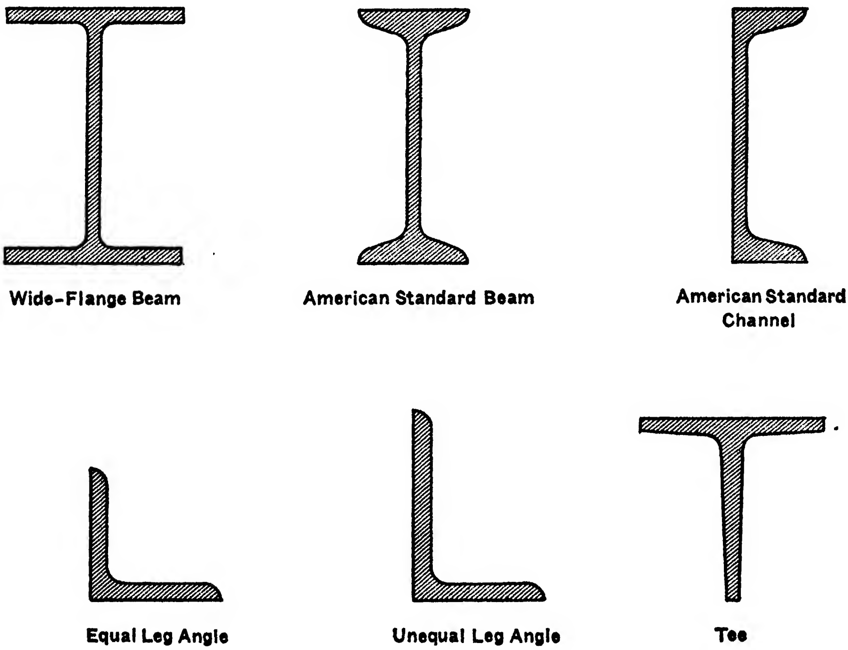


Fig. 1–1.

rolling. When the steel ingot has reached the rolling temperature, it is started through the rolls. After a few passes through the rolls a part of the top of the ingot is sheared off to eliminate the lower-grade steel which usually contains cavities or pipes.

The rolling process consists of a number of passes through rolls which gradually extend the length of the billet and bring the cross section closer to the final shape. The most common shapes are the wide-flange beam, American standard beam, American standard channel, equal leg and unequal leg angles, car channels, and light beams. A few of the common shapes are shown in Fig. 1–1. Each series of shapes is produced in several weights. The weights are varied by spreading the rolls to increase the thickness of metal in the shape.

In the open-hearth or electric-furnace processes, the silicon content may be controlled. Addition of silicon of 0.20 per cent results in a structural silicon steel covered by ASTM designation A94. This steel has a tensile strength of 80,000 to 95,000 psi and yield point of 45,000 psi.

Silicon structural steel is frequently used in long-span structures where the dead load is an important factor. Although the weight of silicon structural steel is no less than that of the common grade of structural steel, it is capable of about one-third increase in unit stress with slight increase in cost per pound, thus economically reducing the sizes of members. When sizes of members are chosen by limiting material thickness rather than by stress, no saving can be accomplished by the use of an alloy steel.

A low-alloy steel covered by ASTM designation A242, *Specification for Low-Alloy Steel*, has gradually replaced silicon structural steel because of its better weldability. This steel has a yield point stress from 40,000 to 50,000 psi.

Nickel structural steel has a tensile value about 50 per cent higher than the ordinary grade of structural steel but is a higher-priced steel.

The addition of 0.2 per cent copper to the specification A7 steel is frequently required where corrosion from rust is a factor. This small amount of copper is inexpensive and adds considerably to the corrosion resistance of the steel.

The aluminum alloys are not used extensively for structural work at the present time, as compared with steel, except in the aircraft industry. These alloys weigh about 35 per cent as much as steel and have tensile values comparative to steel. Aluminum alloys suited to structural purposes will undoubtedly find their place in many structural applications in the future. The use of structural aluminum shapes in the floor system of the Smithfield Street bridge in Pittsburgh prolonged the life of this bridge many years, in spite of increased live loads. Many applications have been made to traveling cranes, floor bulkheads, and other industrial equipment in which weight is an important factor. The aluminum alloys are used extensively in aircraft production and are, in fact, the principal material in this field. Much progress is to be expected in the aircraft industry in the way of standardization of rolled and extruded shapes. At the present time, there are hundreds of shapes used as standards of individual aircraft companies. Economical production will eventually require more standardization of these shapes.

Many types of heat-treated steel and corrosion-resistant steel are used in aircraft production. Magnesium castings have a prominent place in this field because of their high structural value and light weight.

Tables 1–1 and 1–2 show the anticipated physical properties of the more commonly used structural metals. It is usual to refer the properties of other metals to those of structural steel, which is the most commonly used metal. The student should memorize the chief physical properties of structural steel and be familiar with the relations of other metals to this product.

Structural steel manufactured under ASTM designation A7 does not have a limitation on the carbon and manganese content. For this reason it is

TABLE 1-1

COMMON STRUCTURAL METALS

	Structural Steel	Low Alloy Structural Steel	Silicon Structural Steel	Nickel Structural Steel	Rivet Steel	High Strength Bolts	Wrought Iron	Cast Steel	Aluminum Alloy 6061-T6	Aluminum Alloy 2014-T6
ASTM designation	A7	A242	A94	A8	A141	A325	A42	A27(65–30)	GS11A	CS41A
E (Young's modulus)	29,400,000	29,400,000	29,400,000	29,400,000	29,400,000	29,400,000	25,000,000	29,400,000	10,000,000	10,600,000
Ultimate tension	60,000–72,000	63,000–70,000	80,000–95,000	90,000–115,000	52,000–62,000	90,000–120,000	39,000–46,500	65,000	38,000–42,000	60,000–68,000
Yield point—tension	33,000	40,000–50,000	45,000	55,000	28,000	74,000–81,000	27,000	30,000	35,000*	53,000–60,000*
Yield point—compression	......	......	......	......	......	......	......	......	35,000	55,000*
Elongation in 8 in.	21%	18–20%	16%	14%	24%	......	8–13%	......		......
Elongation in 2 in.	24%	24%	19%	......	......	14%	......	24%	8–14%	3–8%
Coefficient of expansion	0.0000065	0.0000065	0.0000065	0.0000065	0.0000065	0.0000065	0.000006	0.0000065	0.000013	0.000013
Endurance limit	......	......	......	......	......	......	......	......	14,500	19,000
Weight: lb per cu in.	0.283	0.283	0.283	0.283	0.283	0.283	0.278	0.283	0.098	0.101
Reduction in area	......	......	0.30	0.30	......	......	......	0.30	......	......
Poisson ratio	0.30	0.30	0.30	0.30	0.30	0.30	0.28	0.30	0.33	0.33

* Stress producing deformation of 0.2 per cent of length

TABLE 1–2

STRUCTURAL METALS COMMON TO AIRCRAFT

	Aluminum Alloy 2024-T4	Aluminum Alloy 7075-T6	Magnesium Castings	Steel X-4130	Alloy Steel Heat Treated 100,000 psi	Alloy Steel Heat Treated 125,000 psi	Alloy Steel Heat Treated 150,000 psi	Alloy Steel Heat Treated 180,000 psi
Specifications	AN-A-12 QQ-A-354	AN-A-11 QQ-A-283	ANQQM58	....	†	†	†	†
E (Young's modulus)	10,600,000	10,400,000	6,500,000	29,000,000	29,000,000	29,000,000	29,000,000	29,000,000
G (shear modulus)	4,000,000	3,900,000	2,400,000	11,000,000	11,000,000	11,000,000	11,000,000	11,000,000
Ultimate tension	57,000–76,000	70,000–80,000	32,000–34,000	90,000–95,000	100,000	125,000	150,000	180,000
Ultimate compression	....	....	41,000–49,000	....	....	....	....	....
Ultimate shear	30,000–40,000	41,000–47,000	18,000–20,000	55,000	65,000	75,000	90,000	105,000
Yield point*—tension	40,000–52,000	60,000–72,000	10,000–18,000	75,000	80,000	100,000	135,000	165,000
Yield point—compression	38,000–50,000	63,000–72,000	13,000–18,000	75,000	80,000	100,000	135,000	165,000
Shear—yield strength	23,000–25,000	32,000–38,000	....	40,000	55,000	65,000	80,000	95,000
Elongation in 2 in.	6–17%	3–8%	1–6%	....	....	....	....	....
Brinell hardness	....	....	63	241+	....	....	....	....
Coefficient of expansion	0.000013	0.000013	....	0.0000065	0.0000065	0.0000065	0.0000065	0.0000065
Poisson ratio	0.33	0.33	....	....	....	....	....	....
Endurance limit‡	20,000	22,000	12,500	45,000	50,000	65,000	78,000	85,000
Weight: lb per cu in.	0.100	0.101	0.066	0.283	0.283	0.283	0.283	0.283

* Yield point for aluminum alloys defined as stress producing deformation of 0.2 per cent of length.

† Steel alloys containing less than 0.5 per cent carbon.

‡ Rotating beam—100,000,000 cycles.

possible for that steel to have quantities of these elements too high to be suitable for welding. During 1954, a new structural steel under ASTM designation A373 was introduced. This specification limits the maximum carbon to 0.25 to 0.27 per cent, and manganese between 0.96 and 0.94 per cent for plates over ½ in. thick. However, the majority of steel produced under the A7 specification meets these requirements and the new specification is not frequently used.

1–6. Special Properties of Aluminum Alloys.* Aluminum is produced by an electrolytic process from bauxite ore. In its commercially pure form, aluminum has an ultimate tensile strength of about 13,000 psi. The strength of aluminum can be increased by adding various alloying elements, by heat treatment, and by cold working. The tensile strength of commercially pure aluminum can be raised to about 24,000 psi by cold working. Heat-treated alloys with tensile strengths greater than 80,000 psi are available.

Aluminum alloys weigh about 0.1 lb per cu in., approximately one-third of the weight of steel. The modulus of elasticity of aluminum is also about one-third of the value for steel. The coefficient of thermal expansion is nearly double that of steel.

Stress-strain curves for aluminum alloys do not have the sharply defined yield-point characteristic of similar curves for structural grade steel. Yield strength for aluminum alloys is defined as the stress which produces a permanent set of 0.2 per cent of the initial gage length. For the aluminum alloys used in structural applications, the yield strength is generally between 65 and 90 per cent of the ultimate tensile strength.

Commercially pure aluminum is given the alloy number 1100. It has high resistance to corrosion and is used extensively for deep-drawn or die-stamped parts not requiring high tensile strength. Aluminum alloys fall into two classifications, the heat-treatable and the nonheat-treatable alloys.

The nonheat-treatable alloys cover a range of strength from low to intermediate. They find extensive application in such products as corrugated roofing sheet, wall panels for office buildings, pressure vessels, and storage tanks. The principal alloying ingredients in these alloys are manganese and magnesium. Various degrees of strength and ductility are attained by introducing various amounts of cold work.

The heat-treatable aluminum alloys are commonly used in structural applications. The American Society of Civil Engineers has published specifications† covering design rules and fabrication procedures for these alloys, designated as 6061-T6 and 2014-T6. Alloy 6061-T6 has a specified tensile yield strength of 35,000 psi and a specified tensile strength of 38,000 to 42,000 psi, depending upon the product. This alloy contains small per-

* Article 1–6 was contributed by J. W. Clark, Research Engineer, Engineering Design Division, Aluminum Research Laboratories, Aluminum Company of America.

† Appendix E.

Boeing Aircraft Co.

THE B-52C. AN EIGHT-JET, 400,000-POUND STRATOFORTRESS USED BY THE U.S. STRATEGIC AIR COMMAND.

centages of magnesium, silicon, copper, and chromium (2.1 per cent in all). It is used where medium strength and high resistance to corrosion are desired. Where higher strength is needed, alloy 2014-T6 is frequently used. The principal alloying element in this material is copper (4.4 per cent), and silicon, manganese, and magnesium are also added (a total of 2.0 per cent). The specified tensile yield strength of 2014-T6 varies from 53,000 to 60,000 psi, depending upon the product in which it is used, and the tensile strength varies from 60,000 to 68,000 psi. The highest-strength, commercially available alloy is 7075-T6, which has a specified tensile strength as high as 80,000 psi. Alloy 7075-T6 is confined largely to aircraft applications.

In alloy designations such as 6060-T6, the "T6" indicates that the material has been solution heat-treated and then artificially aged. The temper designation system used by the Aluminum Company of America is as follows:

- -F As fabricated.
- -O Annealed, recrystallized (wrought products only).
- -H Strain hardened.
  - -H1, plus one or more digits. Strain hardened only.
  - -H2, plus one or more digits. Strain hardened and then partially annealed.
  - -H3, plus one or more digits. Strain hardened and then stabilized.
- -W Solution heat-treated—unstable temper.
- -T Treated to produce stable tempers other than -F, -O, or -H.
  - -T2 Annealed (cast products only).
  - -T3 Solution heat-treated and then cold worked.
  - -T4 Solution heat-treated.
  - -T5 Artificially aged only.
  - -T6 Solution heat-treated and then artificially aged.
  - -T7 Solution heat-treated and then stabilized.
  - -T8 Solution heat-treated, cold worked, and then artificially aged.
  - -T9 Solution heat-treated, artificially aged, and then cold worked.
  - -T10 Artificially aged and then cold worked.

Structural aluminum alloys are available in the form of sheet, plate, shapes, tubes, rods, bars, rivets, and forgings. Shapes may be either rolled or extruded. The extrusion process, in which shapes are formed by forcing hot metal through an orifice having the shape of the desired cross section, is of particular interest since it permits the fabrication of many shapes that cannot be obtained by rolling. Aluminum alloys are also fabricated by various methods of casting. Special casting alloys are used for this purpose.

The aluminum alloys are easily machined, which permits more rapid fabrication and less tool trouble than is encountered with steel. The amount of forming that can be performed on a given product depends on the alloy and the temper. Forming operations that are too severe for a final temper of a heat-treated alloy can be performed when the material is in the annealed condition or in the "freshly-quenched" temper. An interesting and valuable characteristic of heat-treatable aluminum alloys is that the material is relatively soft after quenching, but it continues to gain strength and hardness at room temperatures. About 90 per cent of the aging effect takes place in the first day after heat treatment, and there is a marked change in the workability during the first hour. Hardening, which takes place in the first hour at room temperature, may be retarded for a considerable period by placing the heat-treated, quenched material in a refrigerator at zero temperature. This property of the alloy is utilized in the forming and die processing of the material and in the handling of rivets, which may be kept in the more workable state for about 24 hr after heat treatment. After the forming operation, natural or artificial aging develops the full strength.

The aluminum alloys in general have good resistance to corrosion by weather. For example, alloy 6061-T6 resists corrosion so well that it is generally used in the bare condition with no protective covering. Some other alloys, such as 2014-T6, are painted or given some other protective coating where corrosive conditions are likely to be encountered. Sheets and plates of 2014-T6 and similar alloys are made with a surface coating of relatively pure aluminum, which provides high corrosion resistance and results in an average reduction in tensile strength of about 5 per cent as compared with the bare alloy sheet of the same total thickness. The coated material is designated as "Alclad"; for example, Alclad 2014-T6. For some applications, particularly in the architectural field, the protective oxide coating that forms naturally on aluminum alloys is augmented by an electrolytic process to provide a thicker, more durable oxide coating.

Some of the aluminum alloys, such as 2014-T6, are not very suitable for welding and are generally joined by rivets or bolts. Other alloys, for example 6061-T6, are readily weldable. But welds in aluminum alloys *in their annealed condition* are as strong as the parent metal. When welds are made in aluminum alloys in the cold-worked or heat-treated tempers, the metal near the weld is softened by the heat of the welding so that its strength is generally intermediate between the strength of the original parent metal and the strength of the fully annealed material. In some cases, parts are re-heat treated after welding to restore their strength. Re-heat treating is not always advantageous because it results in some sacrifice of ductility of the weld and is often accompanied by warping.

Since aluminum alloys are available with a tensile strength equivalent to structural grade steel, aluminum alloy tension members can be designed weighing only one-third as much as their steel counterparts. Because of the lower modulus of elasticity, however, the weight advantage of aluminum is not so great in the case of members whose strength is determined by by buckling or of members that must be limited on the basis of a limited amount of deformation. In general, about 50 per cent of the weight of a structure can be saved by using aluminum rather than steel. This saving in weight plus the good corrosion resistance of aluminum alloys, which cuts maintenance costs, are the main points that help to offset the higher first cost.

1–7. Processes. The majority of structural work is processed for riveted construction. Welded construction volume is increasing rapidly. A large volume of welded construction has developed in the building field, while ship building is principally welded construction. Not much welding is used in new railroad bridges, but a large amount of maintenance and repair work employs welding. Extensive research work on the various problems of both riveted and welded design and in the use of high-strength bolts is in progress.

1–8. Structural Types. Early prejudice against the use of continuous and rigid-frame types of construction is becoming less as methods of analysis become simplified and more generally known. Continuous and rigid-frame structures have not always been used with good judgment; many examples of such structures might better have been simple determinate structures. The rigid frame is usually a pleasing structure in appearance, but it is not always the economical construction. Greater care is required in the investigation of supports for continuous and rigid-frame structures. Such structures may be used with advantage provided that sound engineering mechanics are employed in the design.

A structure is most pleasing when every member has a definite function. Loads can be carried with the use of the least material when the main structural members are subjected to direct stress rather than bending. The suspension bridge, carrying its principal load by direct tension in the cable, and the arch, loaded primarily in compression, are typical of purely functional structures. These types are, of course, usually restricted to relatively long spans. The beam, or some adaption of the beam, is the functional structure for short spans. Buildings are necessarily composed of beams, floors, walls, roofs, and footings; such factors as long clear spans, as well as earthquake resistance and wind resistance, enter into the selection of frames or trusses as principal carrying members.

Trusses and structural frames are composed of members having special functions. The beam, carrying its load principally in bending, may be a rolled section such as an I-beam, a wide-flanged beam, or a channel; or it may be a section built up from rolled sections, such as a rolled beam with cover plates, a plate girder, or a box girder. Certain members are required to carry direct tension rather than bending. Such a member may also be a rolled I-beam or wide-flange beam; it is more frequently, however, a rolled angle or plate or some combination of angles and plates. Columns are members taking their principal load in compression. They are most commonly rolled wide-flange sections, with or without cover plates. Many columns are built up from angles and plates, sometimes with lacing to prevent local buckling. A member may need to serve as both a beam and a column. That is, the member may carry an important compressive load and, at the same time, be required to provide for loads causing transverse bending. This member, sometimes called a *beam-column*, may be a rolled section or any one of the built-up combinations serving either as a column or beam.

Bethlehem Steel Co.

Scroll Casing for Hydraulic Turbine, Davis Dam, Colorado River.

The student is referred to Chapters 4, 6, and 7 for examples of the various load-carrying elements of a structure and to the AISC handbook for available rolled sections. Various plates throughout the text illustrate a variety of structures that are of interest to the structural engineer.

1–9. Structural Information. Examination of structural drawings furnishes a source of much information. The student should be on the alert to observe unusual solutions to problems in design, but he should also watch for and avoid the use of improper solutions. The fact that something has been done is not a justification for repetition.

Such books as Ketchum's *Structural Engineers Handbook* and the *Welding Handbook* by the American Welding Society are useful for reference.

Steel Construction by the American Institute of Steel Construction, or a similar handbook published by one of the steel companies, is an essential tool of the structural engineer. The handbook contains dimensions and

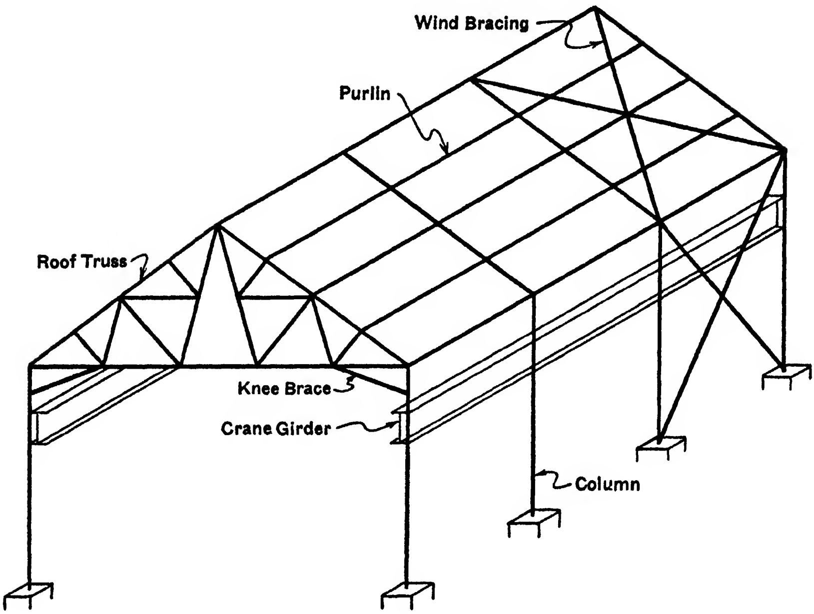


FIG. 1–2.

properties of all available steel shapes, with useful information about details of design and mathematical tables. The AISC handbook is required for the work which follows.

The *Alcoa Structural Handbook*, published by the Aluminum Company of America, is the structural shapes handbook for available aluminum alloy sections.

Continuous reference should be made to sources of reports on structural research. Typical of these sources are the engineering bulletins of colleges and universities, National Bureau of Standards research papers, and the periodicals of engineering societies.

The student should study plans for structures of all types and should take every opportunity to inspect buildings, bridges, and other structures under

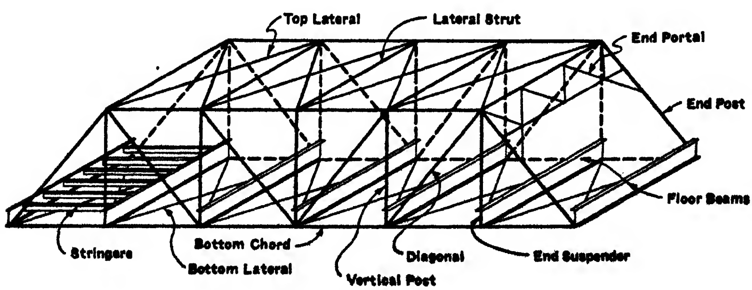


FIG. 1–3.

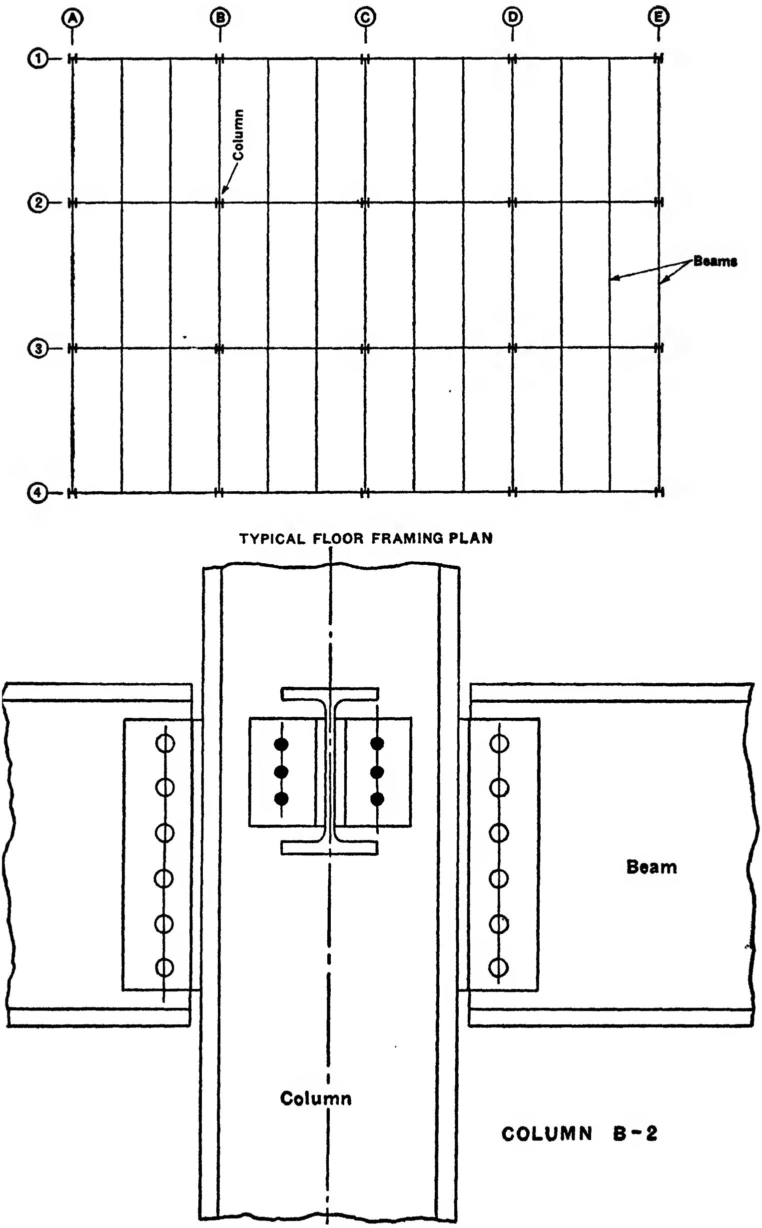


Fig. 1–4.

construction. Figures 1–2, 1–3, and 1–4 are sketches showing the elements of common structures.

The structural designer should recognize the differences in function between design drawings and shop drawings. Design drawings are prepared to present all the required design information necessary for the construction of a structure. On the other hand, the detailing of steel members is considered to be a part of the construction process and is not a part of the design. The design drawing should show all pertinent data affecting the calculation of the strength or utility of the structure, but it should not attempt to detail steel members for the shop.

A detail drawing, usually prepared by the contracting steel fabricator, will show all information necessary for the layout and fabrication of the member or structure in the shop. Figure 6–8 is a design drawing for a plate girder. Figure 1–5 is a detail drawing with sufficient information for fabrication of a beam with its connections.

Too much detail on a design drawing frequently uses the time of a designer unreasonably and may, at the same time, cause the detailer more

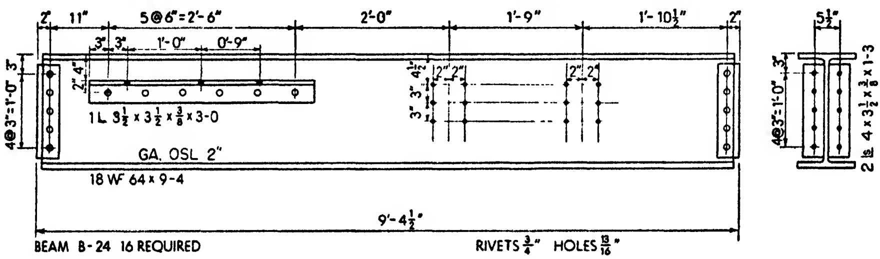


Fig. 1–5. Typical shop detail.

trouble in completing the shop details. On the other hand, information on the design drawings should be complete enough to ensure that the details will provide the structure that is wanted.

1–10. Loads. A structure is designed to carry a useful load, usually termed *live load,* in addition to the weight or dead load of the structure itself. It is frequently necessary to provide for other forces in the design.

The loads and forces for which a bridge may be designed are classified as dead load, live load, impact effect, wind load, longitudinal forces due to traction or friction, centrifugal force, thermal force, earth pressure, buoyancy, shrinkage forces, rib shortening, erection stresses, ice and current pressure, snow loads, earthquake forces, and other lateral forces.

Each design specification defines the loads for which the structure is to be designed and provides for the manner in which the resulting stresses are to be combined.

Determination of the weight of a structure is usually a successive process. The first element to be designed is one that does not carry another member. The weight of the first element must be estimated before a design is completed and the estimate corrected to satisfy the final design. As each element is progressively designed, a part of the dead load on the next member becomes more certain. Much useful information about anticipated weights of structures is available in texts on bridge design and in handbooks. The AISC handbook provides tables of weights of various construction materials.

Live loads are dependent on the type of structure under consideration. Specifications for design provide for live loadings that are intended to take care of the anticipated loadings. These loadings do not necessarily closely resemble the actual loads coming to the structure, but they do provide simplified design criteria which result in an equivalent structure. Special conditions may arise requiring variation in the live loads for a particular situation.

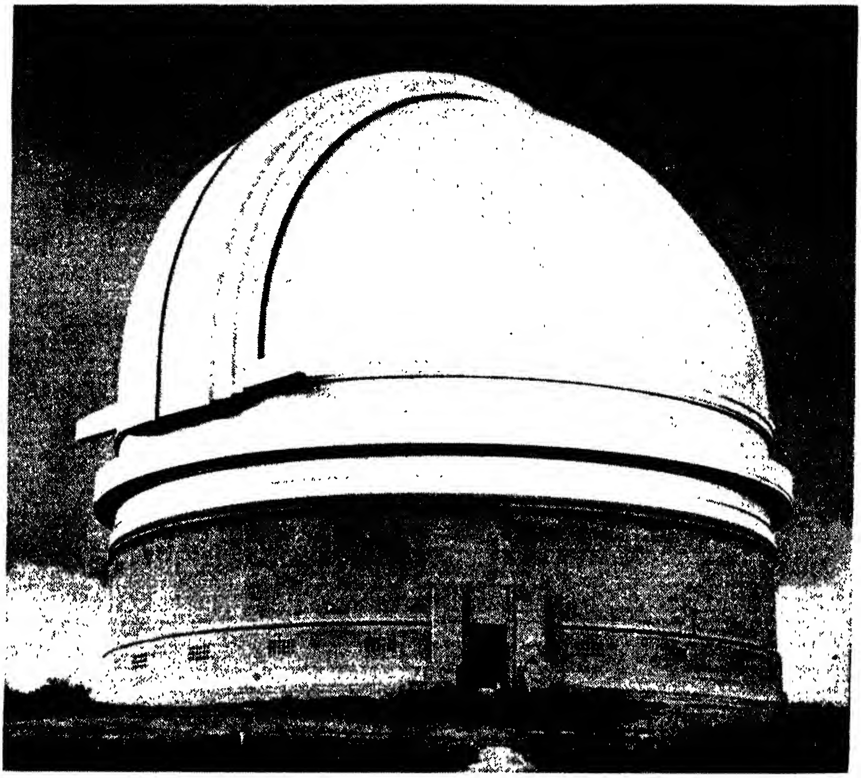
Highway bridges are designed for truck loads or for distributed lane loads estimated to provide stresses equivalent to those caused by the anticipated traffic. Truck loadings are designated by the symbols H20, H20-S16, etc. These designations are adopted by the AASHO specifications and indicate the weight and distribution of that weight over wheels of the trucks. An H20 truck is a 20-ton truck, while H15 and H10 indicate 15- and 10-ton trucks, respectively. The symbol S16 indicates that the H20 truck is followed by a trailer having an axle load equal to the load on the rear axle of the truck.

Railway loading is designated most commonly by a Cooper's standard loading, such as E-60, E-72, etc. The Cooper's standard loadings indicate the distribution of load on wheels by amount and spacing. At the present time, the majority of railway bridges are designed for Cooper's E-72, which provides a live load 80 per cent heavier than the E-40 commonly used prior to 1900. While there are no locomotives having weight distribution like the Cooper's E-loadings, it is usual to classify the actual locomotives, in terms of their effect on the structure, as equivalent to a Cooper's loading. In this way, the Cooper's loadings are successfully used for both design and classification of structures.

The *American Standard Building Code*, A58.1—1945, sponsored by the National Bureau of Standards, recommends live loads for buildings. Provisions of this Code are given in the AISC handbook. Floor live loads are varied, according to the type of occupancy, from 40 to 125 lb per sq ft or more. Partial loading, future changes due to rearrangement of partitions, impact loadings, and concentrated loads are considered in the Code.

The design of structures for industrial use, manufacturing plants, power houses, and shops requires special consideration. In each case the use and selection of equipment will determine the loading to be used.

1-11. Wind Load. The *AREA Specifications for Railway Bridges* specify that wind load is to be computed on the vertical projection of certain areas at 30 lb per sq ft when loaded, and at 50 lb per sq ft when not loaded. Wind is also considered to be acting on the train. Minimum wind forces are specified for each chord of trusses.



Bethlehem Steel Co.

MT. PALOMAR OBSERVATORY, CALIFORNIA.

The provisions of the *AASHO Specifications for Highway Bridges* are similar to those of the AREA specifications, except in certain details.

The AISC *Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings* suggests the use of 20 lb per sq ft on the vertical projection of a building. It then permits unit stresses to be increased by one-third for members designed for wind loads, or for combinations of wind with other loads, provided that the resulting section is not less than that required for a combination of dead load, live load, and impact.

Exact analysis of the force produced by wind on a vertical plane is complicated by the nature of the wind, which usually occurs in gusts. Considering that air has a density of 0.072 lb per cu ft at sea level and at 60F, a value of the force of the wind might be compared to the momentum of a stream of fluid striking a flat surface. In this case,

$$MV = \frac{dV^2}{g}, \text{ or } \frac{0.072V^2}{32.16} = 0.00224V^2,$$

when V is in feet per second. When V is expressed in miles per hour the force $p = 0.0048V^2$. Various experimenters have obtained values by measurement for $p = 0.005V^2$ to $0.0032V^2$, depending on the size of the area investigated. As larger areas were used, the average intensity of pressure was found to decrease.* It is reasonable to expect that, as the size of the area affected is increased, the cushioning effect of the air which cannot escape will reduce the value of the full impact and thus reduce the intensity of pressure. Also, the highest winds are normally accompanied by a drop in barometric pressure, reducing the air density.

A committee of the American Society of Civil Engineers† recommended that the value of the intensity of wind pressure be computed as $q = 0.002558V^2$, when V is in miles per hour. They also pointed out that a suction exists on the leeward side of a building, and suggested that $0.8q$ may nearly represent the pressure on the windward side and $0.5q$ the suction. The total intensity in the direction of the wind would then be $1.3q$. For a wind velocity of 77.8 mph, the value of $q = 15.5$ and the total intensity $p = 20$ lb per sq ft.

The graph of Figure 1–6 is a representation of the recommendations of the ASCE committee for the amount of the normal pressure on sloped roofs. This pressure is seen to vary from a suction of 12 lb per sq ft to a positive pressure of 9 lb per sq ft, depending on the slope of the roof surface from the horizontal. The committee also recommended the use of a suction of 9 lb per sq ft on the leeward side for all sloped roofs.

As a result of their studies the committee recommended the use of internal pressures varying from 4.5 lb per sq ft to 12 lb per sq ft and internal suctions varying from 4.5 lb per sq ft to 9 lb per sq ft, depending on the percentage of the wall surfaces that are open or that might become open. The lesser pressures and suctions are intended for airtight buildings and the larger values for buildings with 30 per cent or more openings. When the openings are on the windward side of the building the internal forces are pressure, and when the openings are on the leeward side, the forces are suction. It is further suggested that an external suction of 9 lb per sq ft be assumed to act on external walls that are parallel to the direction of the wind.

The ASCE committee recommended a uniformly distributed wind force of 20 lb per sq ft for the first 300 ft above ground level, increased by 2.5 lb per sq ft for each additional 100 ft of height, and also recommended that special provisions be made in localities subject to hurricanes or tornadoes.

1–12. Impact. Impact is usually defined as the dynamic load produced by sudden application of live load. Possibly a more nearly complete definition would be that impact is that part of the stress in a member which must be added to the static live-load stress to give a value for the total

* *Proceedings of Institute of Civil Engineers*, Vols. 156 (1903–1904) and 121 (1907–1908).

† "Wind Bracing in Steel Buildings," Final Report of Sub-committee No. 31. Committee on Steel of the Structural Division, *Trans. ASCE*, Vol. 105 (1940), p. 1713.

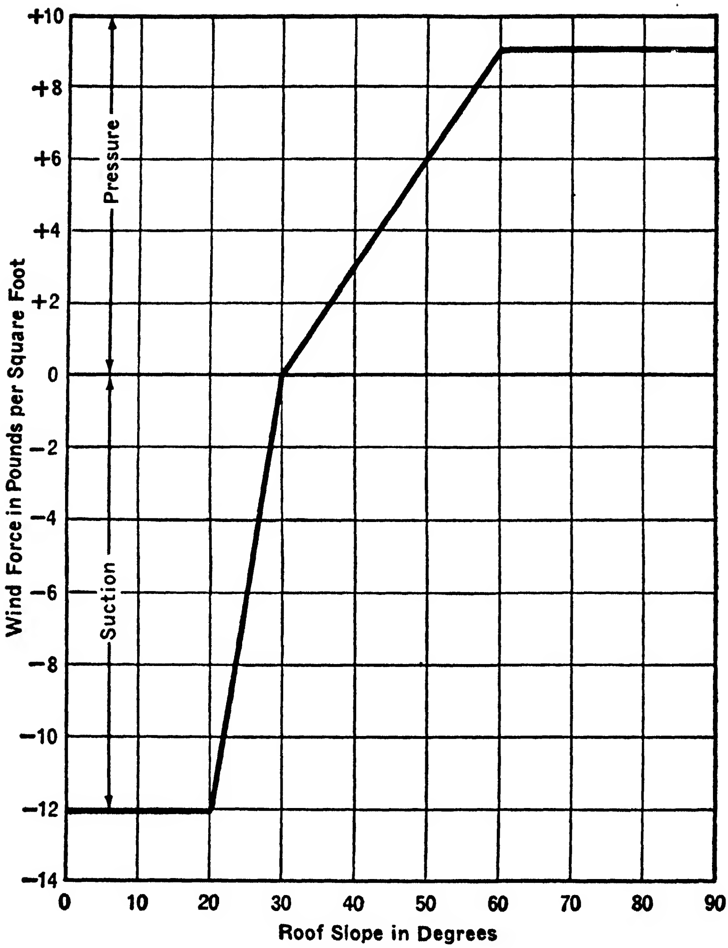


FIG. 1–6.

stress due to live load. This impact stress may be partially due to sudden application of live load, but in many instances it is due to other causes. For example, as a railway locomotive crosses a bridge, the rail loads are increased or decreased by the sway of the locomotive. Open rail joints cause sudden impact forces, and counterbalances on the drive wheels produce a pulsating type of load which varies in intensity and frequency with the speed of the locomotive. If the rate of application of the pulsating loads synchronizes with the period of vibration of the structure, the greatest impact effects will result. Factors which have an effect on the period of vibration of a structure include the weight, span length, type of structure, type of end supports, and material.

Specifications for impact allowance have changed greatly in recent years because of information developed through research. A committee of the American Railway Engineering Association* has accumulated data from tests of a large number of bridges. These tests and the theoretical development by some of the members of the committee have been responsible in large measure for the changes in the AREA specifications.

In 1915 the AREA specifications required an allowance for impact of

$$I = S\frac{300}{L + 300}$$

in which I is defined as the strain that is to be added for impact, S is the strain due to live load, and L is the loaded length of the bridge. The 1955 AREA specifications divide the impact allowance into two parts—the rolling effect and the direct vertical effect. The rolling effect is assumed to increase the load on one rail by 20 per cent and to decrease the load on the other rail by 20 per cent. The direct vertical effect for steam locomotives is specified as a percentage of the live load equal to $60 - L^2/500$ for spans less than 100 ft in length, and $1{,}800/(L - 40) + 10$ for spans of 100 ft or more, in which L is the length of the span or member considered for beam spans, stringers, girders, floor beams, posts of deck truss spans carrying load from floor beam only and floor beam hangers. The direct vertical effect for truss spans is $4{,}000/(L + 25) + 15$. The impact allowance for electric and diesel locomotives is less than for steam locomotives.

The impact allowance for highway bridges is considerably less than for railway bridges, which is reasonable because of the nature of the loading. The AASHO specifications provide for an impact fraction of $I = 50/(L + 125)$, in which L is the loaded length producing the maximum live-load stress in a member. The maximum percentage is 30 per cent of the live load.

The AISC specification provides that impact allowances from 20 per cent to 100 per cent be allowed in buildings where live loads are of types inducing impact or vibration.

1–13. Factor of Safety. The factor of safety provided in a structure is usually considered to be the number of times the load might be increased before the structure would fail because of stress. It might be reasoned that, if it were possible to determine exactly the amount of load and its effect on the structure, the factor of safety could be greatly reduced. One of the uncertain factors in the design of many structures is the manner in which the various forces combine. For example, can it be expected that a bridge will receive its maximum live load, impact, and wind load simultaneously with such forces as those produced by temperature change, traction, or

* "Report of Special Committee on Impact," *Proc. AREA*, Vol. 46 (1945), p. 189.

braking effect? Is it likely that all floors of a building will be loaded fully at one time?

Specifications generally provide either increases in allowable design stress with certain combinations of load or they limit the combinations for which provision is to be made.

In most steel structures, the allowable design stresses are established with a factor of safety with respect to the yield point of the material. In the case of aircraft structures, the factor of safety is multiplied by the anticipated maximum load, and the resulting stresses are compared with the ultimate value of the metal; the percentage that the computed stress (including a factor of safety) could be increased to equal the ultimate stress is called a *margin of safety.* For example, if a factor of safety of 1.5 is specified, and if the stress resulting from a given flight condition multiplied by 1.5 is 95 per cent of the ultimate stress of the material, it is said that a margin of safety is

$$\text{M.S.} = \frac{100}{.95} - 1 = 0.052$$

In the rating of existing bridge structures, the AREA specifications permit higher unit stresses than in the design of new structures. This is because the existing structure is being operated under known conditions of loading and renewal could not be economically accomplished until the useful life of the bridge has been nearly expended.

CHAPTER 2

RIVETS, BOLTS, AND PINS

2–1. Assumptions Used in the Design of Riveted Connections. Certain assumptions are made in the design of riveted connections, most of which are either amply substantiated by tests or provide conservative design for static loads. Some of the assumptions are the subjects of continuing tests and study in connection with loads causing repetition of stress.

The usual assumptions are as follows:

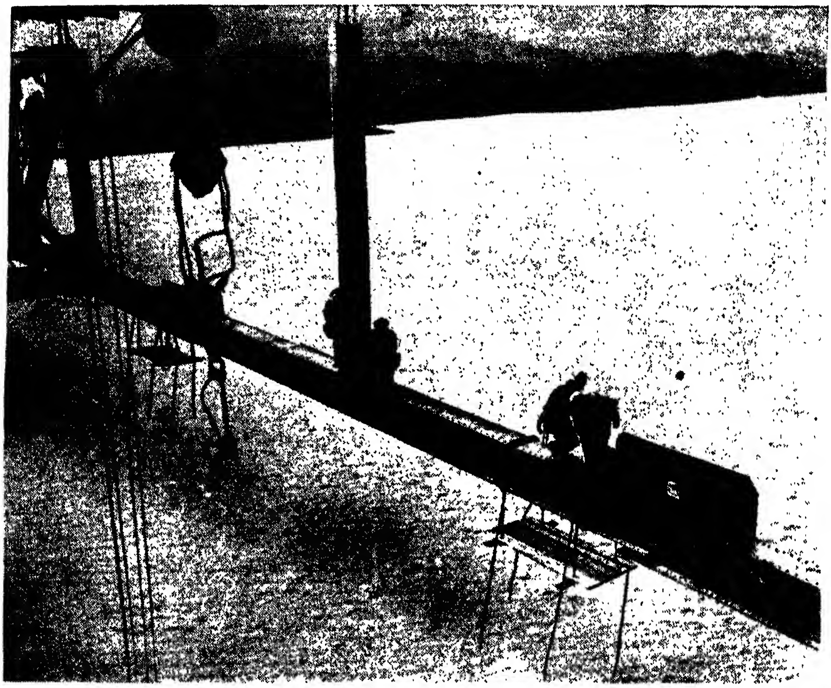
1. Resistance due to friction between connected plates is disregarded. Considerable friction exists because of tension caused by cooling of hot-driven rivets, or by the driving pressure used in the installation of cold rivets. Tests indicate that loads approximating the design values may be applied to a specimen before slip between the plates takes place. This friction will reduce the load necessary for the rivets to transmit.

2. All rivets in a group are assumed to be equally capable of carrying the same load. It may be shown that the deformations of the materials connected will result in certain rivets of a group receiving more load than others. Tests* have shown that unequal distribution exists at design loads and that, when the friction between plates has been broken down and part of the material has been stressed to the yield point, redistribution takes place. At the ultimate value of the connection it has been found that the distribution of load between rivets agrees quite well with the assumption of equal distribution. Possibly the assumption is not entirely conservative for joints which receive repeating loads, causing certain rivets to receive loads somewhat less than the yield point.

3. It is assumed that all rivets of a group completely fill the holes into which they are driven and come into bearing simultaneously. If certain rivets fail to fill the holes, these rivets may receive less load until a certain amount of deformation has taken place. With the high-driving pressures used and careful inspection given to rivets, there is little probability of trouble from the lack of uniformity of fit. Shrinkage due to cooling of hot-driven rivets will result in some space around each rivet. A similar but probably lesser reduction in diameter occurs on removal of the driving pressure used with cold-driven rivets. Misalignment of holes in the connected plates will result in improper filling of holes.

4. It is assumed that tension is distributed uniformly over the net section of a plate. Photoelastic studies show definite increase in stress intensity at the edges of holes through a tension member. However, as a tension member is loaded beyond the yield point of the material, a redistribution

* R. E. Davis, G. B. Woodruff, and H. E. Davis, "Tension Tests of Large Riveted Joints," *Proc. ASCE*, May, 1939.



American Bridge Co.

DELAWARE MEMORIAL BRIDGE (SUSPENSION) NEAR WILMINGTON, DELAWARE.

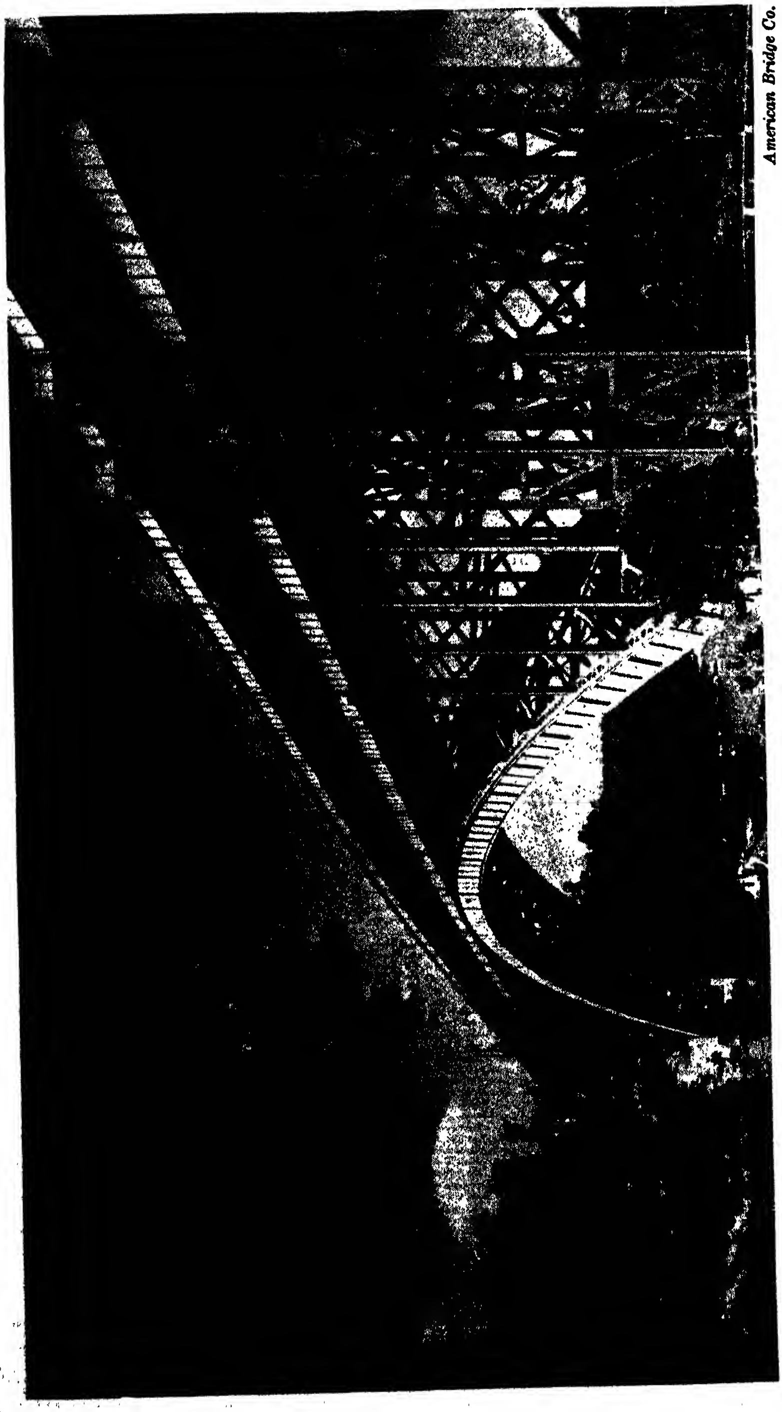
occurs, and the ultimate load is usually consistent with the assumption. Again, repeated applications of load causing average stress somewhat below the yield point result in failure of the tension member with sufficient applications, and the failure usually begins by cracks from the edge of rivet holes.*

The hole for a steel rivet is made $\frac{1}{16}$ in. greater in diameter than the nominal diameter of the rivet, while the reduction of cross section required by specifications is on the basis of a hole $\frac{1}{8}$ in. larger than the nominal diameter of the rivet. This reduction increases the computed average stress and partially compensates for the unequal distribution.

5. The acting area of compression members is considered to be unreduced by the insertion of rivets. Although the holes may not be completely filled by the rivets, the nature of the deformation under compression is in the direction which tends to reduce the defect.

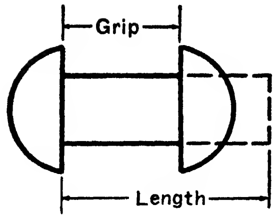
2-2. Types of Rivets. Rivets used in structural steel connections are usually of the button-head type. (See Fig. 2-1.) Occasionally clearance

* "Tests of Riveted and Welded Joints in Low-Alloy Structural Steels," *University of Illinois Engineering Experiment Station Bulletin*, No. 337 (1942). "Net Section of Drilled Plates," *University of Toronto School of Engineering Research Bulletin*, No. 6 (1926).

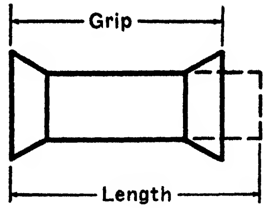


American Bridge Co.

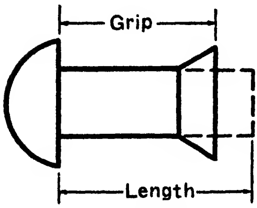
HENRY HUDSON BRIDGE ACROSS HARLEM RIVER, N.Y. DOUBLE-DECK HIGHWAY, HINGELESS ARCH.



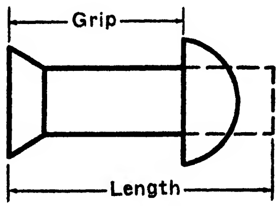
Button-Head Rivet with
Driven Button Head



Countersunk Rivet with
Driven Countersunk Head



Button-Head Rivet with
Driven Countersunk Head



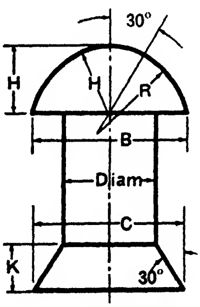
Countersunk Rivet with
Driven Button Head

DIMENSIONS OF RIVETS

FORMULAE

Diam head $B = 1.5D + \frac{1}{8}$
Height of hd. $H = 0.425\ B$
Long rad. $R = 1.5\ H$
Short rad. $= H$

Depth of countersink $K = 0.5\ D$



Diam of Rivet	Button Head Diam B	Button Head Height H	Button Head Radius R	Countersunk Diam C	Countersunk Height K
$\frac{3}{8}$	$\frac{11}{16}$	$\frac{5}{16}$	$\frac{7}{16}$	$\frac{9}{16}$	$\frac{3}{16}$
$\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{8}$	$\frac{9}{16}$	$\frac{3}{4}$	$\frac{1}{4}$
$\frac{5}{8}$	$1\frac{1}{16}$	$\frac{7}{16}$	$\frac{11}{16}$	1	$\frac{5}{16}$
$\frac{3}{4}$	$1\frac{1}{4}$	$\frac{1}{2}$	$\frac{13}{16}$	$1\frac{3}{16}$	$\frac{3}{8}$
$\frac{7}{8}$	$1\frac{7}{16}$	$\frac{5}{8}$	$\frac{15}{16}$	$1\frac{3}{8}$	$\frac{7}{16}$
1	$1\frac{5}{8}$	$\frac{11}{16}$	1	$1\frac{9}{16}$	$\frac{1}{2}$
$1\frac{1}{8}$	$1\frac{13}{16}$	$\frac{3}{4}$	$1\frac{1}{8}$	$1\frac{3}{4}$	$\frac{9}{16}$
$1\frac{1}{4}$	2	$\frac{7}{8}$	$1\frac{1}{4}$	2	$\frac{5}{8}$
$1\frac{3}{8}$	$2\frac{3}{16}$	$\frac{15}{16}$	$1\frac{3}{8}$	$2\frac{3}{16}$	$\frac{11}{16}$
$1\frac{1}{2}$	$2\frac{3}{8}$	1	$1\frac{1}{2}$	$2\frac{3}{8}$	$\frac{3}{4}$

FIG. 2-1. Standard steel rivets

requirements make it necessary to flatten the heads somewhat below the height of the standard button-head rivets. Other connections make a flush surface desirable, and the countersunk rivet is installed. Countersunk rivets are given less load value and require removal of more of the connected material than is required for installation of the button-head type.

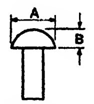
The rivets used in structural steel are usually driven hot. Care must be exercised in the heating of rivets so as to have the rivets heated uniformly throughout their length and not overheated and burned. The manufactured head is "bucked" in a dolly or bucking bar, holding the hot rivet firmly in place while the riveting tool upsets the rivet and forms a head on the other end. It is necessary to upset the shank of the rivet throughout the entire length to fill the hole tightly. Practically all rivets are driven by pneumatic or hydraulic power machines.

Considerable success has been attained in driving cold rivets up to 1 in. diam and as much as 7 in. in length. Special equipment is required for driving cold rivets of large size. The equipment manufacturers claim that better rivets, more nearly filling the holes, are obtained by cold driving. This method is not used extensively at the present time.

The most common sizes of rivets in steel structural work are $\frac{3}{4}$ in., $\frac{7}{8}$ in., and 1 in. diam, with $\frac{5}{8}$-in. and $1\frac{1}{8}$-in. rivets being used in exceptional cases.

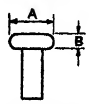
In aircraft structural connections, cold rivets of sizes from $\frac{3}{32}$ in. to $\frac{12}{32}$ in. diam are used. Sizes from $\frac{3}{32}$ in. to $\frac{5}{32}$ in. are usually of 2017-T4 aluminum alloy, while the larger rivets are of 2024-T4. The 2024-T4 rivets are heat treated with the least possible interval of time before driving and are kept on dry ice to retard the hardening process. They must be driven within a very short interval of time after removal from the icebox to ensure good rivets. After driving, the hardening process continues until full development of the effects of heat treatment, which is in about four days' time in normal temperatures. The temper of the material is affected by the driving process, the 2017 alloy becoming 2017-T31 and the 2024 alloy, 2024-T31. The probable shearing strength for the 2017-T31 rivets is from 33,000 to 39,000 psi and for the 2024-T31 rivets approximately 42,000 psi. Rivets of the alloys 6061-T4 and 7277-T4 may be driven hot. The full shear value of these rivets is not obtained as quickly as when driven cold.

The AN430 raised-head rivet shown in Fig. 2–2 is a commonly used rivet for structural connections not in the airflow. The Brazier, or a rivet with flatter head, is frequently used for exterior surfaces to reduce the obstruction to airflow. When it is desired to reduce the parasite drag to a minimum, flush-type riveting is employed. Holes for flush rivets are prepared in one of three general ways. Where the thickness of material will permit, the holes are countersunk and the countersunk rivet used. Some standards permit skins of 0.040 in. thickness to be countersunk, but it is considered better practice not to countersink thicknesses less than 0.051 in. or even 0.064 in. To obtain flush surfaces with thin skins, the counterpunched or "dimpled" hole is prepared for the countersunk rivet. Special rivets are frequently used which more nearly conform to the shape of the dimple. If the understructure has sufficient thickness it may be countersunk and the dimpled skin seated into the countersink. Otherwise, all layers through which the rivet passes will be dimpled and nested. The counterpunching or dimpling is done with pneumatic tools, which force the material to the contour desired. Considerable trouble is frequently encountered because of worn tools, improper pressure or speed, and failure to burr the drilled or punched holes before dimpling. Poor workmanship of this type results in cracked edges on the dimple (making repair necessary) or in leaving a source of potential cracks.



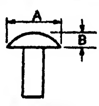
AN 430

SIZE	A	B
3	0.188	0.070
4	0.250	0.094
5	0.313	0.117
6	0.375	0.141
8	0.500	0.188
10	0.625	0.234
12	0 750	0.281



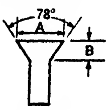
AN 442

SIZE	A	B
3	0.188	0.038
4	0.250	0.050
5	0 313	0.062
6	0.375	0.075
8	0.500	0.100
10	0.625	0.125
12	0.750	0.150



AN 455

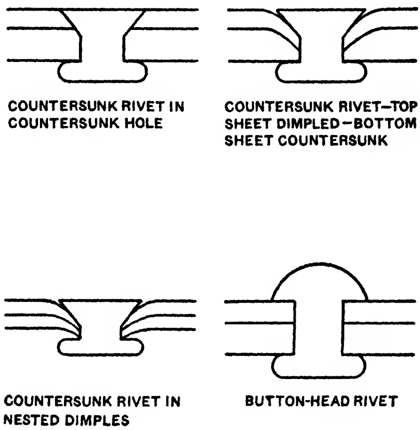
SIZE	A	B
3	0.234	0.031
4	0.313	0.047
5	0.391	0.063
6	0.469	0.078
8	0.625	0.109
10	0.781	0.141
12	0.938	0.172



AN 425

SIZE	A	B
3	0.170	0.047
4	0.225	0.062
5	0.282	0.078
6	0.339	0.094
8	0.452	0.125
10	0.565	0.156
12	0.678	0.187

Aircraft Rivet Head Dimensions



Typical Rivet Installations

Rivet size is diameter of shank in $\frac{1}{32}$-in..

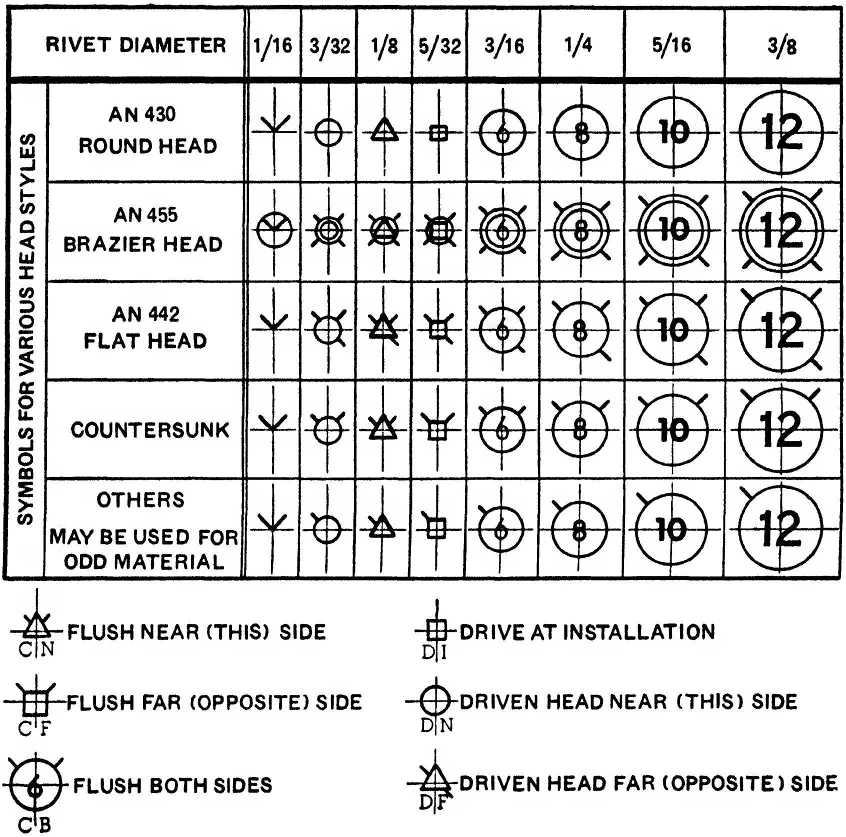
FIG. 2-2. After Boeing aircraft standards.

2-3. Rivet Designations. Conventional symbols for rivets are well established for use on drawings of steel details. Familiarity with the symbols will permit immediate recognition of the type of rivet and determination of whether it is to be installed in the shop or during erection of the steel. Figure 2-4 shows the designations common to structural steel drawings.

Symbols for rivets on aircraft construction vary somewhat between manufacturers, but the designations shown in Fig. 2-3 are typical.

2-4. Functions of Bolts. Bolts are used in structural steel construction as a temporary means of connecting members, for anchoring a structure to masonry, for taking tensile forces beyond the capacity of rivets, and for permanent connections of some lower-stressed members in isolated locations. Frequently a bolt may be installed in a location where it is impossible to drive a rivet or where it is impossible to place sufficient rivets to carry the shear.

In aircraft work, bolts are used for connecting equipment and interchangeable parts which may need to be replaced in service. They are occasionally employed where sufficient rivet value cannot be obtained.



IDENTIFICATION		COMPOSITION	MINIMUM SHEARING STRESS	H.T. BEFORE USING
	NO IDENTIFICATION	MONEL	40,000	NO
	NO IDENTIFICATION	A(2S½H)	10,000	NO
	RAISED CROSS	B(56SH)	27,000	NO
	DIMPLED CENTER	AD(A17ST)	27,000	NO
	RAISED TEAT	D(17ST)	30,000	YES
	DOUBLE DASH	DD(24ST)	35,000	YES

FIG. 2-3. After Boeing aircraft standards.

Bolts used in aircraft work are usually of heat-treated steel having ultimate tensile values from 125,000 psi to 180,000 psi.

The tensile value of a bolt is determined by the cross-sectional area at the root of the threads. Because of the stress concentrations caused by the shape of the threads, somewhat lower unit stresses are sometimes permitted at the root of threads than are used for tension in members of uniform section.

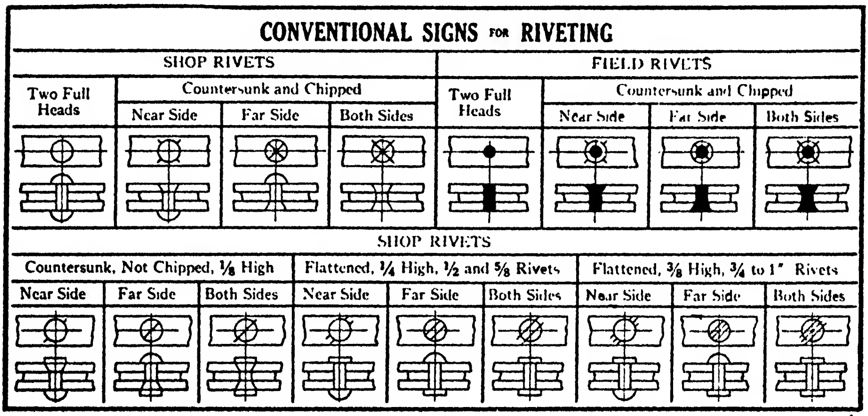


FIG. 2-4.

If a bolt is properly installed, the cross section subjected to shear will be through the shank or full area of the bolt.

When bolts are used for fitting up structural members in preparation for riveting, it is usual to use them in combination with drift pins. The drift pins more nearly fill the holes and ensure proper alignment, while the bolts hold the connecting parts securely together. Approximately half of the holes filled would contain drift pins.

It is generally considered that, when bolts are used in a connection in combination with rivets, the bolts will not take shear. This is because the bolts usually have a loose fit. In aircraft work, where it is necessary that the bolts function in shear, the holes will be prepared with close fit to the bolts—in some cases a light drive fit being required.

Turned bolts with close fit are frequently specified for steel assemblies where shearing forces must be resisted.

2-5. High-Strength Bolts. The use of high-strength bolts rather than rivets is a relatively new development. The bolts are made from a high-tensile steel, such as ASTM designation A325, with an ultimate tensile strength of 120,000 psi and an elastic limit equal to 70 per cent or more of the ultimate. The bolts are installed in holes 1/16 in. larger than the bolts and tightened with impact wrenches or manual torque wrenches until the bolts are stressed to about 55,000 psi in the bolt shank. This results in drawing the connected parts tightly together.

Since the bolts do not fill the holes in which they are installed, the action of the joint depends on friction between the connected parts, resulting from the tension in the bolts.

The action of joints using high-tensile steel bolts and those using hot-driven rivets may be compared. The hot-driven rivet made from steel conforming with ASTM designation A141 has a yield-point stress of 28,000 psi, which is the probable upper limit of useful tension developed when the rivet cools. Slip between the connected parts does not occur until the value of the friction between the connected parts is exceeded by the applied load. The hot-driven rivet is forged to fit the hole into which it is driven; when it cools, the diameter of the rivet reduces slightly, probably in the order of 0.001 in. per in. of diameter. After slip between the connected parts occurs the rivets are presumed to take equal load.

The connection that utilizes high-strength bolts has a higher friction between the connected parts, and slip between the parts will probably not occur until the applied load is about double that required to cause slip when the joint is riveted. After slip has occurred the bolts are required to transfer the loads by shear. Since the bolts are in holes that are larger than the bolts, and the bolts are not exactly centered in the holes, it is not probable that the bolts will receive equal loads. In case of load reversal, the connected parts presumably move after slip has occurred.

In either case the amount of friction between the connected parts is usually adequate to prevent slip when the loads are at the design values.

In some cases it is more economical to use high-strength bolts for field connections than to use rivets.

2–6. Preparation of Holes for Rivets and Bolts. In order that a rivet or bolt may be installed properly and function in shear as assumed, it is necessary that the holes in connecting members be matched and cylindrical.

Detail shop drawings show the exact locations for holes in the parts which would be accomplished with perfect shop work. Permitted tolerances in shop layout and elongation of material during punching or drilling operations determine how nearly the ideal can be accomplished.

Holes may be punched or drilled. When the thickness of the material is not too great, punching may be used. The limit of thickness for successful punching is usually about $\frac{1}{8}$ in. greater than the diameter of the hole being punched. Holes for turned bolts are usually subpunched or subdrilled to a smaller diameter than the bolt and are reamed to the correct size. Specifications vary in requirements for preparation of rivet holes. Bridge and aircraft specifications usually require a higher type of workmanship than do building specifications.

Templets may be used for drilling holes full size. The same templet

Bethlehem Steel Co.

BEAM TO COLUMN CONNECTIONS USING SHOP RIVETS AND FIELD BOLTS.

would then be used on all matching parts. A large part of bridge work must have the rivet holes subpunched or subdrilled and reamed while the parts are assembled. Certain important parts are required to be drilled from the solid while assembled. Aircraft matching parts are either jig-drilled or drilled while assembled, with one part having pilot holes of small size as the guide.

The requirement for reaming while assembled increases shop costs. Bridge specifications list the parts requiring this quality of workmanship. The type of loading received and the number of matching layers of material are factors in determining whether assembled reaming is sufficiently necessary to justify the extra shop cost.

2-7. Pin Connections. Prior to 1912 a majority of the bridges built in the United States were pin-connected. A typical pin connection in a top chord is shown in Fig. 7-5. Since about 1912 the American practice has changed so that practically all bridge members are connected by riveting. In very large bridge structures a few connections are still made with pins and pins are used for special purposes where some angular change between the members is anticipated.

In pin connection design, primary considerations are bearing on the pins, shear and bending of the pins, and net section of the connecting members.

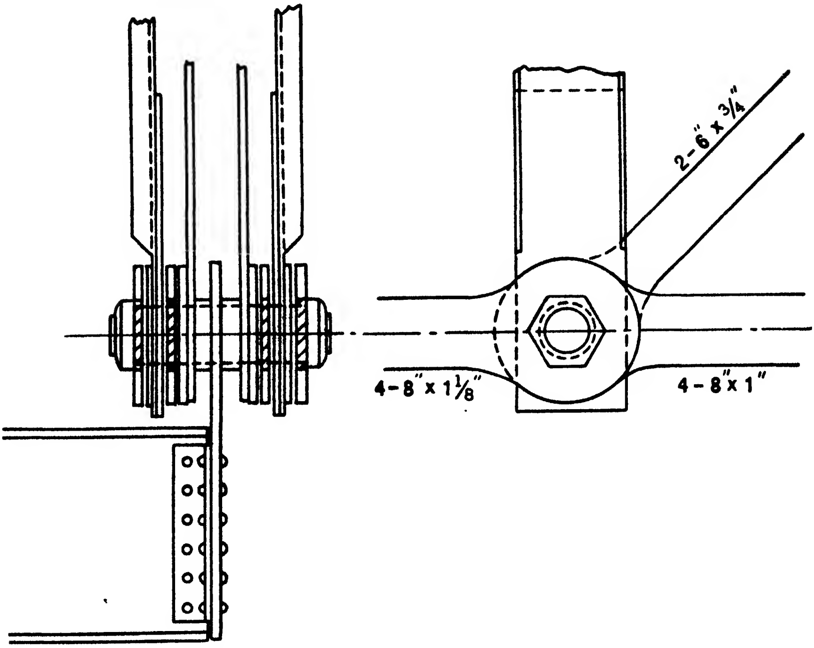


FIG. 2-5. Pin connection at lower chord panel point of truss, illustrating an old style of construction.

Figure 2-5 shows a bottom chord pin connection. Each member entering the joint must have bearing area on the pin sufficient to transmit its design load. The pin must be investigated for bending moment and shear produced by the combination of loads transmitted by the members. The nature of the forces acting on such a pin is shown in Fig. 2-6.

The design of eyebars and other tension members is governed largely by empirical specifications, which are based on provision for the stress concentrations existing near a hole in a tension member.

The AREA specification requires that eyebars shall be not less than 1 in. or more than 2 in. thick. The section of the head through the center of the pin hole shall exceed that of the body of the bar by at least 35 per cent.

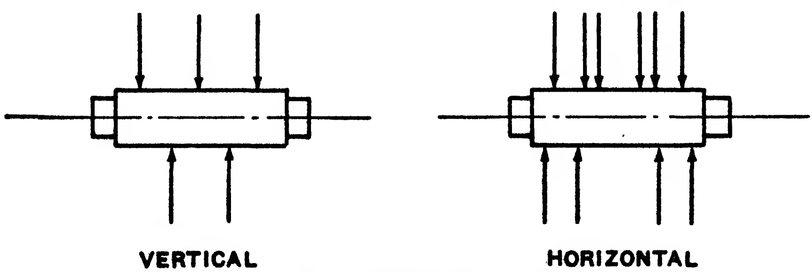


FIG. 2-6. Forces acting on a pin.

The diameter of the pin shall be not less than $\frac{8}{10}$ of the width of the widest bar attached.

The specification also requires that the eyebars of a set shall be symmetrical about the central plane of the truss and as nearly parallel as possible.

For other pin-connected tension members, the net section beyond the pin hole, parallel with the axis of the member, shall be not less than the required net section of the member. The net section through the pin hole, transverse to the axis of the member, shall be at least 40 per cent greater than the required net section of the member. The ratio of the net width (through the pin hole transverse to the axis of the member) to the thickness of the segment shall not be more than eight to one.

CHAPTER 3

WELDING

3–1. Welding Processes. The *Standard Master Chart of Welding Processes*, published by the American Welding Society, lists eight general groups of welding processes which are further classified into 37 subdivisions.

The eight principal groups of welding processes include brazing, flow welding, resistance welding, induction welding, arc welding, thermit welding, gas welding, and forge welding. Brazing includes a group of welding methods in which a nonferrous filler metal is melted at a temperature lower than the melting point of the parts being connected and the attachment is accomplished by capillary attraction of the metals. The flow-welding processes are accomplished by pouring melted metal over the joint to be welded until the connection is heated to welding temperature.

In forge welding the members are heated and the weld is made by means of pressure or hammer blows. A common type of forge welding is that done by the blacksmith. The heat required in resistance welding is supplied by the resistance of the material being welded to an electric current passed through the parts; pressure is used to force the parts together. Induction welding is a process in which coalescence is produced by the heat obtained from resistance of the work to the flow of induced electric current, with or without the application of pressure.

A group of welding processes known as *thermit welding* is accomplished by placing superheated liquid metal and slag, resulting from a chemical reaction between a metal oxide and aluminum, around the parts to be joined. Gas welding is done with gas flames used to melt the metal, and with or without force being applied. The heat required in arc welding is produced by an electric arc; some of the processes use pressure, and metal may or may not be added.

The welding processes that are most important to the structural engineer are arc and gas welding. The arc welding method has two principal subdivisions. These are classified as carbon electrode and metal electrode. *Specifications for Welded Highway and Railway Bridges* and *Code for Arc and Gas Welding in Building Construction*, prepared by the American Welding Society, are applicable to the arc and gas welding processes. A majority of structural welding is done by the electric-arc method.

Gas welding is usually accomplished by the use of the oxyacetylene flame and the addition of weld metal. The oxyacetylene flame is also an important tool for flame cutting and for the preparation of material for arc welding.

The usual carbon-arc process is a process in which an electric current is

Chicago Bridge and Iron Co.

10-Ft Diam Welded Steel Tanks, 30 Ft Long.

passed through a carbon electrode and the material to be welded. The operator holds the electrode at the correct distance from the work to cause an electric arc between the work and the electrode. When the parent metal has reached a plastic heat, a welding rod is inserted in the arc. The welding rod provides the additional metal necessary to build up a bead of metal to reinforce the weld.

The more common type of electric-arc welding is the method which employs the welding rod as the electrode. The welding rod melts because of the high temperature caused by the electric arc. This molten metal from the welding rod provides the additional metal necessary for the process.

3–2. Arc Welding. Electric-arc welding may be accomplished by the use of current from a direct-current power line passing through a resistor to reduce the voltage and current. Because of the limited availability of such electric current and in order to meet field requirements for portability, many electric welding machines have been developed. The welding machine is essentially a direct-current generator with voltage and current control.

In the metal-arc process an electric cable passes from the generator to the welding rod held by the operator; the current passes from the welding rod through the metal being welded and thence through a cable connecting the work with the generator. The operator makes a contact between the welding rod and the area to be welded, causing current to flow, and then draws the rod slightly away from the work to cause an arc to form. The heat developed by the arc melts the welding rod and base metal or welded parts, which causes the metal to fuse and build up.

Because of the varying length of the arc formed by the operator, voltage requirements vary, and the welding machine controls must automatically supply the correct voltage. In factory processes, use is frequently made of the automatically controlled welding apparatus which holds the electrode, eliminating this hand operation. Such apparatus eliminates much of the voltage variation and provides uniformity in the welding operation. The apparatus is useful principally where a particular operation is to be repeated many times. A majority of structural welding is performed by hand-operated arcs.

Many different welding rods are manufactured to provide for varying requirements, such as those caused by differences in materials, positions of work—overhead, vertical, or flat—and the size of the weld material to be added. Because of the superior quality of the resultant weld, most specifications require the use of coated rods. The coating is melted by the heat of the arc and provides a vapor shield around the area being fused. This vapor shields the molten metal, preventing its contamination from contact with the air, and confines the atmosphere of the arc, permitting more nearly complete ionization of the metal particles forming the arc.

The coating serves as a flux carrying the impurities to the surface of the molten metal and forms a slag coating over the weld to protect it from the air during the cooling process. Figure 3–1 is a diagram showing the principal elements of the shielded metallic arc. The slag is removed after the weld has cooled; since the slag is brittle, it breaks up very easily with light tapping of a hammer.

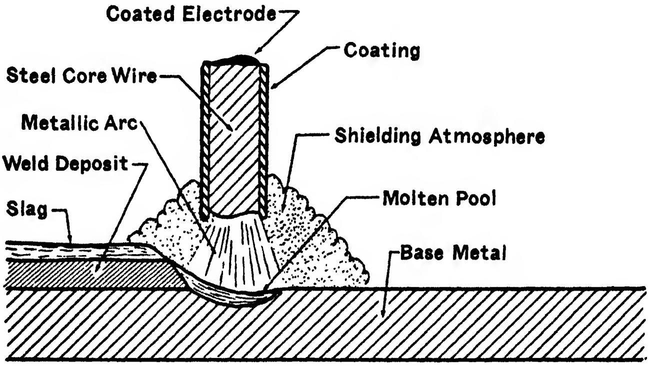


FIG. 3–1.

3–3. Types of Welds. Figure 3–2 shows the basic types of structural joints. Two principal types of weld are indicated in these joints—the fillet weld and the butt weld. Because of the manner in which the welded parts meet, these welds have various shapes.

The edges of the plates to be welded are usually prepared by beveling, by cutting to a J-shape, or by the formation of a U. These shapes are used to provide access and to ensure better fusion of the base metal.

The fillet weld is the type used most extensively. Figure 3–4 indicates two plates connected by the use of fillet welds. The strength of the welded joint is measured by the shear value of the fillet weld per linear inch times the number of inches of weld. The *AWS Code for Building Construction* permits 13,600 psi on the throat of fillet welds. As shown in Fig. 3–3, the throat is the minimum cross section of the weld material. A $\frac{5}{16}$-in., 45° fillet weld would have a throat of 0.22 in. and a shear value of $0.22 \times 13{,}600 = 3000$ lb per lin in. If the weld is 10 in. long, a shear value of 30,000 lb would be developed by the $\frac{5}{16}$ in.-weld. The size of the fillet weld used depends on the force to be transmitted or the thickness of the plates or shapes being welded. However, a majority of welds are $\frac{5}{16}$ in. in size. Smaller welds are difficult to lay, and this size meets the strength requirements in a large number of cases. Drawings usually specify that all welds be of a particular size, unless shown otherwise. The $\frac{5}{16}$-in. size is convenient and economical, when sufficient room is available to develop the connection, because the fillet may be formed by a single pass. Larger fillets require two or more passes. Figure 3–5

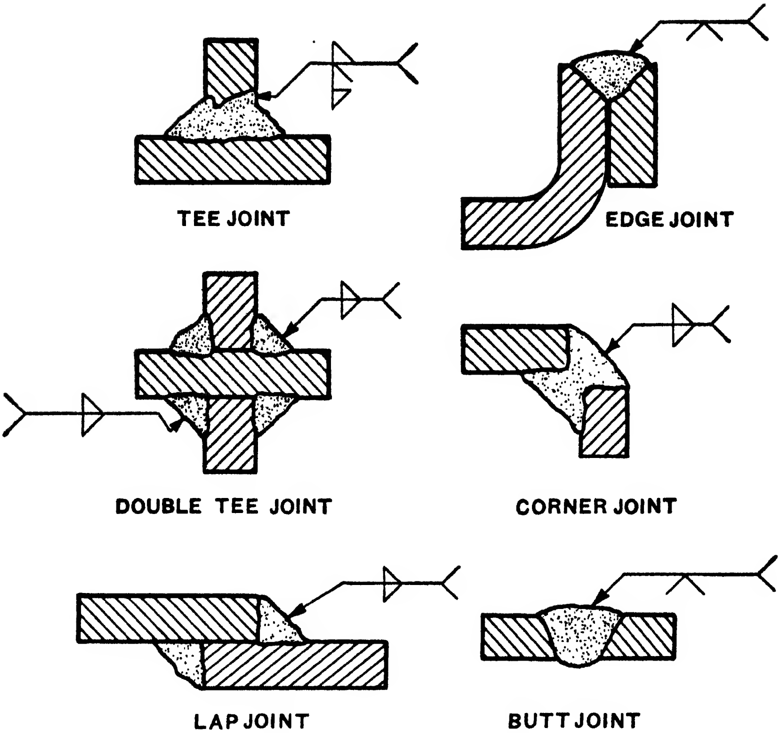


FIG. 3-2. Types of joints.

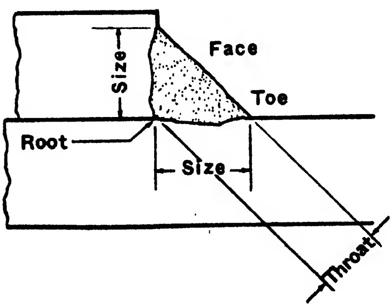


FIG. 3-3.

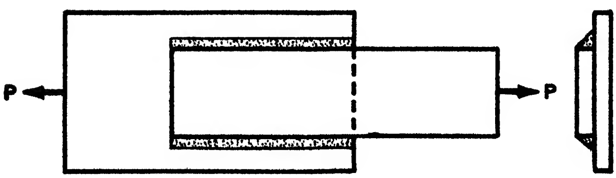


FIG. 3-4.

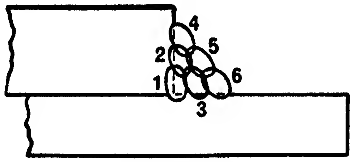


FIG. 3-5.

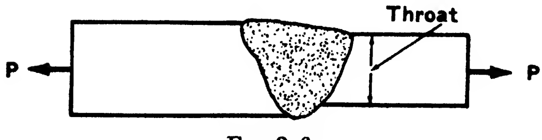


FIG. 3-6.

indicates a large fillet formed by six passes of the electrode. It is necessary to permit each weld pass to cool and to remove the slag before laying the next weld. The diagram indicates the order in which the weld would be made. Larger welds would be formed by continuing the same building-up process.

The butt weld may be used to transmit tension, compression, or shear. Under the *AWS Code for Building Construction*, the allowable unit stress on the throat of the weld is equal to that for the material being welded. A lower value is permitted for conditions of repeated loadings under the *Specifications for Welded Highway and Railway Bridges* in order to prevent fatigue failures. The effective throat of a butt weld is defined as having thickness of the thinner part joined as shown in Fig. 3-6. A butt weld connecting a $\frac{5}{16}$-in. plate to a thicker one would have a tensile value equal to that of the $\frac{5}{16}$-in. plate. For example, if the allowable stress for the plate were 20,000 psi, the value of the weld would be $\frac{5}{16} \times 20{,}000$, or 6,250 lb per lin in. of weld. Similarly, the compressive and shear values of such welds are equal to the compressive and shear values of the thinner of the plates being joined.

A plug weld might be considered as a fillet weld having a length equal to the circumference of the center of the throat. Figure 3-7(*a*) indicates a fillet welded hole and (*b*) shows a plug weld with the cavity completely filled. The value of the fillet weld shown in (*a*) is equal to the throat dimension times the length of the fillet, times the unit shear value of a

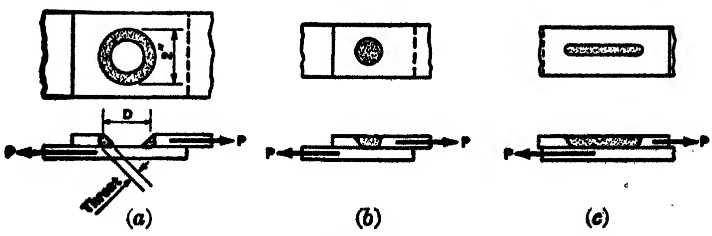
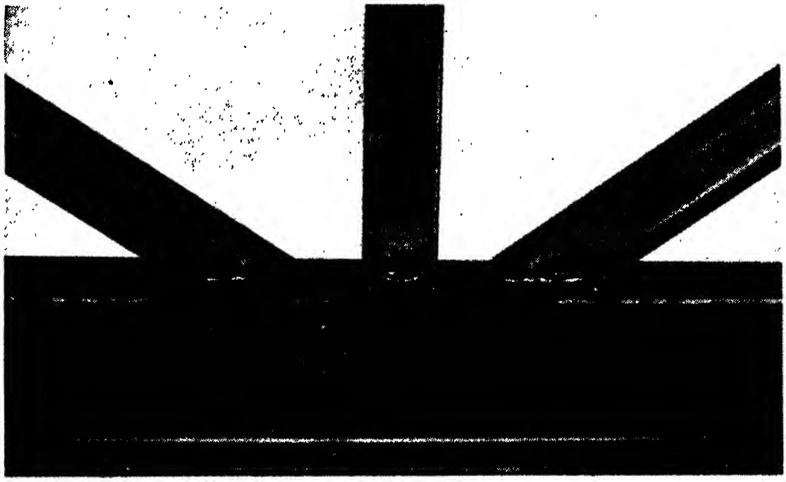


FIG. 3-7.

fillet weld. For a $\frac{5}{16}$-in. weld, the value of the weld in (*a*) would be $0.22 \times 1.844 \times 3.1416 \times 13{,}600 = 17{,}300$ lb. When the hole is completely filled with weld metal, as in the plug weld of (*b*), the value of the weld is the shear value of the circular cross-sectional area. Thus a 1-in. diam plug weld has a value of $0.785 \times 13{,}600$ lb in shear. If the weld is



American Bridge Co.

WELDING DETAIL ON BOTTOM CHORD AT CENTER OF A 60-FT TRUSS.

of the type shown by (*a*), but has so small a diameter that the welds overlap each other, the value is based on whichever area is the smaller, the throat area or the cross-sectional area of the circle.

The slot weld shown in Fig. 3–7(*c*) is an elongated plug weld which has shear value computed in the same manner as for a plug weld.

3–4. Welding Symbols. Figure 3–8 is taken from the American Welding Society publication, *Standard Welding Symbols*, 1947. Instructions for the use of the detailer are reproduced from the same source in Appendix D.

3–5. Inspection of Welds. The value of a weld is dependent to a large degree on the skill of the welder. For this reason it is usual to require that welding be done only by qualified and certified welders. There are commercial welding schools in most large cities that specialize in the training of welders. Certain of the welding apparatus manufacturers conduct welding classes and give qualifying examinations for their certification.

The American Welding Society has prepared a Standard Qualification Procedure covering the details of tests which prospective operators must pass to become certified welders. These standards, or abbreviated forms of the standards, are frequently included in the specifications for work in which welding is to be done. Such specifications usually require that the welder prepare specimens of welding of all types, with all materials, and in all positions in which work is to be done on the contract. The examination of a welder usually requires that he make specimens for visual examination and for simple strength tests. The specimens are visually examined

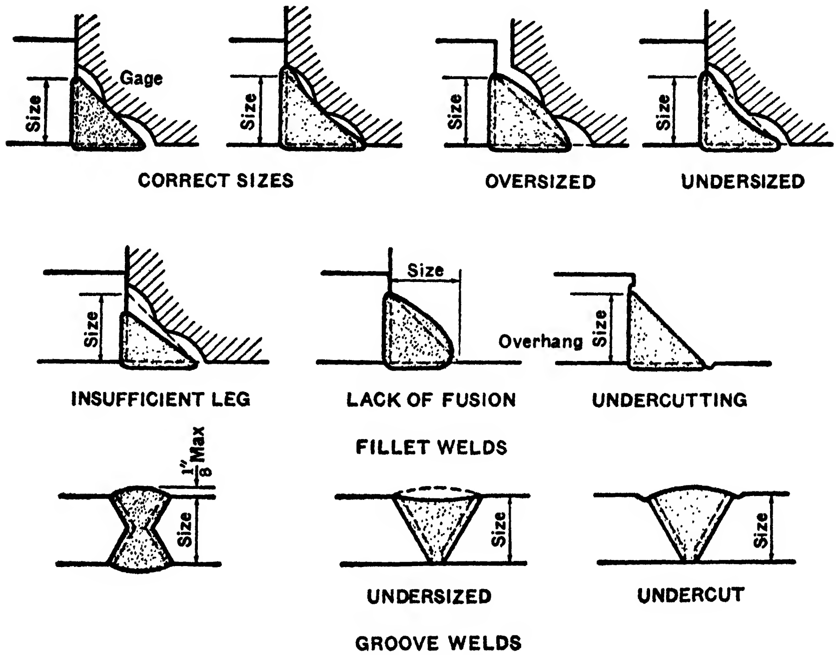


FIG. 3–9. Weld contours and typical defects in weld contour. [Source: H. M. Priest, *The Practical Design of Welded Steel Structures* (New York: American Welding Society, 1943).]

for common defects, such as undercutting, overhang, excessive spatter, and for size and fullness of welds. (See Fig. 3–9 for typical defects.) Simple bending tests indicate whether proper penetration is being obtained; shear and tension tests provide information about the strengths of the welded joints; frequently standard tension tests are made on machined specimens to establish the properties of the weld metal; and occasionally fatigue tests are performed on the weld metal.

Qualification of the welder is an important step toward obtaining satisfactory work, but it does not eliminate the necessity for careful inspection of the work done under the contract. The inspector should be alert to determine whether the parent metal has been properly prepared and is free from oil or dirt, and he should see that all the accepted procedures for welding have been followed. He should check the welds for common defects and sizes; he should watch the formation of the bead, the fusion, the penetration, and the crater, and he should listen to the sound of the arc. Experience in watching these items will indicate quickly to the inspector whether proper procedure is being used.

In most welding operations it is not practical to make strength tests of completed work. On important jobs the X-ray, magnetic particle, gamma ray, or stethoscope examinations are frequently applied. Ultrasonic vibrations may be used also.

3-6. Expansion and Contraction Due to Welding. By the nature of the welding process, a considerable area of the metal being welded is heated to temperatures well above normal. The cooling of the heated portions of the metal results in contraction in proportion to the drop in temperature. When the welding operation is localized, a part of the metal is heated to fusion temperature and the surrounding metal is heated less in amount. Contraction of the heated portion cannot take place without resistance from the surrounding metal subjected to lesser change; hence, residual stresses are set up in the member. Unsymmetrical welds on a member frequently result in deformations of the entire member in a manner similar to the deformations caused by a bending load. Hence, many precautions are necessary to prevent unnecessary deformations of the structure.

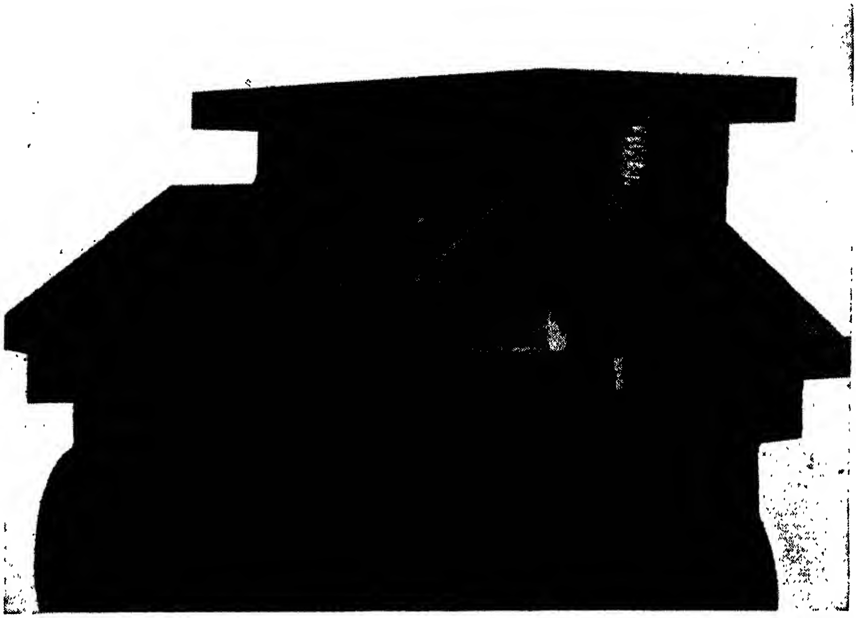
So far as possible, welded designs should provide for symmetrical welding, and the welding operation should be carried out symmetrically. The welding sequence should be such as to provide for the free movement of the parts being welded. It is usually considered advantageous to peen each layer of multiple layer welds after the metal has cooled. Peening is accomplished by light tapping of the weld with a peening hammer. Welds should usually be no larger than necessary, and intermittent welds help to reduce the accumulation of deformation. The sequence of welding operations can be frequently planned to provide compensating effects and to reduce the warping of a member.

The most effective method of relieving the residual stresses set up by welding is to anneal the entire welded member. Unfortunately, the capacities of annealing furnaces do not permit a very large percentage of structural welding to be annealed.

3-7. Weldability of Metals. While the arc and gas welding processes are particularly adapted to use with low-carbon steels, most common metals may be welded by use of properly selected welding rods and procedures. The difficulty in welding steels increases with the carbon content. In general, the welding rod should be selected to provide a deposited metal equivalent to the parent metal.

Metal adjacent to a weld is usually somewhat more brittle than it was before the welding occurred. The extent of the hardening is determined to a large extent by the rate of cooling. The rate of cooling can be reduced by preheating the member before welding. With the higher-carbon content steels, preheating is essential to obtaining satisfactory work.

A great deal of care needs to be used in the welding of cast or gray iron. Localized heating may result in unequal expansion and cracks in the casting. Successful welding of castings can be accomplished by preheating the entire casting to a red heat and welding while hot. If the preheating is impossible,



American Bridge Co.

A Welded Shoe for a Bridge Bearing.

welding may still be possible, but special care should be used in the preparation of the joint and support of the casting during the operation. Some type of mechanical bond between the weld and the casting, such as threaded studs screwed into the casting, is desirable. Unless the entire casting is preheated, care must be taken not to apply too much heat locally during the welding process.

When heat-treated metals are welded, it should be kept in mind that the metal adjacent to the weld has lost the effects of the heat treatment. For this reason, the welding of malleable iron or the aluminum alloys should be done before heat treatment. When heat-treated steels are welded it is necessary to re-heat treat after welding, to maintain the original properties. The welding rods used with alloy and heat-treated steels should, of course, be of such material as to develop properties equivalent to the material being welded. Welding of the stainless steels requires special consideration for each type of steel.

3-8. Flame Cutting. Flame cutting and flame machining are important processes in the fabrication of steel. In general, flame cutting involves the preheating of the material in an oxyacetylene flame, followed by oxidation of the metal on the plane of the cut. Because of the high affinity of oxygen for ferrous metals at melting temperatures, the cutting is largely a chemical process. Oxygen fed to the heated metal through a cutting tip oxidizes the metal, and the pressure of the gas stream forces the melted and oxidized metal out of the cut.

Flame cutting is particularly useful for the preparation of the edges of members that are to be joined by welding. Surprising accuracy can be attained by operators experienced in the use of cutting flames. Thicknesses of metal, which could not be cut practically by other means, are easily cut with the flame; flame cutting has been used to cut steel slabs as thick as 30 in. Considerable precision cutting is being accomplished with machine-guided flames.

The effect of flame cutting is to form a thin layer of more brittle structure adjacent to the cut surface. The amount of this layer and its properties are affected by the rate of cooling of the metal. With thick steel plates or slabs, the amount of heat required for cutting is sufficient to result in slow cooling and some annealing effect. When the cut surface is to be welded later, it is generally considered that little, if any, detrimental effect comes from the flame cutting. Alloy or high-carbon steels must frequently be handled somewhat differently from the low-carbon steels to prevent cracking and warping; considerable success has been attained by preheating the material to 500F or 600F before the cutting operation.

By the manipulation and direction of the cutting flame, bevels, circles, and irregular cuts may be made. Flame machining is accomplished by directing the cutting flame at a small angle to the surface of the work and removing a uniform layer of material of the desired depth and width.

PROBLEMS

3-1. Find the value of a ¾-in. fillet weld 12 in. long, using the AWS Code.

3-2. If 5⁄16-in. fillet welds 3 in. long are spaced 12 in. center-to-center, what is the average value of the weld per linear inch of the member? What is the value of one weld?

3-3. A ½-in. Pl is welded to another by a ½-in. fillet weld placed around the circumference of a 2-in. hole as shown by Fig. 3-7(a). What is the shear value of the connection?

3-4. Find the value of a slot weld 15⁄16 x 4 in. in a 5⁄16-in. Pl. The ends of the slot are semicircles. The slot is filled as shown by Fig. 3-7(c).

3-5. A 4 x ¾ Pl is to be spliced to a 6 x ½ Pl to develop 18,000-psi tension in the plates. How many linear inches of 5⁄16-in. fillet weld will be required?

3-6. If a 4 x ¾ Pl is butt-welded to a 6 x ½ Pl, what will be the value of the connection in tension using the AISC specifications?

CHAPTER 4

SELECTION OF SECTIONS

4-1. Factors Affecting Selection of Section. A most important process in structural design is the determination of the material to be used and of the shape and size of the individual members of the structure. This procedure is preceded by an analysis which provides the amounts of shear, direct load, and bending moment in the members. The selection of section, usually referred to as *design*, is influenced by many factors.

Strength is the factor which usually controls the final selection of section. Members must be strong enough to provide a satisfactory factor of safety against failure from all expected loads—such as dead load, live load, impact, wind, inertia loads, and earthquake loads. Consideration is occasionally given to future loads which might result from extension or conversion of the structure.

The section chosen not only must be strong enough to prevent failure but must also be rigid enough so that excessive deformation (deflection) does not destroy the functional value of the structure or cause failure of attached material (for example, floor slabs or plastered ceilings supported by steel beams).

Weight of the section is always to be considered. Generally, a heavier section costs more than a light one. Weight also adds to the dead load, and so other supporting members may in turn need to be heavier. It is usually a good design practice to choose the lightest and most economical section that fills all the requirements.

In aircraft it is of prime importance that weight be minimized, not so much to reduce the cost as to reduce the dead load and thus increase the carrying capacity of the plane. Design of a light member in aircraft requires consideration not only of the cross section but also of the material to be used. Selection of the lightest section in aircraft design may not always provide the cheapest structure, but the advantages of small increases in the pay-load capacity of the aircraft may easily outweigh the disadvantage of greater cost of the structure.

If the lightest safe section is chosen for each member, the resulting structure may contain a large number of different shapes and sizes of material. Although individually the members may be most economical, collectively they may *not* be most economical. Because of the small amounts of each section required, unit costs may be high. Connection details will be of great variety and may add to fabrication costs. In such cases it is often desirable to limit the number of different sections used, even though some members may then be stronger and heavier than necessary.

The appearance of the structure is often important. There is an increasing tendency to produce structures which are pleasing to the eye. Structures of some types must not only *be* strong enough to do their work, but must also *look* strong enough to give confidence to the user. In aircraft, for example, it is important that visible buckles and wrinkles be limited so that their presence does not disturb passengers or crew. Designers should cooperate to the best of their ability with those responsible for appearance.

Available space is sometimes the determining factor. The section chosen must frequently fill architectural requirement for over-all size and for clearances. The section must be chosen to accommodate any required attachment of detail parts, pipe, conduit, etc.

Service conditions may affect the selection. A structure subject to corrosive influence, erosion, extremes of weather, or unusual temperatures must be designed with consideration of these conditions and of their effect on the useful life of the structure. Specifications for structures which have severe service conditions or which are expected to have a long useful life are often more rigid than those for short-life, light-service structures.

Availability of material must be considered also. The lightest section is probably not the most economical if it is necessary to obtain it from the greatest distance, or if it is necessary for the mill to roll or extrude a small quantity of that section to fill the requirement.

Selection of section is sometimes also affected by ease of handling, fabrication, and erection, or by cost of maintenance.

Tension Members

4-2. Sections for Tension Members. Unit stress in a straight, uniform member having an axial load is given by the equation $f = P/A$. In this expression, P is the applied load and A is the cross-sectional area. The unit stress is the same at all points on the cross section. Design of such a member for tension, when there are no holes, consists only of choosing one with sufficient area so that the unit tensile stress remains below the intensity allowed by the specifications.

A few of the many possible sections for tension members are shown in Fig. 4-1. The simplest include the round bar, flat bar, and single angle shown in parts (*a*), (*b*), and (*c*). In building trusses the double angle, double channel, and I-beam section of parts (*d*), (*e*), and (*f*) are common. The more complex sections, illustrated by parts (*g*) and (*h*), are examples of tension members suitable for bridges. The dotted portions of these members are tie plates or tie bars. They are located at frequent intervals along the length of the member and serve to hold the various elements of the member in the correct position.

4-3. Net Section. Stress in a tension member is increased by the presence of a hole, regardless of whether that hole is filled by a rivet or tight-fitting bolt. The increase is due to reduction of the area over which the load may be distributed and also to concentration of stress at the edge of the hole. Unless the allowable unit stress and the number of repetitions of load application are so high that fatigue failure is probable, it is usual to consider only the effect of reduced area. The formula $f = P/A$ is applicable to members having holes, if, for the value of A, the net area is used. The net area is that area remaining on the cross section after the removal of material by holes.

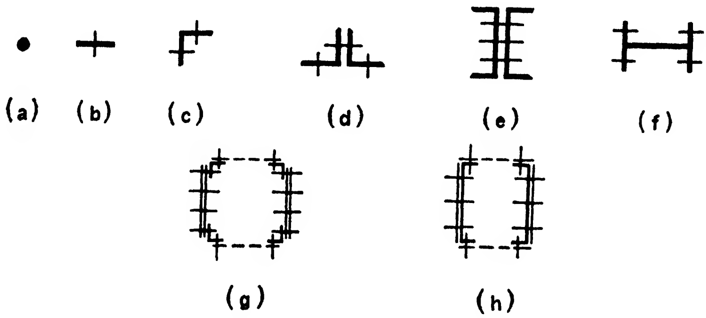


FIG. 4–1.

The area of material removed by a hole is the product of the hole diameter and the thickness of material. However, if the hole is punched, material is often damaged beyond the edge of the hole. For punched holes, the effective hole diameter is usually assumed to be larger than specified. In the case of structural steel, specifications require that holes $\frac{1}{16}$ in. larger than the specified holes be deducted. It should be remembered that specified hole sizes are usually $\frac{1}{16}$ in. larger than the nominal size of rivets to be used. Thus, deduction is made for a hole having a diameter $\frac{1}{8}$ in. larger than the nominal diameter of the rivet.

EXAMPLE 4–1. A structural steel bar 4 x $\frac{1}{2}$ in. in cross section is connected at its ends by $\frac{3}{4}$-in. rivets. There is only one rivet on any cross section. What is the capacity of the bar in tension, using the AISC design specification?

The actual hole size is $\frac{1}{16}$ in. greater than the rivet diameter, or in this case, $\frac{13}{16}$ in. The hole deducted is $\frac{7}{8}$-in. diam.

Thus the net area equals

$$(4 \times \tfrac{1}{2}) - (\tfrac{7}{8} \times \tfrac{1}{2}) = 1.562 \text{ sq in.}$$

Then capacity equals net area times allowable unit stress, or

$$1.562 \times 20{,}000 = 31{,}240\text{-lb tension.}$$

EXAMPLE 4–2. Assume that a plate of 2024-T4 aluminum alloy has an ultimate tensile strength of 62,000 psi. If the thickness of the plate is 0.081 in., and if $\frac{1}{4}$-in. rivets are used in drilled holes of 0.257-in. diam placed 1 in. apart along a line transverse to the direction of the load, what is the ultimate tensile strength of the plate per foot of width?

The net area equals

$$0.081[12 - (12 \times 0.257)] = 0.722 \text{ sq in.},$$

and the ultimate load P is

$$0.722 \times 62{,}000 = 44{,}760 \text{ lb.}$$

Note that the actual hole size is used. This is standard practice in the design of aluminum alloy members with small drilled holes, such as are used in aircraft structures. It should also be noted that the ultimate strength is computed, rather than a safe design load. This is justified by the fact that in aircraft the factor of safety is applied to the value of the load coming to the member, rather than to the value of the unit stresses.

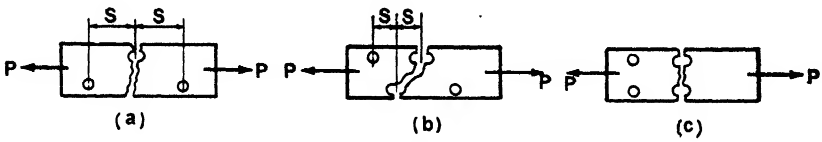


FIG. 4–2.

4–4. Effect of Staggered Holes. In the examples just given, if the members were loaded to ultimate failure, the line of fracture would be transverse to the member and appear as the break shown in Fig. 4–2(*a*).

In the members shown in Fig. 4–2, there are two rows of holes. In (*a*) the holes are staggered, so that failure is on a transverse section. The net area of the member is equal to the gross area minus a deduction for one hole.

In (*c*) the holes are opposite, not staggered. The section of failure is through two holes. The net area is the gross area minus deductions for two holes. Obviously the bar of (*c*) is weaker than that of (*a*).

In (*b*) the holes are staggered, but not so much as in (*a*). This condition is intermediate between (*a*) and (*c*). If the stagger S is not too large, failure will occur along a diagonal path between holes, as shown in (*b*). If we start with a bar as shown in (*a*) and gradually reduce the stagger S until detail (*c*) is reached, the strength changes. But it does not change abruptly from the strength of (*a*) to the strength of (*c*). There is gradual transition in strength. For bars in which failure of type (*b*) occurs, the strength is between that of (*a*) and (*c*).

The exact analysis of stress conditions in (*b*) is very difficult and does not lend itself to engineering design procedures. The strength of (*b*) is known to be greater than that of (*c*). The length of the diagonal path between the holes is greater than the transverse distance between the holes of (*c*). The failure surface of (*b*) is subject to a combination of tensile and shearing stresses. That of (*c*) is subject only to tensile stress. The design methods used for type (*b*) are empirical.

In aircraft work the effect of stagger is often considered to be too uncertain. The full deduction for both holes is required unless the stagger is large enough to ensure failure of type (*a*).

In structural steel specifications the most used method recognizes the greater strength of the diagonal path between holes. In that method, given by the AISC and AREA specifications,* the net area is first computed as if the holes were opposite; then a compensating amount is added to cover the effect of the increased strength of the diagonal path. The resulting total is called the *effective net area.* This method is given by Section 19(*c*) of the AISC specification as follows:

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) in inches of any two successive holes
g = transverse spacing (gage) in inches of the same two holes.

The critical net section of the part is obtained from that chain which gives the least net width.

The application of this method will be illustrated by the next two examples. The method described above is limited to use on members of uniform thickness. If staggered holes occur on sections of variable thickness, a slightly more complex but more general method may be used.

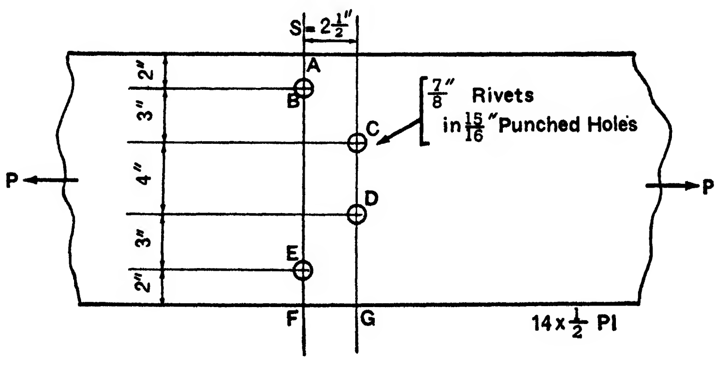


FIG. 4-3.

* See *Proc. AREA*, 1935, p. 775, for history and elucidation of this method of obtaining net sections.

EXAMPLE 4–3. Find the tensile capacity of the plate shown in Fig. 4–3 according to the AISC specification. The effective net widths are computed as follows:

Line A–B–C–D–E–F: $14 - (4 \times 1) + \frac{(2.5)^2}{4 \times 3} + \frac{(2.5)^2}{4 \times 3} = 11.04$, (*Controls*)

Line A–B–C–D–G: $14 - (3 \times 1) + \frac{(2.5)^2}{4 \times 3} = 11.52$,

Line A–B–D–G: $14 - (2 \times 1) + \frac{(2.5)^2}{4 \times 7} = 12.22$,

Line A–B–D–E–F: $14 - (3 \times 1) + \frac{(2.5)^2}{4 \times 7} + \frac{(2.5)^2}{4 \times 3} = 11.74$,

Line A–B–E–F: $14 - (2 \times 1) = 12.00$.

The path which has the least effective net width, and consequently which determines the tensile strength of the member, is path A–B–C–D–E–F. The maximum allowable tensile load P, with the AISC allowable unit stress 20,000 psi, is equal to $20{,}000 \times 11.04 \times 0.5 = 110{,}400$ lb.

EXAMPLE 4–4. The bottom chord of a roof truss consists of 2 ⦣ 4 x 3 x ⅜, as shown in Fig. 4–4(*a*). The total load in the end panel of the bottom chord is 78 kips. Is the chord strong enough to take this load? There are holes for ¾-in. rivets in each leg of the angles. The holes are staggered as shown. Use the AISC specification.

The angle is assumed to be bent flat as shown in Fig. 4–4(*b*). If this were possible, the length along the centerline of the legs would remain unchanged. The material on the inside would be stretched, that on the outside compressed. The angle is thus the equivalent of a flat plate 6⅝ in. wide and ⅜ in. thick. The strength of the angle in tension will be computed, using the plate dimensions.

For failure through one hole only, the net area per angle is

$$A_{\text{net}} = 2.48 - (\tfrac{3}{8} \times \tfrac{7}{8}) = 2.15 \text{ in.}^2$$

For failure through two holes, along the diagonal path, the effective net width is

$$W_{\text{net}} = 6.62 - \left(2 \times \frac{7}{8}\right) + \frac{(2)^2}{4 \times 3.87} = 5.13 \text{ in.},$$

Effective $A_{\text{net}} = 5.13 \times \tfrac{3}{8} = 1.92$ in.² per angle.

The path through two holes gives the smaller effective net area and controls the strength. The allowable load on the member is

$$2 \times 1.92 \times 20 = 76.8 \text{ kips.}$$

This is a little less than the expected load of 78 kips. Whether or not to accept it as satisfactory depends on several conditions. Is the 78-kip figure conservative? Does it include all the probable loadings? Are the loads which were used fairly accurate? If so, the slight overstress that would result is not serious. It is common in building work to allow overstress of from 3 to 5 per cent. The specifications do

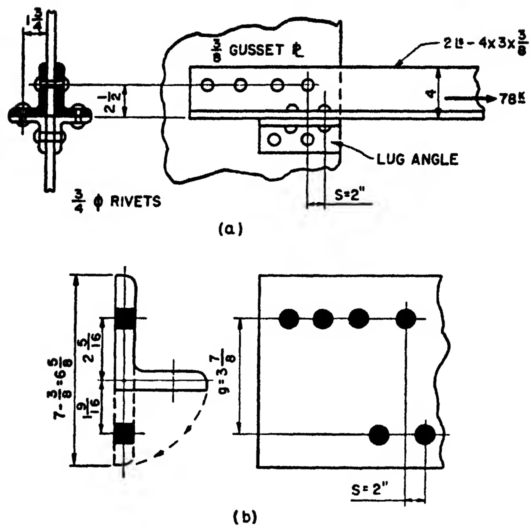


FIG. 4–4.

not suggest this, however. The decision and responsibility is strictly up to the engineer. Most engineers would call the angles analyzed in this example satisfactory.

✓ **4–5. Design of Tension Members.** Because the area to be removed by holes is not always known in advance, the direct choice of section size for tension members is sometimes not possible. In such cases the following procedure is recommended:

1. Using the equation $f = P/A$, determine the net area A required.
2. Estimate the number of rows of rivet holes that will occur at the critical net section.
3. Estimate the thickness of the member and the amount of cross-sectional area removed by these holes.
4. Add the required net area to the estimated area removed by the holes to obtain the approximate total cross-sectional area.
5. Select a section having this value of gross area.
6. For the section selected, compute the net area as illustrated by Exs. 4–1, 4–2, 4–3, and 4–4.
7. If the net area of the trial section is insufficient, or if the section seems uneconomical, revise the selection and resolve as necessary to determine a satisfactory section. Even though the section selected is stressed to the full allowable unit stress, a lighter section may sometimes be found if various thicknesses of material are tried.
8. Check the slenderness or L/r ratio of the member. This ratio is limited by specification requirements, and may control the choice of section. See Art. 4–13.

In the performance of step (2) of the procedure, knowledge of previous similar designs is very useful. For the sections illustrated in Fig. 4–1, probable rivet locations are shown. The single angle of Fig. 4–1(*c*) will likely have two rows of holes as shown. For some applications, however, the designer will assume holes to be deducted from one leg only, provided that he has accurate enough control over the final fabricated product to ensure that condition. For angles of large size, more than one row of holes may occur in each leg.

The flat bar of part (*b*) may have one or more rows of rivets, depending on its width. For ¾-in. or ⅞-in rivets, one row per 3 or 4 in. of bar width would be probable.

The building truss chord sections of parts (*d*), (*e*), and (*f*) generally have rivet patterns as shown. The depth of the channels of Fig. 4–1(*e*) affects the numbers of rows to be expected. Again, rows at 3- or 4-in. centers are probable.

The bridge section of part (*g*) is connected at panel points to two gusset plates, one gusset on the outside of each half of the section. One row of rivets for every 3 or 4 in. of member depth is probable. The rivet holes shown in the horizontal legs of the angles are for the connection of tie plates. The riveting of the section shown in Fig. 4–1 (*h*) is similar.

The rivet patterns suggested may, of course, be varied. A saving of weight may sometimes be made if the rows of rivets are staggered. Occasionally, alternate rows are omitted near the ends of the connections so as to increase the net area of the member at sections where its load is highest. Considerable weight saving may result from the proper choice of rivet pattern. The pattern that has fewer rows of rivets makes possible a saving of weight in the tension member itself. On the other hand, fewer rows of rivets may necessitate longer connections and cause an increase in the weight of gusset plates and other details. There is no formula to give the best arrangement. The best arrangement may be found by comparisons with previous designs and with alternate solutions.

Compression Members

4–6. Long and Short Compression Members. Compression members can be classified according to the manner in which they would fail if subjected to ultimate axial loads. A compression member having a short length relative to its lateral dimensions will usually fail under ultimate axial load by crushing of its material, as shown in Fig. 4–5(*a*). If the material is brittle, failure may include actual fracture in diagonal or longitudinal directions; if it is ductile, a gradual bulging of the member will occur. Failure occurs without bending. Such a short member is often called a *compression block*. A load producing yield-point stress in the material is usually considered as the critical load, inasmuch as this is the limit of the useful value of the compression block. (In a compression test of such compression blocks, however, extremely high unit stresses may be reached as the load is increased beyond that causing the first yielding.)

A column has greater length relative to its least dimension. Failure of a column under axial load involves sudden lateral bending or buckling, as shown in Fig. 4–5(*b*). For a very long column there is a limiting load P, known as the "Euler limiting load," which is critical for the column. When

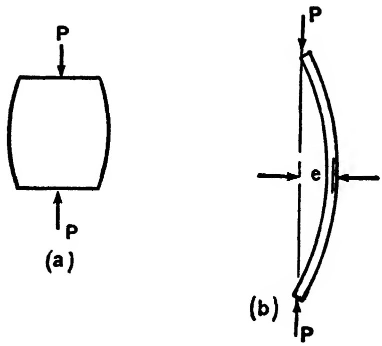


FIG. 4–5.

the critical load has been reached, the column is in a condition of unstable equilibrium and may fail suddenly by buckling, even though the load does not produce an average unit stress as high as the yield point of the material.

Short columns are intermediate between the long columns, whose ultimate load is the Euler limit, and the compression blocks. The true dividing line between the long column and the short column is at the length at which yield-point stress in the material is produced by the Euler load. For constant-section, round-end structural steel columns, the dividing line is at the length for which the ratio L/r is approximately 90.

Figure 4–6 shows a typical curve for the limit of axial stress in a structural steel column. When the unit stress reaches the proportional limit, the value of E decreases and the Euler curve varies from the common form. In the case of a material that has yield-point stress and proportional limit practically equal, the curve of limiting axial stress would be of the type shown for silicon structural steel. The yield point for the aluminum alloys is not well defined; hence, the curve of limiting axial stress for 2024-T4 shown in Fig. 4–6 is approximate.

4–7. Compression Blocks. The dotted line extending upward from the curve for A7 structural steel in Fig. 4–6 shows the upper limits of test results on members behaving as compression blocks.* Since an axial load on a compression block can cause failure because of stressing the member beyond the yield point of its material rather than by buckling, the Euler limiting load is not a consideration, and a factor of safety needs to be

* E. H. Salmon, *Columns* (London: Henry Frowde and Hodder & Stoughton, 1921), Chap. IV, after Von Kármán.

established only with regard to the yield point of the material. The unit stress f produced by the axial load P is equal to P/A at any section. This unit stress should be within the allowable limit permitted by the specifications. However, an allowable stress for compression blocks is not quoted by the usual specification. It is preferred to treat all compression members, other than bearing plates and the like, as "columns."

4-8. Columns. A long column loaded with an axial load has a computed unit stress of $f = P/A$. The limit of axial load on a long column is given by the Euler expression for limiting load, which is $P = \pi^2 EI/L^2$, for a

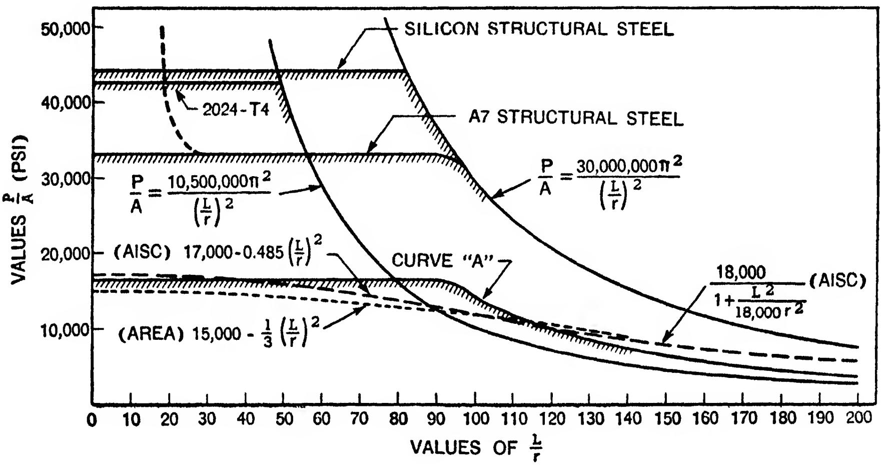


FIG. 4-6.

round-end column of constant section.* Since $I = Ar^2$, the limiting unit stress at the Euler load is

$$\frac{P}{A} = \frac{\pi^2 E}{\left(\frac{L}{r}\right)^2}$$

Figure 4-6 shows the Euler curves for round-end, constant-section columns for values of $E = 30,000,000$ (steel) and $E = 10,500,000$ (aluminum alloys). Horizontal lines are plotted at the values of the yield points of structural steel, silicon steel, and 2024-T4 aluminum alloy. The limit of stress produced by axial load on structural steel columns is considered to be the yield-point stress when L/r is less than 90, and the Euler limiting stress when L/r is greater than 100. A transition curve occurs between these two L/r values. The dividing line between the long column and the

* See C. D. Williams, *Analysis of Statically Indeterminate Structures* (Scranton, Pa.: International Textbook Co., 1951, Chap. 9) for development of Euler limiting loads for columns of variable section.

short column is seen to be at an L/r value of about 50 for aluminum alloys and at an L/r value of about 82 for silicon structural steel.

A theoretical design curve providing a constant factor of safety with respect to the limiting stress in an axially loaded structural steel column could be drawn as a horizontal line between L/r ratios of zero and 90, and as a curve similar to the Euler curve but with ordinates bearing a constant ratio to the ordinates to the Euler curve for L/r values greater than about 90. Such a curve, providing a factor of safety of 2 for round-end, constant-section steel columns, is shown in Fig. 4-6 as *Curve A*. When the yield point is 33,000 psi, the equation of *Curve A* is $P/A = 16{,}500$ for values of L/r between zero and 90 and

$$\frac{P}{A} = \frac{\pi^2 E}{2\left(\frac{L}{r}\right)^2}$$

for values of L/r greater than 100.

The formulas actually used in the designing of columns vary greatly in form. The curves of the formulas, however, are nearly like the theoretical *Curve A*. The 1946 AISC column formulas limit the design axial loads to those which will not produce an average unit stress exceeding

$$\frac{P}{A} = 17{,}000 - 0.485\left(\frac{L}{r}\right)^2$$

for main members with L/r values of up to 120, or

$$\frac{P}{A} = \frac{18{,}000}{1 + \frac{L^2}{18{,}000r^2}}$$

for secondary members with L/r values greater than 120 but not greater than 200. (A secondary member is one which does not carry computed dead load other than its own weight nor live loads other than those from wind or from its action as a brace to other members.)

The AREA column formula for values of L/r between zero and 140 is

$$15{,}000 - \tfrac{1}{3}\left(\frac{L}{r}\right)^2$$

for pinned-end columns. For columns with L/r ratios greater than 140, the AREA requires the use of a secant type of formula which includes bending moment due to various causes. This curve is not shown in Fig. 4-6 because it is not a true axial load formula.

Examination of the AISC and AREA formulas plotted in Fig. 4-6 shows that these curves fall slightly below the developed *Curve A* for most of their lengths between L/r = zero and $L/r = 120$. In other words, these formulas provide factors of safety slightly in excess of 2 within these ranges for round-end, constant-section columns, with the greatest safety for columns having L/r values of about 90.

4-9. Effect of Conditions of End Support. The ultimate axial load on a column is affected by the conditions of end support. A column having its ends pinned (free to rotate) would buckle, as shown in Fig. 4-7(*a*), if loaded

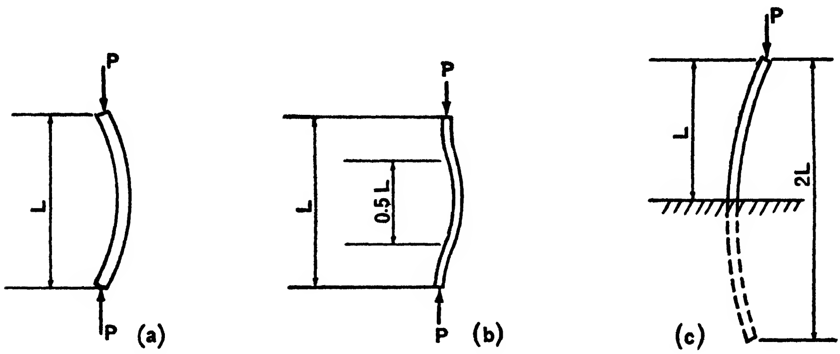


FIG. 4-7.

axially with the critical load. One with ends fixed against rotation would buckle as shown in Fig. 4-7(*b*), and one having one end fixed and the other free to rotate and move laterally would buckle as in Fig. 4-7(*c*).

The Euler formula for the ultimate axial load is developed for the condition of round or *pinned* ends, as shown in Fig. 4-7(*a*). To use the Euler formula for columns with other end conditions would require that a modified or effective length be used to replace the length L. The effective length would be that length of column which functions as if pinned. For example, in Fig. 4-7(*b*) the effective length is only $0.5L$, the distance between points of contraflexure. For the column of Fig. 4-7(*c*) the effective length is twice the column length, since the curve for the deflected column is similar to one-half of the curve for a pinned-end column having a length of $2L$.

In the design formulas for structural steel columns, these refinements of effective length are usually ignored, the column being treated as if pin-connected. The AISC column formula is suitable for use with a pinned-end column, giving a factor of safety slightly in excess of 2 for most L/r ratios. If it is used for a column with some end restraint, the factor of safety will be somewhat higher. The AREA and AASHO specifications recognize the difference in end restraint between pinned-end and riveted-end columns; these specifications allow a slightly higher stress for the riveted-end column.

The exception to the custom of assuming all columns to be pin-connected is made in the case of the column with one free end, as shown in Fig. 4-7(*c*). Regardless of whether the engineer is calculating the ultimate strength by

Euler's formula or by the allowable load, according to the formulas of the various specifications, the effective length used must be twice the actual column length.

In the design of aluminum columns the refinements for conditions of end support are observed. See Section B of Appendix E.

4–10. Types of Column Sections. In Fig. 4–8 a few of the many types of cross section suitable for columns are illustrated. The solid lines of the illustrations indicate elements which extend for the full effective length of the member. The dotted lines represent intermittent lacing or tie plates, the function and design of which are treated later in this chapter.

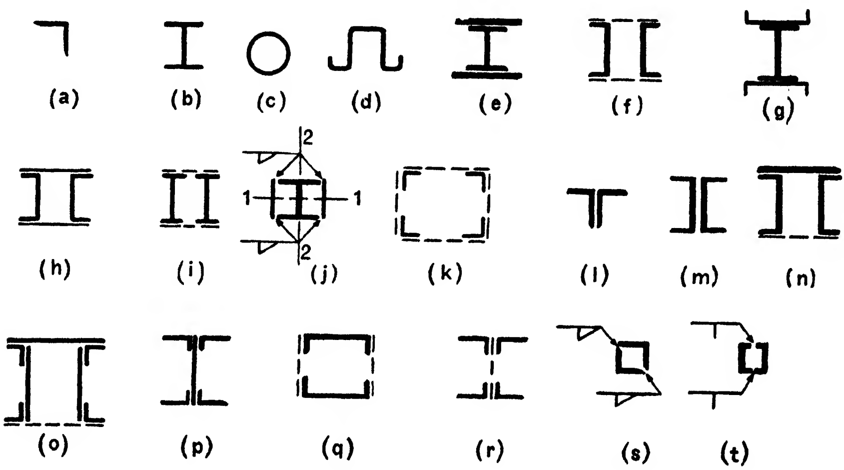


FIG. 4–8.

The single angle of Fig. 4–8(a) is commonly used for relatively light axial loads. The WF section of part (b) is the most usual section employed for building columns and compression members in highway bridges. Columns of the shape shown by part (c) are made of large pipe for building use and of steel or aluminum tubing for aircraft use. Part (d) shows the popular "hat section" of aircraft, which is usually formed by bending a flat sheet to the shape shown.

A column load may be so great that sufficient area cannot be practically or economically provided by a simple section such as illustrated by parts (a) to (d); or the slenderness ratio L/r of such a section may be so great as to reduce the permissible average unit stress P/A to an undesirably low value. In either case a good solution is frequently found to be the use of a section consisting of several simple elements connected together so as to function as a single unit. Parts (e) to (t), inclusive, of Fig. 4–8 illustrate such built-up sections.

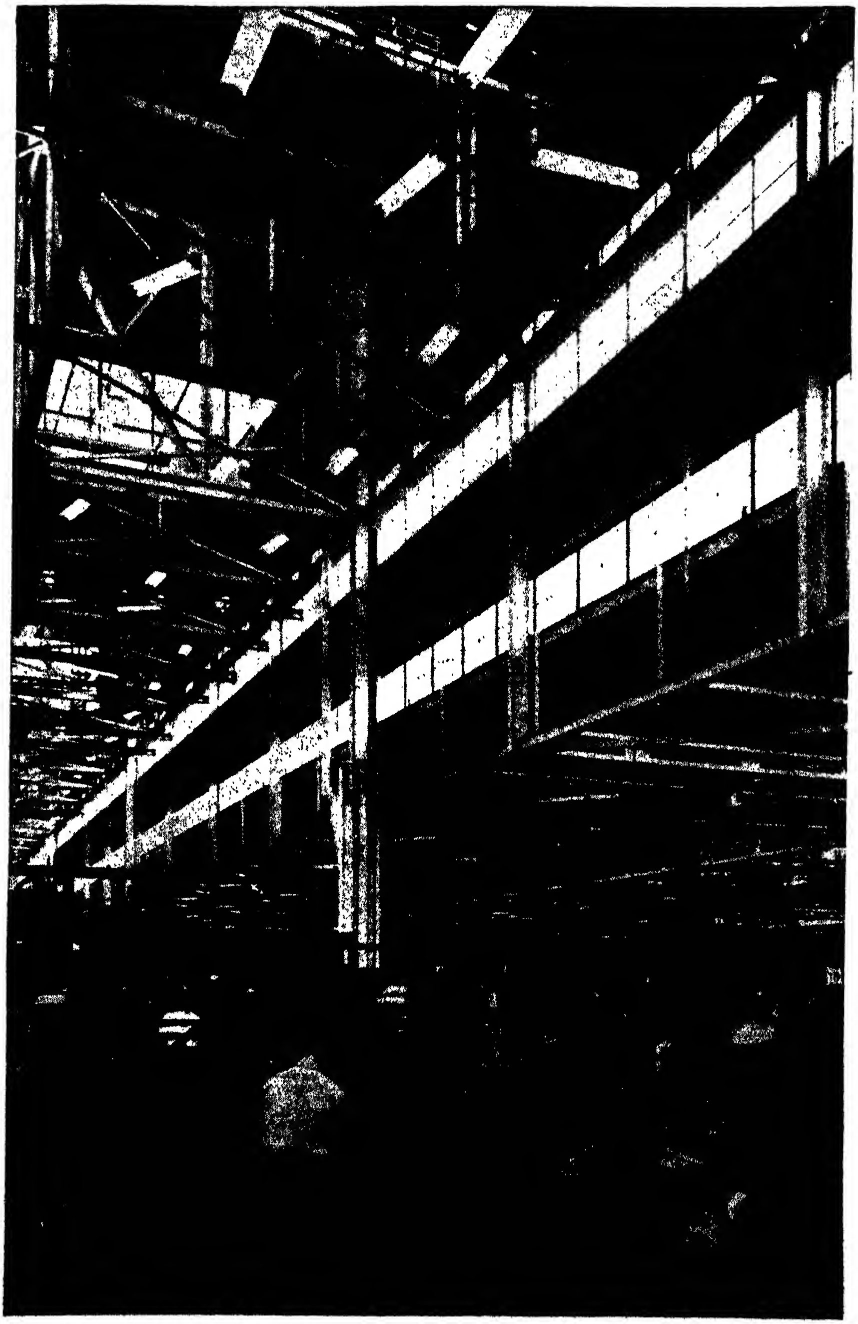
Parts (*e*) to (*j*), inclusive, show built-up sections that are well adapted for use as building columns. Type (*j*) is a welded type in which the added plates serve to equalize the stiffness of the member about both axes by increasing the lower radius of gyration of the core section. The four-angle section with lacing shown in part (*k*) is often observed on crane booms or on small masts or towers. Types (*l*) and (*m*) are commonly used as compression members on light trusses and as struts or compression bracing. Types (*n*) and (*o*) are well adapted for use as compression chords or as end posts of larger trusses, particularly bridge trusses. Types (*p*), (*q*), and (*r*) are used for building columns and for web members of larger trusses. Parts (*s*) and (*t*) illustrate but two of the many possible combinations of elements made practical by welding. Such sections are often valuable for structural parts of cranes or other machinery, as well as for members of buildings.

4–11. Design of Columns. Since the allowable average unit stress P/A for a column is not known until a section has been chosen and its slenderness ratio L/r computed, the selection of a column section is by successive approximation. A recommended procedure is as follows:

1. Assume a logical value of the allowable stress P/A and compute the approximate required cross-sectional area.
2. Select a trial section having this area and compute the maximum slenderness ratio for that section.
3. Determine the allowable average stress P/A and compare it with the actual average stress P/A for the trial section. If the actual does not exceed the allowable, the section is satisfactory.
4. If the section is satisfactory, examine the list of available sections to determine whether or not a more economical satisfactory section can be found. If the actual P/A exceeds the allowable, select another trial section, using the allowable P/A for the first section as a guide in selecting the second.
5. Repeat as often as necessary to determine a section which satisfies the strength and slenderness requirements of the specification and which satisfies the practical requirements of economy, lightness, and over-all size.

As an aid in approximating the allowable P/A [step (1) of the recommended procedure], tables and equations are available in some handbooks which give the approximate radii of gyration for various types of section for which the over-all dimensions are known.

Example 4–5. A building column is required to support a load of 165 kips. The column is 19 ft. in length and cannot be braced laterally except at its ends. Available space limits the size of the column to not more than 9 in. in width in either direction. Using the 1946 AISC specification, choose a suitable rolled section for the column.



Bethlehem Steel Co.

CURTISS-WRIGHT PLANT, BUFFALO, NEW YORK.

The full value of allowable average stress, as given by the specification formula

$$\frac{P}{A} = 17{,}000 - 0.485\left(\frac{L}{r}\right)^2$$

may be used for main compression members in which L/r does not exceed 120. Thus r for the section to be used should be not less than $(12 \times 19)/120 = 1.9$ in. (See also Art. 4–13 and Ex. 4–11.) Since the allowable stress is not known until L/r for the chosen section is known, an assumption must be made. Assuming that the allowable P/A is 11,000 psi, the required area is 15 sq in.

For the first trial, assume the use of an 8 WF 58. For this section, $A = 17.06$ sq in. and the least $r = 2.10$ in. The greatest L/r equals

$$\frac{12 \times 19}{2.10} = 108.5;$$

and the allowable P/A equals

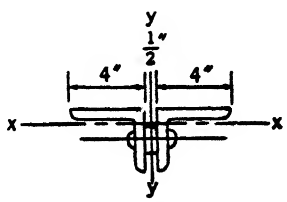
$$17{,}000 - 0.485(108.5)^2 = 11{,}290 \text{ psi.}$$

The actual P/A equals

$$\frac{165{,}000}{17.06} = 9{,}670 \text{ psi.}$$

The actual P/A does not exceed the allowable; therefore, the section will satisfy the specification. A further check will show that the next lighter section of 8-in. nominal depth will not be large enough. The 8 WF 58 is the most economical rolled section which will fill the requirements. If it were possible in this case to use sections of greater than 9-in. depth, greater economy would result; a 10 WF 49 or a 12 WF 53 would satisfy the specification.

EXAMPLE 4–6. The top chord of a truss consists of two 4 x 3½ x ½ angles placed as shown in Fig. 4–9. The chord is braced in a vertical plane by adjoining web



Stitch rivets and fills at 24-in. centers.

FIG. 4–9.

members which enter at 5-ft intervals. Horizontally, the chord is braced at 10-ft intervals by a system of bracing rods. What is the allowable compressive load, according to the 1946 AISC specification? Assume the two angles to be riveted to each other at close enough intervals so that buckling of an individual angle cannot occur at a lower load than the load required to cause failure of the entire section. The properties of the combined section are computed, giving

$$r_{x-x} = 1.04;\ r_{y-y} = 1.94;\ A = 7.00 \text{ in.}^2,$$

$$\frac{L}{r} = \frac{12 \times 5}{1.04} = 57.7, \text{ about axis } x\text{–}x,$$

$$\frac{L}{r} = \frac{12 \times 10}{1.94} = 61.9, \text{ about axis } y\text{–}y.$$

The strength is determined by the larger value of L/r; hence, the allowable P/A equals

$$17{,}000 - 0.485(61.9)^2 = 15{,}140 \text{ psi.}$$

The allowable load P is

$$7.0 \times 15{,}140 = 106{,}000 \text{ lb.}$$

4–12. Local Buckling. If the metal in a compression member is sufficiently thin, failure may result from local buckling at a load which is not great enough to cause the member to buckle as a whole. The local buckling occurs as a wrinkle or series of wrinkles which are affected by the properties of the cross section but not by the length of the member. Although local buckling usually occurs at the edge of a thin flange or cover plate, it may also occur at interior sections, such as a thin web. Consideration of local buckling is extremely important in aircraft structures, since compression members for aircraft are frequently formed of bent flat sheets, and in other structures having stiffened flat sheets functioning as compression members.

There are two methods in common use in the design of members susceptible to local buckling. The methods recognize that the factors controlling the occurrence of local buckling include the width, thickness, and condition of the edge support for the portion in question.

The first method is common to most structural steel specifications. It consists of preventing local buckling from occurring at loads less than the design limit of the column as a whole. This is accomplished by limiting the ratio of the width of the outstanding portion to the thickness of metal and by limiting the distance between connecting rivets in built-up sections. An example of this method is shown by the following excerpt from Section 18 of the AISC specification:

(b) Projecting Elements Under Compression.

Projecting elements of members subjected to axial compression or compression due to bending shall have ratios of width to thickness not greater than the following:

Single angle struts; 12.

Double-angle struts; angles or plates projecting from girders, columns or other compression members; compression flanges of beams; stiffeners on plate girders; flanges or stems of tees; 16.

The width of plates shall be taken from the free edge to the first row of rivets or welds; the width of legs of angles, channels and zees, and of the stems of tees, shall be taken as the full nominal dimension; the width of flanges of beams and tees shall be taken as one-half the full nominal width. The thickness of a sloping flange shall be measured halfway between a free edge and the corresponding face of the web.

When a projecting element exceeds the width-to-thickness ratio prescribed in the preceding paragraph, but would conform to same and would satisfy the stress requirements with a portion of its width considered as removed, the member will be considered acceptable without the actual removal of the excess width.

(c) **Compression Members.**

In compression members the unsupported width of web, cover or diaphragm plates between the nearest lines of rivets or welds, or between the roots of the flanges in case of rolled sections, shall not exceed 40 times the thickness.

When the unsupported width exceeds this limit, but a portion of its width no greater than 40 times the thickness would satisfy the stress requirements, the member will be considered acceptable.

The unsupported width of cover plates perforated with a succession of access holes, only the least net width across holes being assumed available to resist compression, may exceed 40, but shall not exceed 50, times the thickness.

In the second method the stress causing local buckling (or this value divided by a factor of safety) is actually computed. The critical local buckling stress is then compared with the yield strength and with the critical stress for buckling of the column as a whole (or with these values divided by the factor of safety). Whichever is the lowest of these three critical stresses controls the capacity of the column. This method, with tables and equations relating the critical local buckling stresses (ultimate) to the type of alloy and to the dimensions and type of edge support of the element, is shown in detail in the *Alcoa Structural Handbook*. Section D of the aluminum specifications (Appendix E) shows the method for use with aluminum alloy 2014-T6. The factor of safety has been established by the writers of this specification, so that allowable stresses, rather than ultimates, are computed and compared.

If a flat element which buckles locally is connected at one or both edges to other material that has not buckled, the stress on the buckled portion will remain as high as it was before buckling, but it will not increase beyond this value. The stress on the adjacent unbuckled portions may be increased, however. Thus, at the ultimate load for the entire member, the stress on the cross section may be nonuniform. Both methods mentioned above make provisions for considering this action. The AISC specification provides for the complete ignoring of portions which might buckle locally. Its application will be shown by Exs. 4–7 and 4–8. The aluminum specifications provide rules for computing an *effective area* for members in which parts may buckle locally. This effective area is a compromise area, less than the gross but larger than the area of the unbuckled portions alone, on which the total column load is assumed to be uniformly distributed.

Example 4–7. A new bin is to be supported by four columns, as shown by Fig. 4–10(*a*). A supply of 6 x 6 x 3/8 angles is on hand and available for this use. Would

they be satisfactory according to the AISC specification? The axial load per column will be 16 kips.

Figure 4–10(b) shows the cross section of one post. The ratio of outstanding leg width to thickness is $6/0.375 = 16$. The maximum ratio allowed for single-angle struts is 12. Therefore, the 6 x 6 x ⅜ angle cannot be considered fully effective. The effective size is limited to $12t$ for each leg, or 4½ x 4½ x ⅜ (shown shaded). The properties of a 4½ x 4½ x ⅜ angle will be used. The radius of gyration r is obtained by interpolation from the tables of properties as 0.89 in.

$$\frac{L}{r} = \frac{13.5 \times 12}{0.89} = 182$$

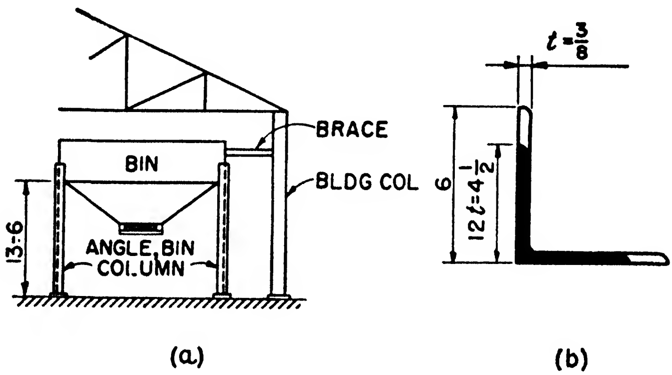


FIG. 4–10.

This is in excess of the 120 limit for main members. Therefore, the additional reduction of Section 16(b) of the specification must be applied.

$$\text{Allowable } \frac{P}{A} = \frac{18{,}000}{1 + \dfrac{(182)^2}{18{,}000}} = 6{,}340 \text{ psi, for secondary members}$$

$$1.6 - \frac{L}{200r} = 0.69$$

$$\text{Allowable } \frac{P}{A} = 6{,}340 \times 0.69 = 4{,}370 \text{ psi, for main members.}$$

The effective area of a 4½ x 4½ x ⅜ angle is

$$\tfrac{3}{8}(9 - \tfrac{3}{8}) = 3.24 \text{ sq in.}$$

$$\text{Allowable } P = 3.24 \times 4.37 = 14.2 \text{ kips}$$

The 6 x 6 x ⅜ angle is not satisfactory. If bracing could be provided so as to reduce the unbraced length L, it might be made satisfactory.

EXAMPLE 4–8. Would the angles of Ex. 4–7 be satisfactory if braced at the midpoint in one plane only? This bracing is shown in Fig. 4–11(a).

A section of one column is shown in Fig. 4–11(b). The midpoint of the length is braced against any movement having a component in the Y direction. The midpoint can move only in the X direction. Buckling of the entire 13 ft 6 in. length would occur about axis Y-Y only. The length above or the length below the braced point could buckle in any direction. Thus two L/r values must be computed and compared. The interpolated value of r for axis Y-Y of a 4½ x 4½ x ⅜ effective angle is 1.39 in.

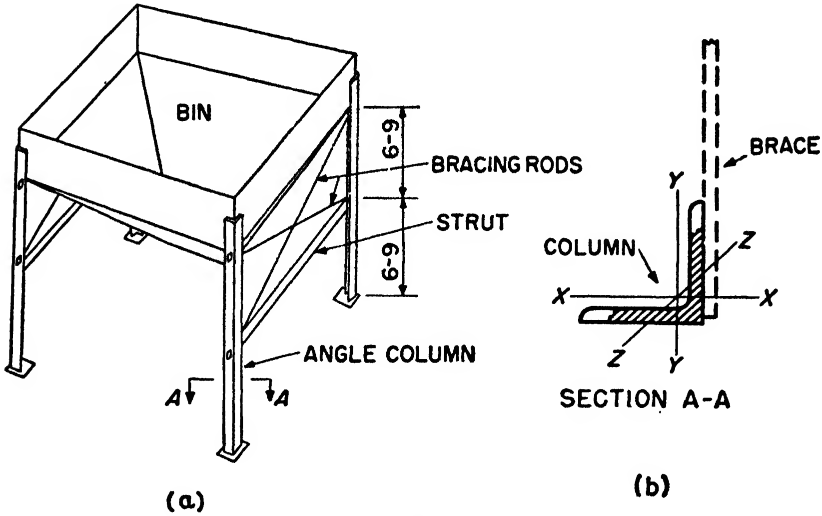


FIG. 4–11.

For axis Y–Y: $\dfrac{L}{r} = \dfrac{13.5 \times 12}{1.39} = 116.5$ *Controls*

For axis Z–Z: $\dfrac{L}{r} = \dfrac{6.75 \times 12}{0.89} = 91$

$$\text{Allowable } \frac{P}{A} = 17{,}000 - 0.485(116.5)^2 = 10{,}420 \text{ psi}$$

The effective area is still limited to 3.24 sq in., that of a 4½ x 4½ x ⅜ angle.

$$\text{Allowable } P = 3.24 \times 10.42 = 33.8 \text{ kips}$$

If so braced, the 6 x 6 x ⅜ angles would be satisfactory, but they would not be the most economical in weight of material. The fact that they are on hand, however, could make their use the most economical solution with regard to the total cost.

EXAMPLE 4–9. A 5 x 5 x ⅜ angle of aluminum alloy 2014-T6 is 5 ft in length and has each end riveted to two gusset plates. (Such a connection still does not have a fixed-end condition, unless the gusset plates themselves are rigidly held so as to prevent their movement.) Compute the allowable axial load, using the aluminum specifications.

For this section the area is 3.60 sq in. and the minimum r is 0.98 in. (axis 3–3).

For over-all buckling of the column, assume k to be 0.75 (Section B, aluminum specifications).

$$\frac{L}{r} = \frac{60}{0.98} = 61$$

The allowable stress is given by the curve of Fig. 2 of the aluminum specification as 17,300 psi. For local buckling, Section D and Fig. 4 of the specification are used.

$$b/t = 4.62/0.375 = 12.3$$

$$L/b = 60/4.62 = 13.0$$

$$\text{Allowable } P/A = 10{,}500 \text{ psi}$$

The lower allowable controls, and P allowable $= 3.60 \times 10{,}500 = 37{,}800$ lb.

EXAMPLE 4-10. What is the ultimate capacity of the single angle of Ex. 4-9, according to the data of the *Alcoa Structural Handbook?*

The ultimate strength of 2014-T6 rolled shapes of this thickness is given by the equation

$$f_a = 64{,}700 - 543KL/r, \text{ for } KL/r \text{ up to } 54,$$

or by

$$f_c = \frac{102{,}000{,}000}{\left(\frac{KL}{r}\right)^2}, \text{ for } KL/r \text{ over } 54.$$

The effective KL/r for over-all buckling is

$$0.75 \times 60/0.98 = 46.$$

For local buckling, the effective KL/r is found by using tables in the Alcoa publication.

$$L/b = 13$$

$$k' = 5.13$$

$$\frac{KL}{r} = 5.13b/t = 63$$

The effective slenderness ratio is larger for local buckling than for over-all buckling. Local buckling therefore controls.

$$f_c = \frac{102{,}000{,}000}{(63)^2} = 25{,}700 \text{ psi}$$

The probable ultimate load is

$$P = 3.60 \times 25{,}700 = 92{,}500 \text{ lb.}$$

4-13. Limiting Length of Tension or Compression Members. Most steel structure specifications require that the slenderness ratio L/r of a tension member or a compression member be limited to not more than a stated maximum allowable value, regardless of the magnitude of the expected load. The purpose of an upper limit of slenderness is to ensure reasonable stiffness of the structure and to reduce undesirable deflection and excessive vibration under moving loads. This requirement also serves to increase safety by prohibiting the use of extremely long columns for which slight imperfections of manufacture or unpredicted small transverse loads greatly reduce the ultimate load.

As an example of specification requirements governing slenderness, consider the 1946 AISC specification, which limits the slenderness ratio as follows:

(a) The ratio of unbraced length to least radius of gyration L/r for compression members and for tension members other than rods shall not exceed:

For main compression members	120
For bracing and other secondary members in compression	200
For main tension members	240
For bracing and other secondary members in tension	300

(b) The slenderness of a main compression member may exceed 120, but not 200, provided that it is not ordinarily subject to shock or vibratory loads and provided that its unit stress under full design loading shall not exceed the following fraction of that stipulated under Section 15(a)(2) for its actual ratio L/r:

$$1.6 - \frac{L}{200r}.$$

AISC specifications previous to 1946 did not contain the provision of part (b) of the preceding quotation. The addition of that provision makes possible great savings in the quantity of steel needed for lightly loaded building columns. Its use will be illustrated by Ex. 4–11.

Example 4–11. Select a section for a building column 18 ft long, subject to an axial compressive load of 25 kips. Lateral support is provided at the ends only. Use the AISC specification.

The use of AISC specifications previous to the 1946 issue would result in the selection of an 8 WF 31 for this member. The selection would be controlled not by stress considerations but by the requirement that the slenderness ratio L/r not exceed 120 for main members. The 8 WF 31 is the lightest section for which L/r is not over 120, but its axial load capacity is 104 kips, or over four times the load to be applied.

It will now be shown that, by application of the 1946 AISC specification provision for main compression members with L/r of over 120, much saving is possible. The following analysis is for a 5 WF 18.5 section:

$$A = 5.45 \text{ in.}^2; \quad r_{\min} = 1.28 \text{ in.}; \quad \frac{L}{r} = \frac{12 \times 18}{1.28} = 169.$$

The allowable P/A for secondary members with $L/r = 169$ is

$$\frac{18{,}000}{1 + \dfrac{1}{18{,}000}(169)^2} = 6{,}960 \text{ psi}.$$

For a main member having L/r of 169, the stress P/A allowed is the product of 6,960 and the quantity $1.6 - 169/200 = 0.755$.

The allowable P/A for the column is $0.755 \times 6{,}960$, or 5,250 psi.

The actual P/A for the section is 25,000/5.45, or 4,590 psi. The allowable exceeds the actual; therefore, the section is satisfactory. A saving of 225 lb of steel per column is made possible by use of the provision quoted.

The *AASHO Standard Specifications for Highway Bridges* contain an instruction on computing the slenderness ratio. Often the section for a compression member, selected to meet the maximum L/r requirement, has greater weight than would a section having greater slenderness but sufficient strength. The following quotation from the specification (Section 3.6.9 for the limiting length of members) and an example of its application illustrate how some material saving may be made in a long compression member with small load. The specification recognizes that stiffness of a member is measured by its I value and not by its r value.

For compression members, the ratio of unsupported length to radius of gyration shall not exceed 120 for main members, or those in which the major stresses result from dead load or live loads, or both; and shall not exceed 140 for secondary members, or those whose primary purpose is to brace the structure against lateral or longitudinal forces, or to brace or reduce the unsupported length of other members, main or secondary.

In determining the radius of gyration, the area of any portion of a member may be neglected, provided that the area thus neglected shall not be used in calculating the strength of the member. For members resisting calculated compressive stress, all of the specified requirements for minimum thickness of component parts to resist buckling shall apply to material comprising any area thus neglected.

The application of this provision of the AASHO specification will be illustrated by Ex. 4–12.

EXAMPLE 4–12. Assume that a compression member 28 ft long is to be designed to carry a compressive load of 150,000 lb, primarily due to wind or longitudinal loadings on the bridge. Design a member in accordance with the AASHO specification.

A section consisting of four angles, 5 x 3½ x ⅜, and one 10 x ⅜ Pl, arranged as shown in Fig. 4–12, will be tried.

The properties of the complete section are first computed:

	A	I_{2-2}
$4 - 5 \times 3\frac{1}{2} \times \frac{3}{8}$ Ls	12.20	70.7
$1 - 10 \times \frac{3}{8}$ Pl	3.75	0.0
	15.95	70.7

Then

$$r_{2-2} = 2.10$$

and

$$\frac{L}{r} = \frac{28 \times 12}{2.10} = 160,$$

which exceeds the maximum permitted by the specification.

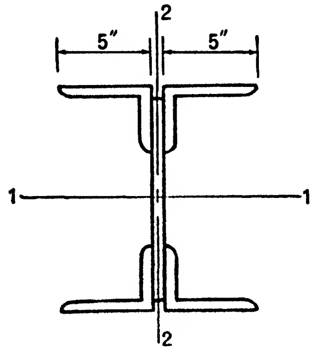


FIG. 4–12.

The properties are recomputed, neglecting the effect of the plate:

	A	I_{2-2}
$4 - 5 \times 3\frac{1}{2} \times \frac{3}{8}$ Ls	12.20	70.7

$$r_{2-2} = 2.40$$

and

$$\frac{L}{r} = \frac{28 \times 12}{2.40} = 140,$$

which is satisfactory.

The computed unit stress, neglecting the plate area, is

$$\frac{P}{A} = \frac{150{,}000}{12.20} = 12{,}300 \text{ psi}.$$

The allowable stress is then computed:

$$\frac{P}{A} = 15{,}000 - \frac{1}{4}\left(\frac{L}{r}\right)^2 = 15{,}000 - \frac{140 \times 140}{4} = 10{,}100$$

+ 25% (since loading includes wind; see Section 4, Article 3.4.1 of AASHO spec.) = 2,530

Allowable stress = 12,630 psi

Since the allowable stress exceeds the actual, the section is satisfactory. The plate may be used in the member and frequently will be desirable to reduce the work required in fabrication of a laced member. The member shown is also less susceptible to corrosion than a laced member.

The AISC and the AREA specifications do not provide for computation of the values of one part only of any compression member. In the application of these specifications, the entire section must be included in the calculation of both A and r.

Beams

4–14. Design. A beam is subject principally to transverse loads. The effect of these loads is to cause shear and bending moment in the member. Choosing a section for a beam consists of finding (sometimes by direct methods, but more often by trial) a shape and size of member in which neither the bending (flexural) stresses nor the shear stresses exceed those permitted by the specifications used.

4-15. Flexural Stresses. The amount of stress caused by a bending moment is found from the expression $f = My/I$, in which

f = unit flexural stress (psi),
M = bending moment at section in question (in.-lb),
y = distance from center of gravity axis of the cross section to the point for which f is desired (in.), and
I = moment of interia (in.^{4}) of the cross-sectional area about the center of gravity axis of the section (axis perpendicular to the plane of loads).

The development of this formula may be found in most texts on strength of materials. It is based, however, on certain assumptions and limitations, which it is well to review. The assumptions are as follows:

1. A section which is plane before bending remains plane after bending.
2. The material of the beam is not stressed beyond its elastic limit; therefore, stress is proportional to strain.
3. The transverse cross section of the beam is symmetrical about an axis in the plane of the loads.
4. The beam is homogeneous and straight.

In a beam subject to bending and shear only, the flexural stress at the center of gravity is zero, since $y = 0$ in the flexure formula. A line drawn either transversely or longitudinally through the points of zero flexural stress is called the *neutral axis.* (See Fig. 4-13.) For a beam having all its transverse loads in the same plane of symmetry and having no axial loads, the neutral axis and the center of gravity axis of the cross section coincide. The stress on either side of the neutral axis varies from zero at the neutral axis to maximum intensity at the extreme (outermost) fiber.

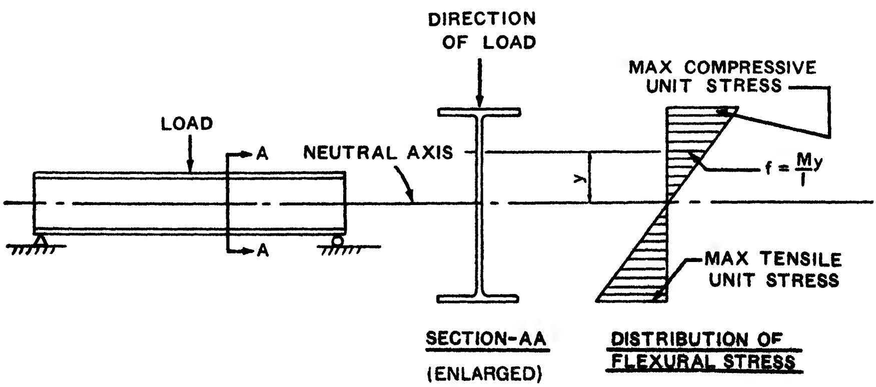


Fig. 4-13.

On one side of the neutral axis the stress is tensile; on the other side, compressive. For beams with axial load as well as moment, the neutral axis and the center of gravity do not coincide (see Art. 4–31).

If the beam is so supported that deflection is possible in the plane of the loads but is prevented in a transverse direction, the formula $f = My/I$ is sufficient to determine a section to resist bending. Tables of standard sections usually contain values for I/c, in which c is the y distance from the neutral axis to the outermost or extreme fiber. The term I/c is called the *section modulus.* The flexure formula then becomes $f = Mc/I = M/S$. The value of S required to resist the expected bending moment equals $M/(allowable\ stress)$. The value of S for the section chosen must be at least as large as that obtained by the last equation; if it is less, the actual stress will exceed the allowable. (Note that, if the section is not symmetrical about the neutral axis, a section modulus may exist for the top fibers different than for the bottom fibers of the section.)

Example 4–13. Let it be required to choose a wide-flange steel section for a beam, loaded as shown in Fig. 4–14, in which transverse deflection is prevented. The design is to be in accordance with the allowable unit stresses of the AISC specification. The weight of the section to be used is not known in advance, and so it is necessary to assume a value which will be confirmed later or corrected. Assume the weight of the section to be 30 lb per ft. The total load w = 830 lb per ft; $M = wL^2/8$ = 41,500 ft-lb, or 498,000 in.-lb. The allowable bending stress is 20,000 psi (either tension or compression). The required S = 498,000/20,000 = 24.9 in.^{3} From tables of WF (wide flange) sections in the AISC handbook, any of the following will provide the necessary section modulus: 8 WF 31 (S = 27.4); 10 WF 25 (S = 26.4); 12 B 22 (S = 25.3).

Of these sections, the 12-in. section is the lightest and is the logical choice of section, provided requirements other than flexural strength do not indicate a different size.

In solving for bending moment, an assumed value of 30 lb per ft was used for the weight of the beam. If the weight of the section selected were materially different

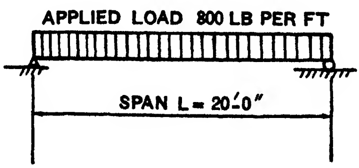


Fig. 4–14.

from the value assumed, it would be advisable to recompute the moment and shear, using the weight of the first-selected beam, and then to select a new section.

Ordinarily, the assumed weight of the beam may be considered accurate enough if its correction will not change the total bending moment sufficiently to result in the selection of a different section. Specifications frequently limit the error which may be permitted in estimated dead-load values.

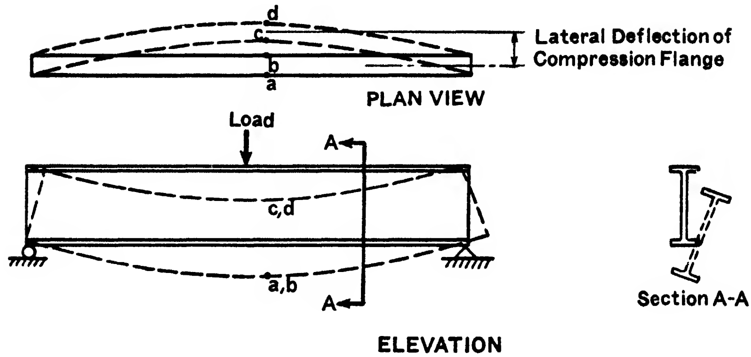


FIG. 4-15.

4-16. Lateral Stability of Beams. Thus far, only beams laterally restrained to prevent deflection perpendicular to the plane of the load have been considered. If the lateral restraint or lateral support were removed, the ultimate strength of the beam might be reduced.

When an axial compressive load is gradually applied to a long slender bar, that bar remains straight for a time; but when the load becomes great enough, it suddenly deflects sidewise. When the buckling occurs, the bar has reached its ultimate load, corresponding to the Euler limit for a column. In this condition it has no further useful value.

If the compression flange of a beam is sufficiently narrow, it may buckle in somewhat the same manner. Figure 4-15 illustrates both deflection in the plane of the load and lateral deflection, or buckling. Lateral buckling occurs in the compression flange only; the tensile stresses of the opposite flange tend to cause it to remain in a straight line. Consequently, lateral buckling is a combined action like that of a column and that of torsion or twisting of the beam.

The compression flange of a beam may be considered as a column loaded along its length by increments of load transferred to it by horizontal shear from the web. While the column discussed in Art. 4-8 has its loads applied entirely at the column ends, the compression flange of the beam has zero load at points of zero bending moment. The load on the flange increases as the point of maximum moment in the beam is approached. The type of loading received by the beam affects the type of load received by the flange. For a beam loaded with a single concentrated load at the center of the span, the shear is constant from the end to the center of the beam. For this condition of loading, a beam with constant section will have uniform increments of load transferred from the web to the flange for each inch of its length. The amount of this increment will be VQ/I, in which V is the shear in the beam, Q is the statical moment of the flange about the neutral axis, and I is the moment of inertia of the beam about its neutral axis.

It should be obvious that the critical axial load for a column loaded in the manner of a beam flange will be greater than for a column of equal dimensions loaded entirely at its ends. Except for local buckling, the failure of the flange as a column could occur only in a direction perpendicular to the plane of the web.

The factors which affect the critical compressive stress (that required to cause lateral buckling) include the shape of the shear diagram (type of loading on the beam flange), conditions of end restraint of the compression flange, position and type of intermediate supports, the support which the web offers against lateral buckling, the properties of the flange itself, and the modulus of elasticity of the material.* A set of equations considering all these factors and giving the allowable compressive flexural stress in beams of 27S-T aluminum alloy has been published.† These equations are quite complicated, each being limited to one material, one condition of end restraint, and one type of loading.

A slightly simpler equation for the allowable compressive stress, considering all factors listed above except the shape of the moment diagram, is given in curve form by the aluminum specifications (see Appendix E). Because elastic deformations of aluminum alloy beams are relatively large, it is recommended by the specification that a rigorous analysis considering these factors be used. Such equations are recommended for bridges and other structures of aluminum alloys, even though the equations are more complex than those specified for use with structural steel.

EXAMPLE 4–14. To illustrate the use of the equation of the aluminum specifications, let it be required to compute the maximum permissible concentrated load at the center of a laterally unsupported simple beam of aluminum alloy 2014-T6. The span is 10 ft. The beam is of I-shape, is 8 in. deep, and weighs 6.35 lb per ft. The properties of the section are given by the *Alcoa Structural Handbook* as:

$S_c = 14.39$ in.^{3};
$I_y = I_1 = 3.73$ in.^{4};
J (torsion factor) $= 0.34$ in.^{4}.

(For information and explanation of the torsion factor the student is referred to texts on strength of materials.)

To determine the allowable compressive stress from the curve, the quantity B must be first computed. (See Section C of the aluminum specifications.)

$$B = I_1 d\sqrt{11.7 + \frac{J}{I_1}\left(\frac{L}{d}\right)^2}$$

Substituting for I_1, d, J, and L gives $B = 169$.

* Clifford D. Williams, *The Limit of Stress in the Compression Flanges of Beams* (Preliminary Publication, Fourth Congress, International Association for Bridge and Structural Engineering, 1952).

† Leon S. Moisseiff, *Design Specifications for Bridges and Structures of Aluminum Alloy 27S-T* (Pittsburgh: Aluminum Company of America, 1940).

$$\frac{L}{\sqrt{B/S_c}} = \frac{120}{\sqrt{169/14.39}} = 35$$

Entering the specification curve at an abscissa of 35 gives

$$F_b \text{ (allowable)} = 8{,}800 \text{ psi.}$$

The allowable bending moment is

$$S_a F_b = 14.39 \times 8{,}800 = 126{,}500 \text{ in.-lb,}$$

or

$$M = \frac{126{,}500}{12{,}000} = 10.54 \text{ ft-k.}$$

$$\text{Dead load moment} = \frac{0.00635 \times 10^2}{8} = 0.08 \text{ ft-k}$$

$$\text{Allowable live-load bending moment} = 10.46 \text{ ft-k}$$

$$\text{Allowable concentrated load} = \frac{4M}{L} = \frac{4 \times 10.46}{10} = 4.18 \text{ kips}$$

The allowable concentrated load, as limited by lateral buckling, is 4.18 kips. Other types of failure, including local buckling of the outer edge of the flange, may control the magnitude of the allowable load. These will be considered in articles and examples which follow.

4–17. Other Formulas for Allowable Compressive Flexural Stress. The method for aluminum, while it includes most of the factors affecting the critical buckling stress for compression flanges of beams, is quite complex. Many simpler ones are in common use. The simplifications are obtained by neglecting the effect of variation of one or more of the factors which affect the critical compressive stress.

The greatest simplification is present in the equations commonly used for steel. Generally, these equations are similar to column formulas, except that the flange width b is used instead of the radius of gyration of the compression flange

$$\left(r = \sqrt{\frac{I}{A}} = 0.289b\right).$$

The AREA formula, which includes only the effects of the laterally unsupported length L, the flange width b, and the material properties, is

$$F_b \text{ allowable} = 18{,}000 - 5\left(\frac{L}{b}\right)^2.$$

The formula of the 1946 AISC specification includes the effects of the material properties, the laterally unsupported length L, the flange width b, the flange thickness t, and the beam depth. The AISC formula is

$$F_b \text{ allowable} = \frac{12{,}000{,}000}{Ld/bt}, \text{ but not more than } 20{,}000 \text{ psi.}$$

If the flexural compressive stress allowed by the AISC is plotted against values of Ld/bt, a curve like that shown by Fig. 4–16 results. The similarity of this curve to one given earlier in this chapter for compression members is apparent.

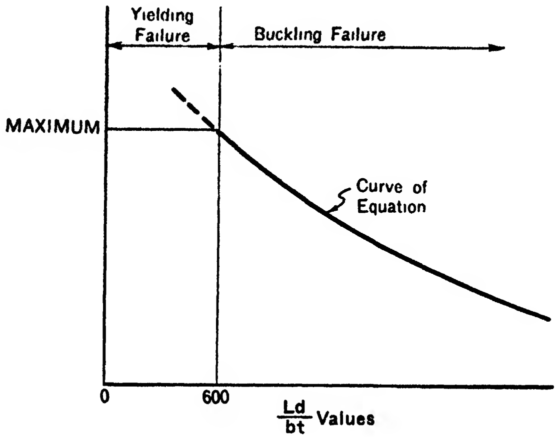


FIG. 4–16.

Specifications having formulas which include L and b as the only variables usually limit the ratio L/b to a stated maximum of 40. The reasons for limiting L/b are the same as those given for limiting the slenderness ratio L/r of columns.

The length L to be used in the formulas for allowable stress is the distance in inches between points of lateral support, or the distance from the point of contraflexure to a lateral support or to another point of contraflexure. The length L to be used for a cantilever beam, laterally free at its outer end, is twice the actual length.

Tests and experience have shown that the simpler formulas are safe when used within the limits of the specification, even though they may not be economical. Methods of analysis that include more of the variables may eventually find their way into specifications for the design of steel beams.

4–18. Design of Laterally Unsupported Beams. No matter which specification is used, there is no direct method of choosing a section for a laterally unsupported beam (except by using a table of beam sections and allowable loads). Selection must be made by successive approximation or, as it is commonly called, trial and error. A trial section is chosen and the computed unit stress is compared with the allowable unit stress. If the computed unit stress exceeds the allowable, the beam is unsatisfactory, and a stronger section must be chosen for a second trial. As many trials as necessary are made to determine the lightest section for which the computed stress does not exceed the allowable.

Such a procedure may at first seem unscientific to the student. Yet, because of the number of unknown factors involved, it is the only practical method to use. Successive approximation is used in many types of engineering problems. In fact, the processes of long division and of finding the square root of a number are well known applications of the trial-and-error method.

EXAMPLE 4–15. Consider again the beam of Ex. 4–13, but without lateral support of the compression flange. Until a section is chosen, the flange dimensions and beam depth are unknown, and the allowable compressive flexural stress cannot be determined. Solution of the problem may be started by estimating the allowable stress and using it to select a trial section.

Assuming that the allowable flexural stress F_b is 15,000 psi,

$$S \text{ required} = \frac{M}{15{,}000} = \frac{498{,}000}{15{,}000} = 33.2 \text{ in.}^3$$

For the first trial section a 12 WF 27 will be used. For this section S is 34.1 in.^{3}, the depth d is 11.95 in., the flange width b is 6.500 in., and the flange thickness t is 0.400 in. The allowable compressive flexural stress F_b is determined by substituting for L, t, b, and d in the 1946 AISC formula. Thus,

$$F_b = \frac{12{,}000{,}000}{Ld/bt} = 10{,}900 \text{ psi.}$$

The computed flexural stress is

$$f_b = \frac{498{,}000}{34.1} = 14{,}600 \text{ psi.}$$

The computed stress exceeds the allowable; therefore, the 12 WF 27 is not satisfactory.

Try a 12 WF 31 for which S is 39.4 in.^{3}, d is 12.09 in., b is 6.525 in., and t is 0.465 in. Substituting in the AISC formula, the allowable stress is

$$F_b = 12{,}547 \text{ psi.}$$

The computed flexural stress is

$$f_b = \frac{498{,}000}{39.4} = 12{,}640 \text{ psi.}$$

Since the computed stress is practically equal to the allowable, the 12 WF 31 is satisfactory flexurally. An analysis of the 14 WF 30, the next lighter section in the next deeper series of beams, shows it to be unsatisfactory; therefore, the 12 WF 31 is the most economical section for this beam. Although an 8 WF 31 is satisfactory from the strength standpoint, it will have greater deflection.

EXAMPLE 4–16. Assume that the weight of the beam in Fig. 4–17 is 60 lb per lin ft. The beam has lateral support at the midpoint and the ends only. Choose a rolled steel section, using the AREA specification.

Because of lateral support at the center of the span, the top flange cannot buckle as shown in Fig. 4-15, but can fail by buckling in two segments as shown in Fig. 4-17. The half of the beam on one side of the center of the span will have a curvature similar to the entire span of Fig. 4-15. The limiting value of the compressive stress will be a function of L/b, in which L is the length of one loop, or one-half the span.

It should be noted that a lateral support does not affect the bending moment due to the vertical loads. The lateral support affects only the *ability* of the beam to resist loads in a vertical plane without failure by horizontal buckling.

The bending moment is

$$M = \frac{35{,}000 \times 30}{4} + \frac{60 \times 30 \times 30}{8} = 269{,}250 \text{ ft-lb.}$$

In order to start the solution, the value of the allowable compressive stress is estimated. Assuming a reasonable flange width (say, 8 in.), $L/b = (12 \times 15)/8 = 22.5$, and the allowable compressive stress by the AREA specification is

$$18{,}000 - 5\left(\frac{L}{b}\right)^2 = 15{,}470 \text{ psi.}$$

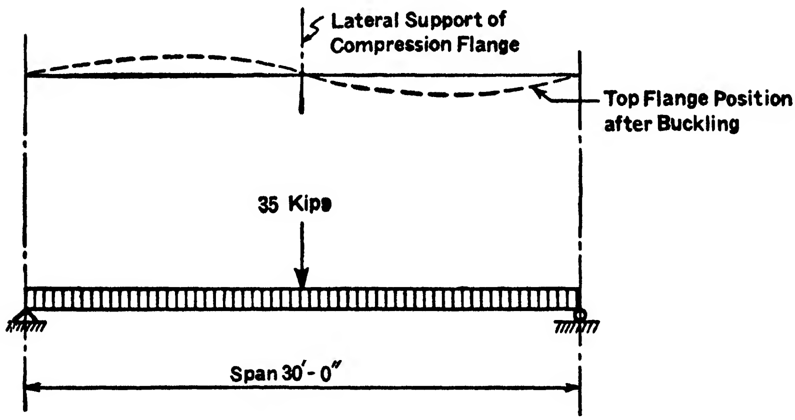


Fig. 4-17.

If the allowable compressive flexural stress is 15,470 psi, the required section modulus is

$$S = \frac{12 \times 269{,}250}{15{,}470} = 209 \text{ in.}^3$$

Try a 24 W 94. For this section, $S = 220.9$ in.^{3} and $b = 9$ in.; hence,

$$\frac{L}{b} = \frac{12 \times 15}{9} = 20.$$

Then

$$F_b \text{ allowable} = 18{,}000 - 5(20)^2 = 16{,}000 \text{ psi,}$$

and the computed value of f_b is

$$\frac{M}{S} = \frac{12 \times 269{,}250}{220.9} = 14{,}600 \text{ psi.}$$

The computed actual compressive unit stress does not exceed the allowable; therefore, the trial section is satisfactory. (If the assumed weight of the beam is corrected and the actual stress recomputed, the section may still be shown to be satisfactory.)

Since the AREA requirement that the depth of a rolled beam shall be preferably not less than one-fifteenth of the span is satisfied by the chosen section, it is assured that excessive vertical deflection will not occur.

EXAMPLE 4–17. Is the 12 I 31.8 shown in Fig 4–18 flexurally satisfactory under the AISC specification? Lateral support is provided by the crossbeam at each support, but not at the loaded end.

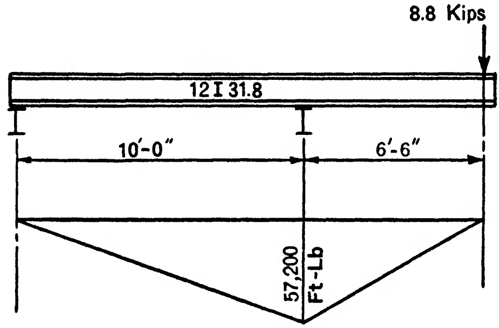


FIG. 4–18.

The bending moment is indicated by the diagram shown under the beam. The computed actual stress is

$$f_b = \frac{M}{S} = \frac{12 \times 57{,}200}{36.0} = 19{,}100 \text{ psi.}$$

The end of the beam, at which the vertical load is applied, is free to move horizontally; hence, the bottom flange in buckling acts in a manner similar to the column of Fig. 4–7(*c*). The effective length L to use in determining the allowable stress F_b for the portion between supports is the distance between supports, or, for the overhanging portion, twice the distance from the right end to the right support. The allowable flexural compressive stress for the overhanging portion is

$$F_b = \frac{12{,}000{,}000\ (5 \times 0.544)}{(2 \times 78 \times 12)} = 17{,}400 \text{ psi.}$$

The computed stress exceeds the allowable, and so the section is unsatisfactory. A comparison of f_b with F_b was made for the overhanging part only, since F_b is lower for that part, while f_b is the same for both parts. With other moment diagrams, comparisons might need to be made for each portion.

EXAMPLE 4–18. A beam continuous over three supports is illustrated by Fig. 4–19. Is a 16 WF 36 satisfactory according to the unit stresses allowed by the AISC specification? The beam is supported from above in such a manner that lateral restraint cannot be assumed for the compression (bottom) flange at the center support. The bending moment diagram is shown below the beam on Fig. 4–19.

The strength of the section must be checked in two locations: at the points of maximum compressive stress in the top flange and maximum compressive stress in the bottom flange.

For the top flange the effective L to use in determining the allowable F_b is the distance from the end of the beam to the point of centraflexure, or 19.5 ft. For the bottom flange the effective L is the distance between points of contraflexure, or 13.0 ft. Substituting in the AISC formula, the allowable stresses are obtained.

For the top flange, $F_b = 9{,}680$ psi.

Since slight yielding of the beam at the support does not reduce the ultimate load capacity of a continuous beam, the AISC specification allows a 20 per cent increase in the allowable bending stress at the support. Thus, for the bottom flange,

$$F_b = 14{,}520 \text{ psi} + 20 \text{ per cent, or } 17{,}420 \text{ psi.}$$

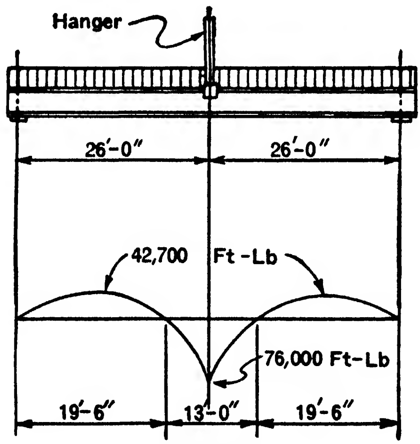


FIG. 4–19.

The section modulus of the section is 56.3 in.³ The computed actual stresses f_b are

$$\frac{12 \times 42{,}700}{56.3} = 9{,}100 \text{ psi for the top flange,}$$

and

$$\frac{12 \times 76{,}000}{56.3} = 16{,}200 \text{ psi for the bottom flange.}$$

The computed stresses f_b are less than the allowable F_b. Thus, the beam is satisfactory in flexure.

4–19. What is Lateral Support? It is occasionally difficult to decide whether connecting members and other parts of the structure actually do give lateral support to the compression flange of a beam. The cases illustrated by Fig. 4–20 and described in the ensuing paragraphs may help to clarify the problem.

The beam in Fig. 4–20(*a*) is obviously laterally supported because lateral movement of its compression (top) flange would require great deformation of the concrete slab in which it is recessed. In (*b*) however, lateral movement is prevented only by static friction between the floor material and the

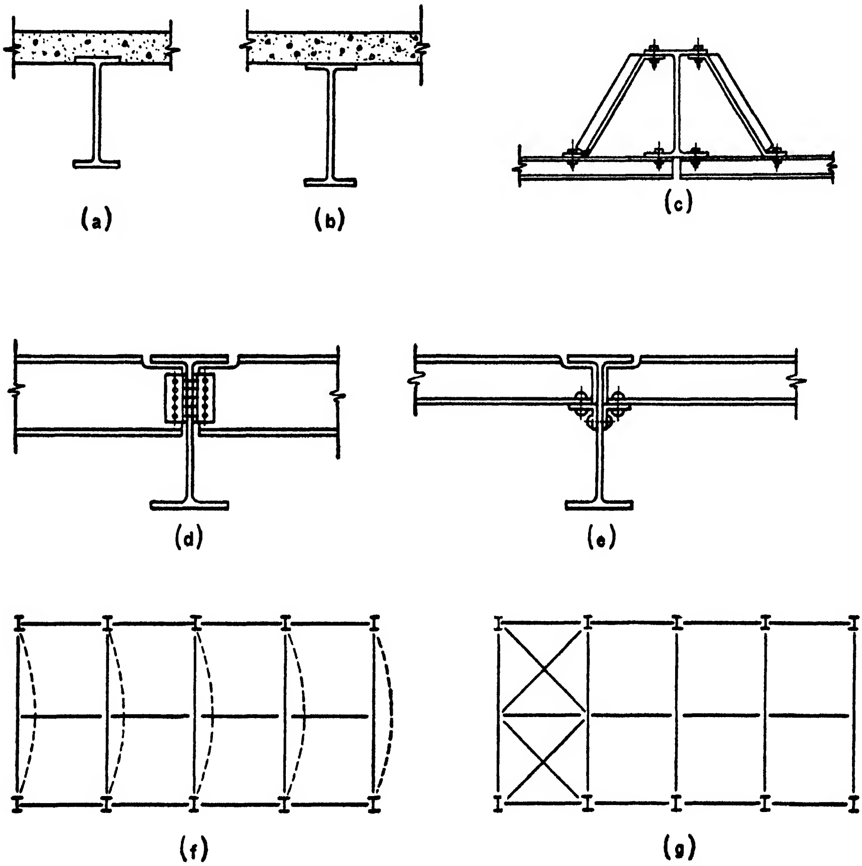


Fig. 4–20.

top flange of the beam. If the friction is great enough, lateral support may be assumed. There is no rational method of determining how much friction is required, although some attempts have been made to provide methods suitable for design use. The lateral force required to prevent buckling is, theoretically, zero for a straight beam, and is very slight even for those beams having initial crookedness equal to the full mill tolerance. It is reasonable, therefore, to assume that lateral support is provided by the friction when the majority of the flexural stress is due to loads which contribute also to the frictional resistance. If the load is a movable one, however, which may cause vibration or impact, or if it is eccentrically applied so as to cause some initial torsion, it would be preferable not to depend on friction to provide lateral support.

The braces of part (*c*) of Fig. 4–20 may be expected to give adequate lateral support to the flange of the upper beam; but when one of the lower members deflects downward and the other upward, bending of the flange will occur.

The larger beam of part (*d*) is laterally supported by adjoining members having connections of appreciable depth and having sufficient stiffness to prevent twisting of the larger beam. Deflections and rotations of the attaching beams may cause horizontal bending of the larger beam. For such a detail, lateral support can be assumed when the connection is made near the compression flange. If the connection is close to the tension flange (for example, assume that the bottom flange of Fig. 4–20(*d*) is in compression), it is questionable whether lateral support should be assumed.

Adjoining members having connections such as illustrated by part (*e*) should not be relied upon for lateral support, unless some system of lateral bracing is provided to prevent their longitudinal movement. [See also (*f*) and (*g*).]

Adequate lateral bracing is not provided by the crossbeams of part (*f*) because lateral buckling of all the girders may occur, as shown by the dotted lines. One girder cannot be considered to brace the adjacent parallel girder. To prevent such buckling, members may be provided to brace the system so that longitudinal movement of the crossbeams is prevented. Part (*g*) illustrates this type of bracing. For lateral movement of the girder (or longitudinal movement of the crossbeam) to occur, elongation or shortening of the diagonals would be required. The diagonal bracing need not be placed in all bays, since movement in any one bay requires movement in the others also.

The material connected to the crossbeams of (*f*) or (*g*) could be considered as doing the job of diagonal bracing if that material were rigid enough. A concrete slab, or a steel deck, connected firmly to the crossbeams could be considered to brace the system and could possibly eliminate the need for the diagonal bracing of (*g*).

4–20. Deflection and Depth Ratios. The amount of deflection permissible for a beam is affected by (1) the probable damage by deflection to attached materials, (2) the importance of good appearance in the completed structure, and (3) the effect of deflection on the function of the completed structure.

The amount of deflection can be computed by a method such as elastic weights or by application of standard formulas for deflection under the more usual type of loading. For example, the AISC handbook, *Steel Construction*, gives expressions for the deflection of simple and fixed-end beams under concentrated, uniform, and uniformly varying loads. An unusual load condition can often be broken down into a few elementary loads, and the deflections of the elementary loads can be added together to give the deflection for the complete condition.

Specifications may include a limiting allowable ratio of depth of section to span. It will be shown that such a limit serves, for a particular load condition and fiber stress, to control deflection. For a simple beam having a uniform load w per unit of length,

$$\Delta = \frac{5wL^4}{384EI} \text{ (from handbook, or by elastic weights),}$$

$$M = \frac{wL^2}{8},$$

$$f = \frac{Mc}{I} = \frac{wL^2d}{16I},$$

$$\frac{f}{d} = \frac{wL^2}{16I}.$$

Substituting the last expression in the equation for deflection,

$$\Delta = \frac{5L^2f}{24Ed}.$$

Solving for d/L,

$$\frac{d}{L} = \frac{5Lf}{24E\Delta}.$$

For Δ equal to 1/360 of the span L and for a beam stressed to a full allowable stress of 20,000 psi,

$$\frac{d}{L} = \frac{5L \times 20{,}000 \times 360}{24 \times 30{,}000{,}000 \times L} = \frac{1}{20}.$$

Similarly for Δ of 1/300 of the span, and f of 20,000 psi,

$$\frac{d}{L} = \frac{1}{24}.$$

Limiting depth-span ratios, such as the two illustrated, are given by many specifications.

It should be noted that deflection is limited to 1/360 of the span to prevent cracking of plaster. The amount of deflection causing cracking is only that resulting from loads applied after the hardening of the plaster. The total deflection for such a beam may then exceed $L/360$, provided that the portion of the deflection caused by live load alone does not exceed $L/360$.

4-21. Shear in Beams. In most texts on strength of materials is found the derivation of the formula $v = VQ/Ib$, in which v is the intensity of shearing stress at the point in question; V is the total shear at the section; Q is the statical moment of the area between the extreme fiber and the point in question, about the center of gravity axis of the entire section; b is the width or thickness (perpendicular to the plane of bending) at the point in question; and I is the moment of inertia of the entire cross section about the center of gravity.

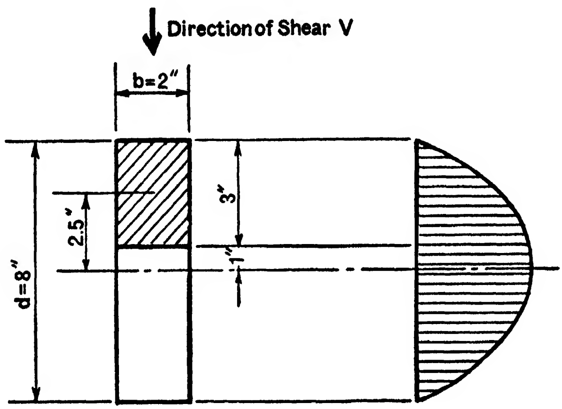


Fig. 4-21.

The shear formula is based on the same assumptions as the flexure formula, $f = My/I$; therefore, the limiting conditions of the flexure formula apply also to the shear formula.

If a shearing unit stress of intensity v occurs on one plane, an equal unit shearing stress occurs on a plane through that point perpendicular to the first plane. The shear formula may then be used to determine either transverse or longitudinal shearing stresses in a beam.

To illustrate the application of the formula, find the shearing stress in the beam section of Fig. 4-21 at a point 1 in. above the center of gravity of the section when the total shear V is 10,000 lb. The value of Q is the moment of the shaded area about the center of gravity of the entire section. Then

$$Q = 2 \times 3 \times 2.5 = 15.0 \text{ in.}^3,$$

$$I = \frac{2 \times 8 \times 8 \times 8}{12} = 85.3 \text{ in.}^4$$

The unit shear at 1 in. from the center of gravity is $v = VQ/Ib$, or

$$v = \frac{10{,}000 \times 15}{85.3 \times 2} = 879 \text{ psi.}$$

If v is computed for points at various distances from the center of gravity and the values are plotted, a parabola results, as shown on the right part of Fig. 4–21. At the center of gravity

$$Q = \frac{bd^2}{8} \quad \text{and} \quad I = \frac{bd^3}{12},$$

so that

$$V = \frac{VQ}{I_b} = \frac{3V}{2bd}.$$

The quantity V/bd is the average shearing stress. It is important to remember that, for any rectangular section bending as a beam, the maximum unit shear is 1.5 times the average unit shear.

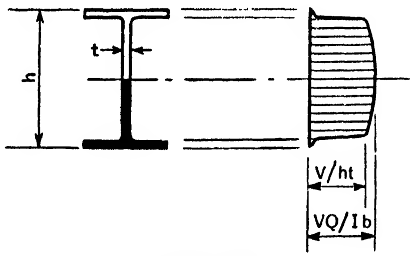


Fig. 4–22.

In a beam having relatively large flanges and a thin web, such as an I-beam, the intensity of shear varies as shown by Fig. 4–22. From this figure it can be seen that the flanges do little to resist the shear V, whereas the web resists most of the shear. To illustrate this fact, assume that the section shown is a 14 WF 30 and that the total shear V is 10,000 lb. To determine the intensity of shear at the center of gravity, let Q be the moment of the half-beam about the center of gravity of the entire section. Dimensions and properties of the half-beam may be found in the *AISC Manual* under the heading "Structural Tees Cut from WF Beams." The section which is one-half of a 14 WF 30 is listed as ST 7 WF 15. The distance from the center of gravity of the tee to the outer fibers of the flange is 1.59 in., the depth of the tee is 6.93 in., and its area is 4.41 in.^{2} Therefore,

$$Q = 4.41(6.93 - 1.59) = 23.5 \text{ in.}^3$$

At the center,

$$v = \frac{VQ}{Ib} = \frac{10{,}000 \times 23.5}{289.5 \times 0.27} = 3{,}000 \text{ psi.}$$

Compare the above value with that obtained when it is assumed that the shear is distributed uniformly over the web and that the flanges offer no help in resisting shear; by this assumption $v = V/ht$.

If h is the over-all depth of the section, then

$$v = \frac{10{,}000}{13.86 \times 0.27} = 2{,}670 \text{ psi.}$$

If h is the clear depth of the web between flanges,

$$v = \frac{10{,}000}{13.09 \times 0.27} = 2{,}830 \text{ psi.}$$

The answers obtained from this assumption of uniform distribution on web only are, respectively, 11.0 and 5.7 per cent lower than those found by use of the shear formula. For other sections of the I-beam or channel type, the error resulting from the use of the approximate method is similarly small. Since the error is small, it is possible to design beams of this type with safety, using the approximate method, providing the allowable unit shearing stress is established with the knowledge that the approximate method will be used. It is usually assumed in the design of flanged beams that the allowable unit shearing stress on webs given by the specification is the average shear, rather than the maximum intensity, assuming that the web takes all the shear. The shear strength of beams is checked by this method in the majority of structural designs, but the reader should again be reminded that the error will be great if the approximate method is used for a section not having relatively large flanges and a light web.

In the preceding example, h was taken first as the over-all depth and then as the clear depth between flanges. Engineers do not agree on which to use; it is the more common, however, to use the over-all depth for beams and the web dimensions for plate girders.

4–22. Effect of Shearing Stress in a Web. A small element of a beam web is shown in Fig. 4–23 and is acted upon by the forces shown. It is assumed that the element is taken from a region where the flexural stresses are zero. On the left and right sides equal unit shearing stresses v_1 occur, which are caused by the external shear V in the beam. To maintain equilibrium of the element, equal shearing stresses v_2 occur at right angles to v_1 and in such direction that the moment couple of stresses v_2 is equal to and opposite to that of v_1. Thus, when a unit shearing stress occurs in one direction, an equal unit shearing stress exists in a direction perpendicular to the first for any element of material from a beam web or elsewhere.

Consider the element cut along line a–a. In order that the triangular portions formed may be in equilibrium, it is necessary that an internal tension T exist to balance the resultant R_1 of the shearing forces v_1 and v_2. If the element is cut on line b–b, a compressive force C is necessary to ensure equilibrium.

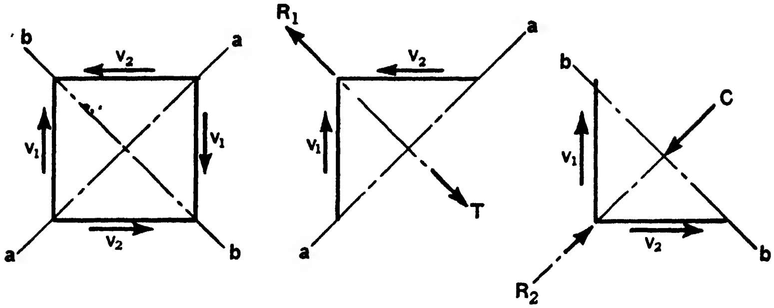


FIG. 4-23.

The web of a beam has flexural and shearing stresses, and as a result of shear, has diagonal tension and diagonal compression stresses. In the web of the beam of Fig. 4-24, for example, tensile stresses occur parallel to the strips marked *a–a*, while along strips *b–b* the stress is compressive. At the ends of the beam, where the bending moment is zero, the intensity of the tensile or compressive stress is the same as that of the unit shearing stress at that point, but in diagonal directions. In regions where the flexural stresses in the web ($f = My/I$) are appreciable, the diagonal tensile or compressive stresses are combinations of the flexural stresses and the tensile or compressive stresses due to shear. Thus, the directions of the maximum web tensile or compressive stresses (principal stresses) vary according to the proportions of the shearing and flexural stresses in the web.

If the unit compressive stress on the line *b–b* is high enough, that element will buckle as a column. Such buckling (diagonal buckling) causes a series of waves or wrinkles approximately parallel to line *a–a*.

The intensity of compressive stress at which such buckling occurs depends on the slenderness of the diagonal strip and on the elastic properties of the material. Failure by buckling will occur under lower stress if the length of the diagonal strip is great and the thickness small, as compared with a short and thick element. Because of the varying direction of the compressive stress and the restraint to buckling offered by the diagonal tensile strips, determination of the critical buckling stress is complicated.

Empirical equations including the effects of intensity of shear stress, thickness of web, and depth of section, but having constants established

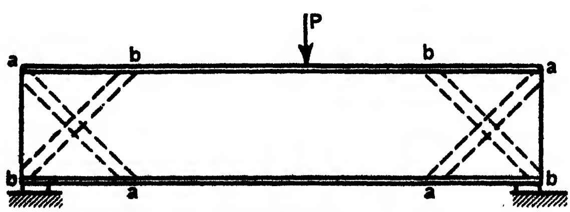
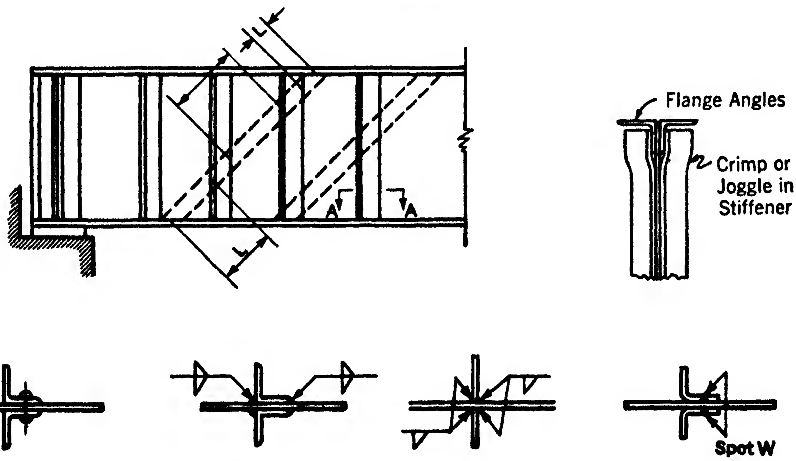


FIG. 4-24.

by experiment or experience, are used to determine measures of control of diagonal buckling. Prevention of diagonal buckling is accomplished either by keeping the diagonal compressive stress low or by making the buckling resistance high through limitation of the ratio of length of the element to thickness of the web.

The unit diagonal compressive stress, which is about equal to the unit shear in regions of low bending moment, can be kept low enough to prevent diagonal buckling by the selection of a section with a relatively large web area.

The ratio of length to thickness of the diagonal strip can be lowered (and the critical buckling stress increased) either by reducing the length of the element or by increasing the web thickness. The length of the element may be reduced either by reducing the depth of the section or by installing "stiffeners" which hold the diagonal strips of the web, thus decreasing their laterally unsupported lengths and increasing their critical buckling stress.



Section A--A Showing Alternate Methods

Fig. 4–25.

Stiffeners are usually placed vertically on both sides of the web, as shown in Fig. 4–25. When used on built-up members, stiffeners of this type are often crimped or joggled. In this case, if the sole purpose of the stiffener is to restrain the web against diagonal buckling, it need not bear tightly against the flange.

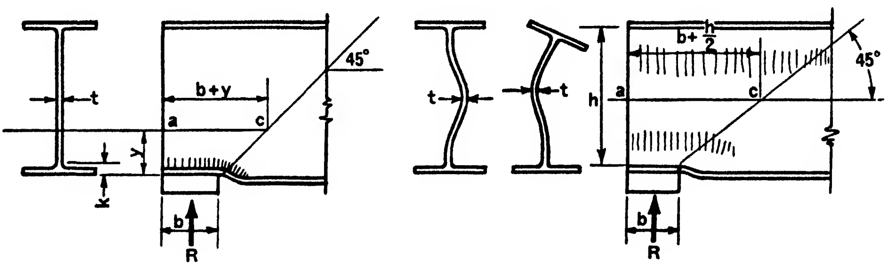
The spacing of the stiffeners along the beam must be such that the effective lengths L of the diagonal strips in compression will cause the ratio L/t to be small enough so that those strips will not buckle under the load for which the beam is designed. The determination of the spacing is accomplished by application of empirical formulas for that purpose. The determination of stiffener spacing for plate girders is graphically illustrated in Chap. 6.

4-23. Web Strength at Concentrated Loads or Reactions. A heavy load or reaction concentrated on a short length of the member may cause one of the types of web failure shown in Fig. 4-26. These types of failure result from compressive stresses in the vertical elements of web directly above or below the concentrated reaction or load.

4-24. Web Crippling. In Fig. 4-26(a) is shown the failure known as either *web crushing* or *web crippling*. Obviously, the reaction R is transmitted not only to the vertical strip directly above the length b but also, by shearing stress, to the adjacent strips of web. It is assumed by most specifications that the vertical compressive load is distributed over the width from a to c of Fig. 4-26(a), where c is a point on a line drawn from the edge of the load or reaction at an angle of 45° to the flange. The unit vertical compressive stress at a distance y from the flange is given by

$$f_c = \frac{R}{\text{area}} = \frac{R}{(b + y)t},$$

in which t is the thickness of the web at y distance from the flange.



(a) WEB CRIPPLING (b) WEB VERTICAL BUCKLING

FIG. 4-26.

If web crushing or crippling occurs, it does so at the points of highest compressive stress. This stress occurs where the area $(b + y)t$ is least. The area is least where the web meets the fillet of the beam, at distance k from the edge. The unit stress at this point is given as

$$f_c = \frac{R}{(b + k)t},$$

from which

$$b = \frac{R}{f_c t} - k.$$

If the load or reaction does not occur near the end of the beam, distribution is assumed along a 45° line at each end of length b, so that

$$f_c = \frac{R}{(b + 2k)t},$$

or

$$b = \frac{R}{f_c t} - 2k.$$

The equations for web crippling are most often used to determine the required length of bearing b, over which the load or reaction must be spread so that f_c will not exceed the allowable value. The 1949 AISC specification permits f_c to be not more than 24,000 psi.

The 1950 *Structural Aluminum Handbook* suggested a formula which differs from those just presented, in that distribution was assumed to start at the inside of the flange rather than at the outside. This formula is $f_c = R/bt$.

The unit stresses allowed by each specification agree with the method specified for the calculation.

EXAMPLE 4–19. Compute the length of bearing (b) required at the end of a 20 I 65.4 having a reaction of 90 kips. Use the AISC specification.

The web thickness given by the *Steel Handbook* is 0.50 in., and the distance k from the outer fibers to the point of tangency of web and fillet is 1.56 in.

The required bearing length is

$$b = \frac{90}{24 \times 0.50} - 1.56 = 5.94 \text{ in.}$$

A 6-in. bearing length would be used.

Often the bearing length, as computed above, is very small or even negative. In such cases the bearing length to be used is determined by practical considerations, such as the length required for safety during erection of the beam, the need for holes in the beam flange or the bearing plate, the width of the brickwork on which the bearing plate rests, or the bearing stress in the supporting masonry.

4–25. Vertical Buckling. The failure illustrated by Fig. 4–26(b) is called *vertical buckling*. It occurs when the intensity of vertical compressive stress near the center of the section becomes greater than the critical buckling stress for the web acting as a column.

Originally, consideration of vertical buckling was required by most specifications. Tests* indicate, however, that for rolled structural steel beams the initial failure is by web crippling rather than by buckling. Although the test admittedly covered a number of beams "too small on which to base final conclusions," the later specifications for rolled structural steel beams were changed to require investigation of web crippling but not of vertical buckling.

* Inge Lyse and H. J. Godfrey, "Investigation of Web Buckling in Steel Beams." *Trans. ASCE*, Vol. 100 (1935).

For built-up beams having greater ratios of depth to thickness of web, failure by vertical buckling may still be more probable than failure by web crippling. Calculation of the resistance to vertical buckling is made unnecessary, however, by the requirement of steel specifications that bearing stiffeners be provided for plate girders at all concentrated loads or reactions.

Since the elastic properties of the material affect its resistance to buckling, the tests made on steel beams are not used as criteria for other metals. For aluminum alloys, the 1950 *Alcoa Structural Handbook* recommended consideration of both web crippling and vertical buckling. It is assumed for aluminum beams that the critical section in vertical buckling is at mid-height and that the compressive stress is distributed over the length of web *ac*, as shown in Fig. 4–26(*b*).

For an end reaction or load, distribution is over a length $(b + h/2)$, and

$$f_c = \frac{R}{t(b + h/2)};$$

for an interior reaction or load, distribution is over a length of $(b + h)$, and

$$f_c = \frac{R}{t(b + h)}.$$

The term h is the clear height between flanges.

The stress f_c is limited to that allowable for a column having length equal to the clear height of the web and thickness equal to that of the web.

EXAMPLE 4–20. What length of bearing is required to provide a factor of safety of 2.0 against failure in web compression of a 6061-T6 aluminum alloy beam, 12 I 11.31, having an end reaction of 30 kips? The dimensions given by the *Alcoa Structural Handbook* are as follows: web thickness t, 0.350 in.; over-all depth, 12 in.; and clear depth h between flanges, approximately 10.5 in.

The two solutions must be made: one for prevention of web crippling and one for prevention of vertical buckling. The solution giving the larger bearing length will control the design.

Web crippling is considered first. The compression in the web at the toe of the fillet is given as

$$f_c = \frac{R}{bt}.$$

Solving for the bearing length,

$$b = \frac{R}{f_c t}.$$

Aluminum alloys have no definite ultimate compressive stress, but they do have a compressive yield strength equal to the tensile yield strength. Assuming that the factor of safety is referred to the yield strength (35,000 psi),

$$b\ (\text{required}) = \frac{30{,}000}{\frac{35{,}000}{2} \times 0.350} = 4.90 \text{ in.}$$

At the mid-depth of the web, the vertical compressive stress is given as

$$f_c = \frac{R}{t\,(b + h/2)}.$$

Solving for the required bearing length,

$$b = \frac{R}{f_c t} - \frac{h}{2}.$$

The critical buckling stress for rolled sections of 6061-T6 is given by the expression

$$\frac{P}{A} = 38{,}300 - 202\,\frac{KL}{r},$$

for values of KL/r not exceeding 63, and by

$$\frac{P}{A} = \frac{102{,}000{,}000}{\left(\frac{KL}{r}\right)^2},$$

for values of KL/r greater than 63.

Assuming K to be 0.67,

$$\frac{KL}{r} = \frac{0.67 \times 10.5}{0.29 \times 0.350} = 69.3.$$

Substituting for KL/r in the first equation gives the critical buckling stress as

$$38{,}300 - (202 \times 69.3) = 24{,}300 \text{ psi}.$$

For a factor of safety of 2 with respect to this value,

$$b\ (\text{required}) = \frac{30{,}000}{\frac{24{,}300}{2} \times 0.350} - \frac{10.5}{2} = 1.80 \text{ in.}$$

Web crippling controls, since its prevention requires the greater length b.

4–26. Bearing Stiffeners. Either web crippling or vertical buckling can be prevented by one of three different methods. These methods are: (1) to increase the bearing length b; (2) to increase the web thickness t; or (3) to provide bearing stiffeners at points of concentrated loads or reactions.

The function of the bearing stiffener is to receive the reaction or load directly at its end, acting as a compression member. Through the welds or rivets which connect the stiffener to the web, the vertical force is transferred into the web. Thus, an intense vertical compressive force in the web is avoided. The load is immediately distributed over the height of the web as shear.

The design of the stiffener is, then, the design of a compression member. Consider the ways in which such a member might fail.

1. It might be crushed at its end, where the vertical load is transferred to it through the flange of the beam.
2. It might fail through high compressive stress farther up in the stiffener.

3. Its outer edge might wrinkle in a local buckling failure.
4. In an extremely high member the entire beam, stiffeners included, might buckle as in vertical buckling, shown by Fig. 4-26(*b*).

Prevention of the first type of failure is accomplished by limiting the bearing stress on the contact area. The AISC specification, Section 15(a)(5), shows two values for this stress. The one to use depends on the manner in which the contact end is made smooth. The contact area is of width *a* only (Fig. 4-27). Even if an attempt were made to fit the fillet, only the flat area should be considered. Stiffeners for rolled beams should be ground to fit the flange. They cannot effectively be milled because the flanges are seldom at right angles to the web. Design drawings should specify which end of the stiffener is to be milled or fitted.

The second type of failure is prevented by limiting the average stress on the gross area of the stiffener. The AISC specification permits 20,000 psi [Section 15(a)(2)].

Local buckling is avoided by limiting the ratio of outstanding width to thickness, w/t. It is usual to allow a maximum of 16 for this ratio on stiffeners. See AISC Section 18(b).

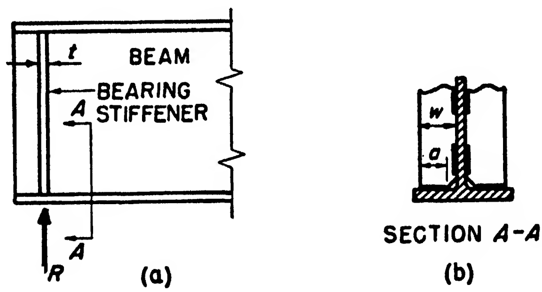


Fig. 4-27.

The fourth type of failure is important only in built-up members, such as very deep plate girders.

The bearing stiffener, since it is a compression member, is not crimped or joggled, but has all spaces beneath it filled with filler plates.

Example 4-21. Assume a 14 WF 30 beam having an end reaction of 40 kips. Only $1\frac{1}{2}$ in. of bearing length is available. Are stiffeners needed? If so, design them in accordance with the AISC specification.

The required length of bearing, without stiffeners, is

$$b = \frac{40}{24 \times 0.27} - \frac{7}{8} = 5.30 \text{ in.}$$

This is more than can be provided. Stiffeners are required.

The stiffeners will be assumed to carry the entire reaction of 40 kips. (If necessary, in order to obtain a reasonable stiffener design, the bearing value of the $1\frac{1}{2}$-in.

length of web could be deducted. The amount that could be carried by the beam web is 15.4 kips, leaving 24.6 kips to the stiffeners. This refinement is usually ignored.)

The end-bearing area required for the stiffeners is

$$A_b = \frac{40}{27} = 1.48 \text{ in.}^2,$$

assuming fitted stiffeners.

The gross area required is

$$A_g = \frac{40}{20} = 2.00 \text{ in.}^2$$

The stiffener is generally made as wide as the width of the beam flanges will permit. A standard bar size is usually selected, so that the stiffener comes close to the edge of the flange but does not project beyond it. The maximum width of space available is dimension a of "Dimensions for Detailing" in *Steel Construction.* For a 14 W 30 this is 3¼ in. A 3-in. bar is the maximum practical width.

The minimum allowable thickness is given by AISC Section 18(b) as $\frac{1}{16}$ of the 3-in. dimension, or $\frac{3}{16}$ in.

To provide a gross area A_g of 2.00 sq in. by two stiffeners, the required thickness is $2.00/2 \times 3 = 0.333$ in.

By subtracting the flange thickness from the k distance, the fillet radius is obtained as $r = \frac{7}{8} - \frac{3}{8} = \frac{1}{2}$ in. Thus the flat width of flange bearing against the stiffener is only $3 - 0.50 = 2.5$ in. per bar. To provide an end-bearing area A_b of 1.48 sq in., the thickness must be

$$\frac{1.48}{(2 \times 2.5)} = 0.296 \text{ in.}$$

The largest of the three thickness requirements is 0.333 in. The stiffener bars used would be 3 x ⅜ minimum.

The balance of the design is a connection problem. The weld joining the bars to the web is required to transfer the stiffener load into the web. The design of welded connections will be presented in Chapter 5.

4–27. Webs in Which Buckling Is Permitted. In the preceding articles, buckling of the web through diagonal compression is treated as a type of failure, the occurrence of which the designer must prevent. However, a beam which fails in diagonal compression does not necessarily experience complete collapse. In fact, it may have considerable moment and shear resisting capacity even after the web has buckled. Occasionally, beams are intentionally designed with such proportions that diagonal buckling does occur. Such beams are known as *tension-field beams, semitension-field beams,* or *Wagner beams.* Because they may be lighter than beams with nonbuckling webs, they are used to advantage in aircraft structures.

Consider a beam cut as shown in Fig. 4–28(a). If the web does not buckle, tension will be induced in web strips inclined as strip a, and compression in those inclined as strip b. If the reaction R is increased sufficiently, strips inclined as strip *b* will buckle, so that further increase of load and shear will be balanced entirely by increase of tensile stress in the strips inclined as strip *a*. Even though buckled, the strips in compression will still resist some portion of the shear. If the web is very thin, however, this amount may be so low that we may consider the shear in the beam to be resisted entirely by the vertical components of the diagonal tensile stresses.

Assuming that the web does buckle diagonally and that it is so strong in diagonal tension that collapse does not ocur, stress changes are induced in the flanges and stiffeners of the beam. In Fig. 4–28 (*a*) (web not buckled) satisfaction of the equation of equilibrium, $\Sigma H = 0$, requires that C_f and T_f be equal; in Fig. 4–28(*b*) (web buckled), the same equation shows that they cannot be equal. Application of the equation $\Sigma M = 0$ will show that C'_f is larger and T'_f smaller than the corresponding flange loads of Fig. 4–28(*a*).

Web stiffeners, unstressed before buckling, are subject to compression after buckling, as is shown by a section of the beam acting as a free body

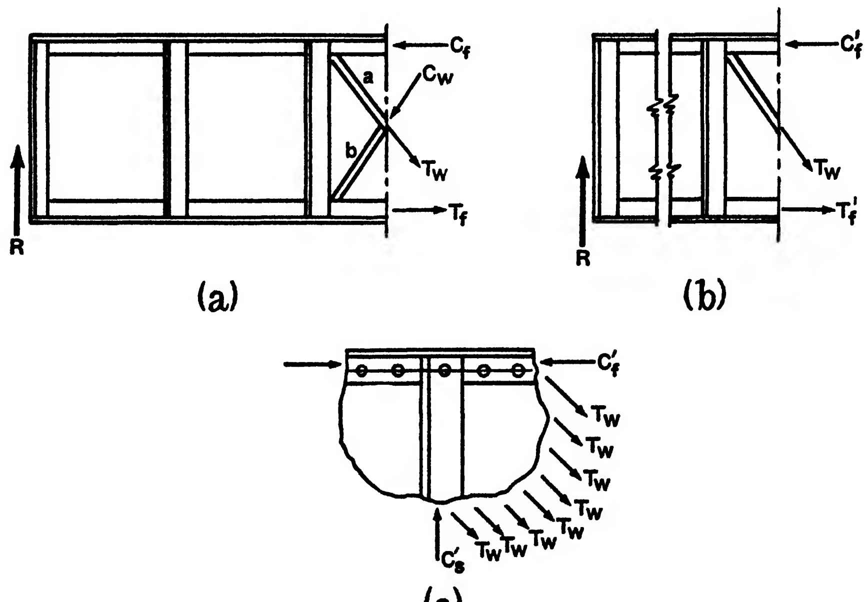
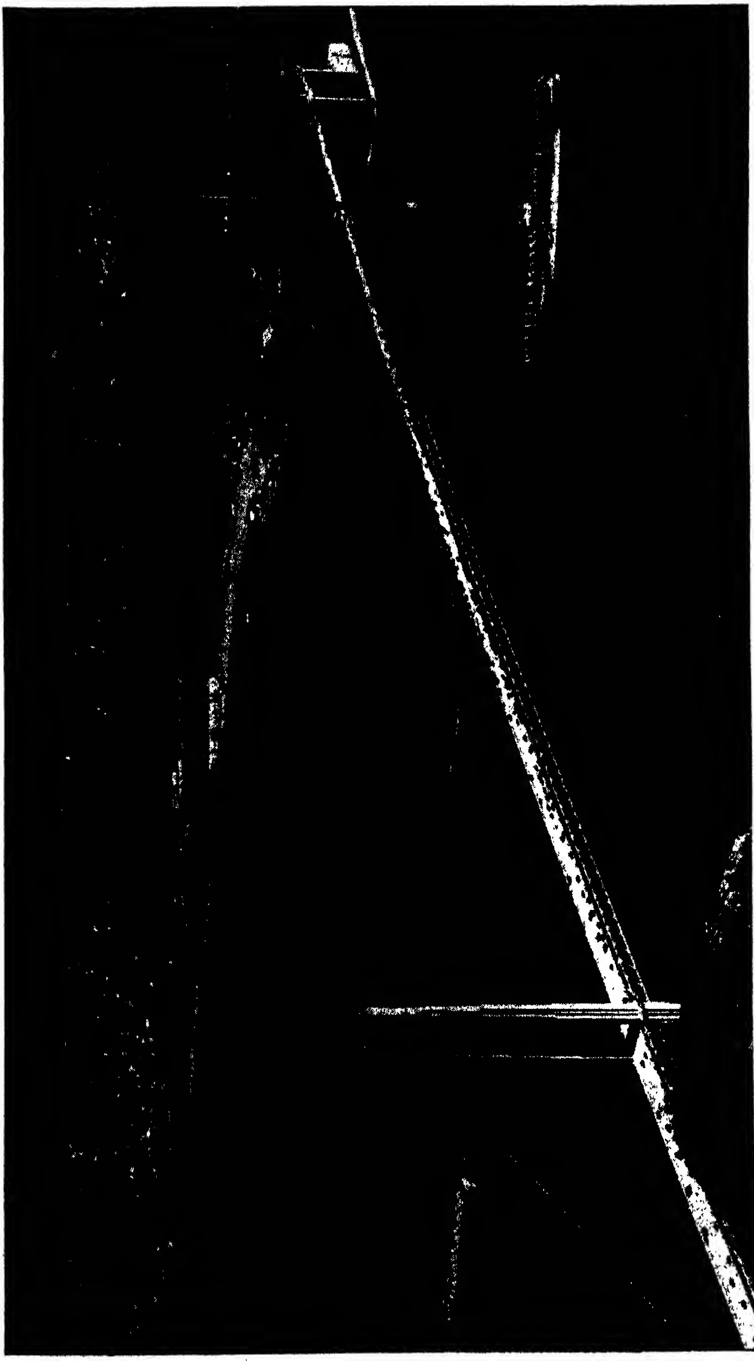


FIG. 4–28.

in Fig. 4–28(*c*). Welds or rivets connecting the flange to the web have inclined loads rather than the usual horizontal shearing loads.

For details of the design of tension-field beams, the student is referred to one of the many excellent textbooks dealing with aircraft structural design.



Bethlehem Steel Co.

GOLDEN GATE BRIDGE, SAN FRANCISCO, CALIF.

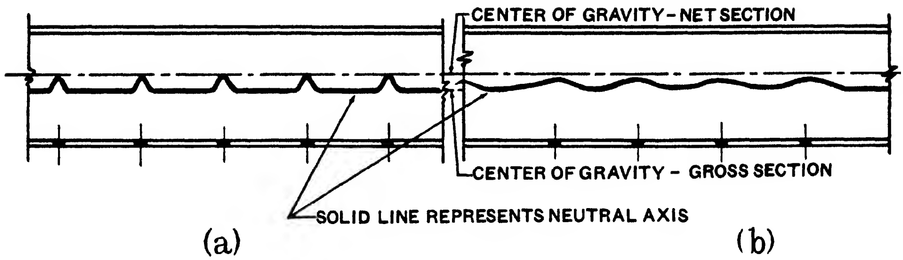


Fig. 4-29.

4-28. Effect of Holes in Beams. A hole in the flange of a beam causes an increase of stress. If the hole is located at or near the section of highest flexural stress, its presence may affect the ultimate strength of the beam. The increase in stress can be considered as of two types. The first is the increase caused by redistribution of the total flexural stresses to a cross section whose area and moment of inertia are smaller than those for the adjacent section without holes. The second is the concentration of stress near the edge of the hole. Stress concentration occurs at any sudden change of section and results from the inability of the stresses to redistribute uniformly over the reduced area. The conditions are analogous to those of water passing a rock in a stream. At the obstruction, the average velocity of the water increases because of the reduced area of the channel. However, the velocity at the section containing the obstruction does not increase uniformly. Next to the obstruction the velocity is much increased, while at other points on the cross section the velocity is only slightly affected. Irregular surfaces and eddies result.

Analysis of stresses in a beam with holes depends on the effect of the holes on the position of the neutral axis. The flexure formula is based on a neutral axis located at the center of gravity of the effective area of the cross section. At a section where holes occur in one flange only, the center of gravity of the remaining (net) section is shifted from the position it occupies in the gross section. It is consistent with the use of the flexure formula to assume that the neutral axis is at the center of gravity of the net section and to use the moment of inertia about that neutral axis.

But if the condition just described were true, the neutral axis of a beam having holes in one flange would shift abruptly at each hole location. It would move from the center of gravity of the gross section to that of the net section, and back again to that of the gross. Figure 4–29(a) illustrates this condition. Recalling that the neutral axis is the locus of points of zero stress, it seems impossible that such fluctuation of the neutral axis can occur. Stress conditions cannot change abruptly enough to allow a complete redistribution of stress within the short length of beam occupied by the hole. If the hole extended over the entire length of beam, the neutral axis would, of course, remain at the net center of gravity. However, if holes occur only at intervals along the beam, a smooth fluctuation of the neutral axis position is more probable. In a beam with widely spaced holes, the neutral axis probably remains close to the gross center of gravity but shifts in the direction of the net center of gravity at the holes. As the space between the holes is decreased, the amount of shift probably increases until, for very closely spaced holes, the neutral axis remains close to the net center of gravity. In any case, the neutral axis is not a sharply deformed line, as in Fig. 4–29(a), but a smoothly fluctuating curve, as shown in Fig. 4–29(b).

Although the neutral axis probably does not shift all the way to the net center of gravity, it is usually regarded as a safe practice to assume that this is true. Three approximate methods have been developed that are intended to save time in calculation. A description of these methods follows.

One of these methods is the same as the theoretical method (described above) except that no shift of the neutral axis is assumed.

The second method, that of the AREA specification (Section 60, "Proportioning Girders and Beams") requires that:

> Plate girders, I-beams, and other members subject to bending that produces tension on one face, shall be proportioned by the moment-of-inertia method. The neutral axis shall be taken along the center of gravity of the gross section. The tensile stress shall be computed from the moment of inertia of the entire net section and the compressive stress from the moment of inertia of the entire gross section.

The AISC specification permits rivet holes to be neglected up to a specified limit. If the rivet holes in a flange do not remove more than 15 per cent of the gross area of that flange, they are neglected. If the rivet holes remove over 15 per cent of the area of a flange, the excess over 15 per cent is considered removed when computing the net properties. The deduction is made for both flanges if rivet holes are present in both flanges. (For countersunk rivet holes and for holes other than rivet holes, the full amount of the holes is considered when computing net properties.) The student should study Section 26(a) of the AISC specification. The AISC specification does not indicate whether a shift of neutral axis is to be assumed when the holes are in one flange only.

EXAMPLE 4–22. Consider a beam section of 18 WF 50 having a bending moment of 1,500 in.-kips and two holes for ⅞-in. rivets in each flange. Analyze this section by each of the four commonly used methods described above. Compare the results obtained by the different methods.

1. *By the Theoretical Method.* The rivet holes in the compression flange are filled by the rivets. The compressive stress from the flange is carried across the hole as compression in the rivet shank. These holes are assumed to have no weakening effect. The holes in the tension flange weaken the section. The center of gravity of the net section is computed first. See Fig. 4–30.

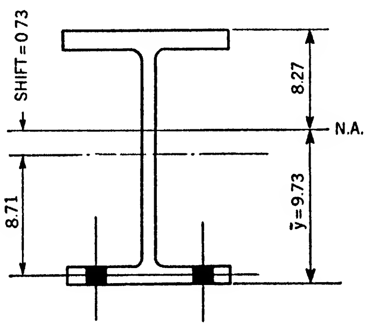


FIG. 4–30.

The effective area of each hole in the tension flange is $0.57 \times 1.0 = 0.57$ sq in. (See Art. 4–3.) Computations for the properties of the section follow:

Section	A	y (to Base)	Ay	Ay^2	I_0
18 WF 50	14.71	9.00	132.39	1,191.51	800.6 in.^{4}
2 holes	−1.14	0.285	−0.33	−0.09	0.0
	13.57		132.06	1,191.42	800.6
					1,191.42
				I about base =	1,992.02 in.^{4}
				$-A\bar{y}^2 = 132.06 \times 9.73 =$	−1,284.94
				I about gravity axis =	707.08 in.^{4}

$$\bar{y} = \frac{132.06}{13.57} = 9.73 \text{ in.}$$

Assuming that the neutral axis is at the center of gravity of the net section, the section modulus for the compression and tension flanges are

$$S_c = \frac{707.08}{8.27} = 85.5 \text{ in.}^3,$$

and

$$S_t = \frac{707.08}{9.73} = 72.7 \text{ in.}^3$$

2. *Holes Considered on Tension Flange Only, No Shift of Neutral Axis.*

$$I = 800.6 - 1.14(8.71)^2 = 714.1 \text{ in.}^4$$

Since the distance to the extreme fiber is alike for both flanges,

$$S_c = S_t = \frac{714.1}{9.0} = 79.3 \text{ in.}^3$$

3. *By the AREA Method.* For computing the compressive stress the gross section is used; S_c is 89.0 in.^{3}

For computing the tensile stress, both the top and bottom holes are assumed to reduce the strength. The neutral axis is not shifted.

$$I = 800.6 - 2.28(8.71)^2 = 627.4 \text{ in.}^4$$

$$S_t = \frac{627.4}{9.0} = 69.7 \text{ in.}^3$$

4. *By the AISC Method.* The flange is 7.50 in. wide. Fifteen per cent of 7.50 is 1.12 in. Two inches of flange width are removed by the holes in one flange. The excess over 15 per cent is a width of 2.0 − 1.12 = 0.88 in. An area reduction totaling 0.88 × 0.57 = 0.502 sq in. must be considered for each flange.

$$I = 800.6 - (2 \times 0.502)(8.71)^2 = 724.3 \text{ in.}^4$$

$$S_c = S_t = \frac{724.3}{9.0} = 80.5 \text{ in.}^3$$

5. *Comparison of the Methods.* Table 4-1 shows a comparison of the section modulus values and of the computed stresses for each of the four methods. It is assumed that the stresses computed by the theoretical method are correct. They are shown as 100 per cent. Stresses by other methods are related to those of the theoretical method by the percentages shown.

Of the three approximate methods illustrated, that outlined by the AREA specifications appears to give results closest to the stresses that may be

TABLE 4-1

Method Used	Tension Flange			Compression Flange		
	S_t	f	Per cent	S_c	f	Per cent
1. Theoretical axis shifted	72.7	20,630	100	85.5	17,540	100
2. Holes from tension flange only; no shift of axis	79.3	18,900	92	79.3	18,900	108
3. AREA method	69.7	21,520	104	89.0	16,850	96
4. AISC method	80.5	18,620	90	80.5	18,620	106

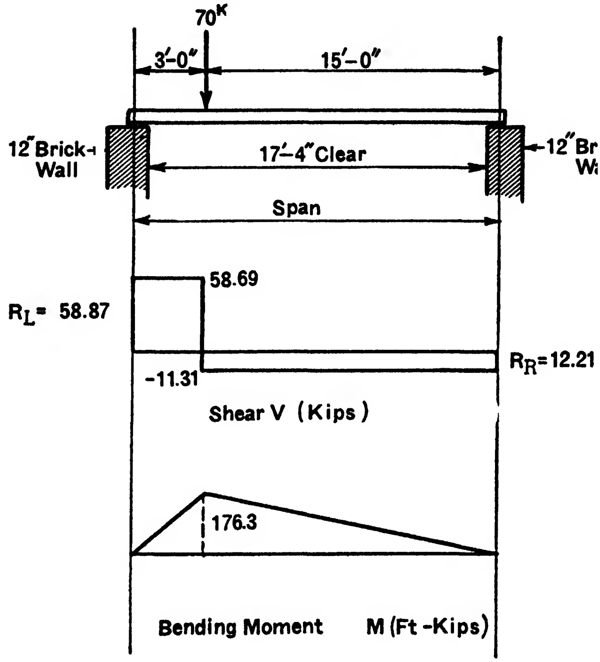


FIG. 4-31.

anticipated. For the case computed, the AREA method indicates stresses within 4 per cent of accepted values; for other cases this percentage variation changes.

The use of the AISC method should be limited to cases governed by the AISC specification, as defined in its title and preface. Method 2 is not recommended.

Further applications of these specifications are made in examples of plate girder calculations in Chap. 6.

4-29. Effect of Holes in Beam Webs. Holes in beam webs have less effect on flexural strength than holes in the flanges. However, if they are far from the neutral axis, or if the web is unusually thick, their effect on the moment of inertia and flexural stress should be considered.

If ⅞-in. rivets are spaced 4 in. center-to-center on a vertical line in the web, it is expected that the average shear stress on a line through the rivet holes is $4/(4 - 1) = 4/3$ times the average shear on the gross section. The AISC and AREA specifications provide for limiting design stresses for shear on the gross section, thus making it unnecessary to compute the shear on the net section.

To prevent buckling, reinforcing plates are sometimes welded or riveted to the web around large-size clearance or access holes. In lighter material a flange may be formed by bending the material at the edge of the hole. The flange stiffens the web and raises the value of the compressive stress required to buckle the material. An example of the latter detail is the flanged lightening hole frequently found in aircraft members.

EXAMPLE 4-23. As an illustration of a complete beam design, select a wide-flange steel section for the conditions shown in Fig. 4-31. Use the AISC specification. Lateral support of the compression flange is provided at the ends and at the point of application of the concentrated load.

In the usual case of beam design, an experienced structural engineer may correctly recognize by inspection the fact that certain types of failure do not control, and he may thus eliminate steps from the design procedure. Such reduction of work is very desirable, but it should result from experience. Since the purpose of this example is to illustrate consideration of each of the types of failure, no attempt is made to shorten the amount of work, even though some of the answers obtained may be obvious.

The effective span may be taken as the distance from center-to-center of wall bearing plates. Assuming that the center of the bearing plates will be 4 in. from the inside wall lines, the span length is 18 ft.

To compute the shear and bending moment, the weight of the beam is assumed to be 60 lb per ft. Thus,

$$R_L = (9 \times 0.06) + \frac{70 \times 15}{18} = 58.87 \text{ kips},$$

$$R_R = 12.21 \text{ kips}.$$

The bending moment M is maximum where the shear is zero. The shear diagram of Fig. 4-31 shows this to be under the concentrated load. The maximum value of M is equal to the area of the shear diagram from the left end to the concentrated load. Then

$$M \text{ (max)} = \frac{58.87 + 58.69}{2} \times 3 = 176.3 \text{ ft-kips}.$$

Assuming that the allowable flexural stress will be about 19,000 psi, the approximate required section modulus will be $(12 \times 176.3)/19 = 111.4$ in.^{3}

Try a 21 WF 62, for which S is 126.4; the flange width b, 8.24 in.; the flange thickness t, 0.615 in.; and the depth d, 20.99 in. The allowable compressive flexural stress is

$$F_b = \frac{12{,}000{,}000 \times 8.24 \times 0.615}{12 \times 15 \times 20.99} = 16{,}100 \text{ psi}.$$

The computed actual stress is

$$f_b = \frac{12 \times 176{,}300}{126.4} = 16{,}750 \text{ psi}.$$

The first trial section, 21 WF 62, is overstressed and is not satisfactory. For the second trial a 21 WF 68 will be checked. For this section, we also compute stress as shown below.

$$F_b = \frac{12{,}000{,}000 \times 8.27 \times 0.685}{12 \times 15 \times 21.13} = 17{,}900 \text{ psi},$$

$$f_b = \frac{12 \times 176{,}300}{139.9} = 15{,}100 \text{ psi}.$$

The 21 WF 68 is the lightest wide-flange section that is satisfactory in flexure. The section will now be checked for shear and crippling of the web.

Check for shear:

$$V = 58.87 \text{ kips, max},$$

$$v = \frac{V}{dt} = \frac{58{,}870}{21.13 \times 0.43} = 6{,}480 \text{ psi},$$

which is less than the allowable stress of 13,000 psi.

Check for diagonal buckling:

The ratio of clear depth of web to thickness of web equals 19.76/0.43 = 46.0. According to the AISC specification, diagonal buckling need not be considered imminent if this ratio is less than 70.

Check for web crippling:

At the left end the required bearing length b equals

$$\frac{R}{f_c t} - k = \frac{58{,}870}{24{,}000 \times 0.43} - 1.25 = 4.45 \text{ in.}$$

At the right end the required length b by similar computation is zero. Use a 6-in. length of bearing at each end. This size of bearing plate will permit a 1-in. setback from the face of the brickwork. The assumed span of 18 ft is correct. The bearing strength of the wall material on which the beam rests may necessitate a longer length b. (See Chap. 8.)

At the concentrated load, the required bearing length is

$$b = \frac{70{,}000}{24{,}000 \times 0.43} - (2 \times 1.25) = 4.28 \text{ in.}$$

The bearing length to be used at the concentrated load will depend on the detail of the member that transmits the load to the beam, but in any case the load must be distributed, by a bearing plate or by other means, over a length of not less than 4.28 in.

If the required length of bearing at either end or at the concentrated load were not available because of limited space, it would be necessary to increase the web thickness or to provide bearing stiffeners.

The determination of the thickness and width of bearing plates and the design of other connections would follow the design just completed. Also, if holes occurred in locations where they would affect the strength of the section selected, consideration of their effect would be necessary.

4-30. Bi-axial Bending. Figure 4-32(a) shows a beam with the applied load inclined to the axes of symmetry. Provided the loads and reactions

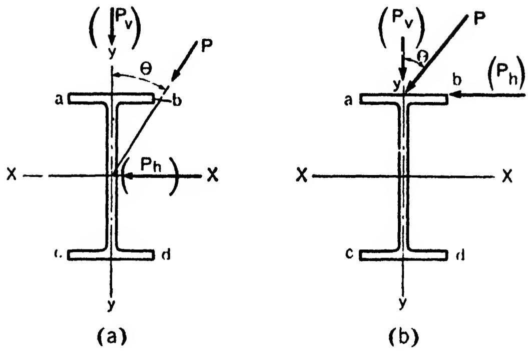


FIG. 4-32.

are on a line through the center of the section, and provided the section has two axes of symmetry, the analysis which follows may be used.

Substitute for the force P its two components, P_h and P_v, which act at the center of gravity of the section. The stresses due to the force P may now be determined by computing and adding algebraically the stresses due to P_h and the stresses due to P_v.

The bending moment due to the vertical force P_v is called M_v; that due to the horizontal force P_h is called M_h.

The extreme fiber stress due to P_v is M_v/S_x, compressive at points a and b and tensile at points c and d.

The extreme fiber stress due to P_h is M_h/S_y, compressive at b and d and tensile at a and c.

Using a positive sign for tension and negative for compression, the combined stresses are:

$$\text{At point } a, \ -\frac{M_v}{S_x} + \frac{M_h}{S_y}$$

$$\text{At point } b, \ -\frac{M_v}{S_x} - \frac{M_h}{S_y}$$

$$\text{At point } c, \ +\frac{M_v}{S_x} + \frac{M_h}{S_y}$$

$$\text{At point } d, \ +\frac{M_v}{S_x} - \frac{M_h}{S_y}$$

The maximum compressive stress occurs at point b, and the maximum tensile stress at point c.

To determine the safety of the section, these maximum stresses are compared with the allowable stresses or with the ultimate stresses.

The compressive stress should be compared with whichever is the least: the yield strength, the stress that would cause local buckling, or the stress which would cause lateral buckling of the entire section. It is generally considered satisfactory to use for this last stress the same value that would be critical for the member acting under load P_v only. In structural steel, local buckling need not be considered if the section conforms to the limiting ratios of outstanding width to thickness.

EXAMPLE 4–24. Assume that the section of Fig. 4–32(a) is a 10 WF 49, that the load P is 7 kips applied at the center of the span, the angle Θ is 30°, the span 16 ft, and that lateral support is provided at the ends only. Does the section satisfy the flexural requirements of the AREA specification?

The flexural stress permitted by the AREA specification is

$$F_b = 18{,}000 - 5\left(\frac{L}{b}\right)^2 = 18{,}000 - 5\left(\frac{12 \times 16}{10}\right)^2 = 16{,}160 \text{ psi.}$$

Resolving P into components parallel to the x–x and y–y axes,

$$P_h = 7 \sin 30° = 3.50 \text{ kips,}$$

$$P_v = 7 \cos 30° = 6.07 \text{ kips,}$$

$$M_h = \frac{P_hL}{4} = \frac{3.50 \times 16}{4} = 14.0 \text{ ft-kips,}$$

$$M_v = \frac{P_vL}{4} + \frac{wL^2}{8} = \frac{6.07 \times 16}{4} + \frac{0.049 \times 16^2}{8} = 25.8 \text{ ft-kips.}$$

The maximum compressive stress is

$$\frac{M_h}{S_y} + \frac{M_v}{S_x} = \frac{12 \times 14{,}000}{18.6} + \frac{12 \times 25{,}800}{54.6} = 14{,}700 \text{ psi.}$$

This is less than the allowable based on lateral buckling of the entire section. The ratio of outstanding flange width to thickness is only 4.83/0.558, or 8.66, and so local buckling does not control. The section is, therefore, satisfactory.

Often the inclined load is located as shown in Fig. 4–32(b). In this case the horizontal component P_h causes a twist or torsion of the section. However, if the end connections are detailed so as to provide for horizontal reactions in the plane of the horizontal loads (i.e., at top flange level), it is possible to make a simplifying assumption. That assumption is that the bending moment M_h caused by the horizontal load P_h is resisted by the top flange alone. Thus, point b would have the highest compressive stress, while point a and either c or d must be checked for tensile stress.

An example of the analysis of a crane girder loaded as in Fig. 4–32(b) will be shown by Ex. 4–28. A similar case is the example design of a roof purlin in Chap. 7.

4–31. Combined Bending and Direct Load. A common design problem is that of members having simultaneous bending moment and axial load. Such members function primarily as either tension or compression members, but they also have bending moment resulting from the rigidity of adjoining members, eccentric application of the tensile or compressive loads, initial crookedness, deflection, or transverse loads.

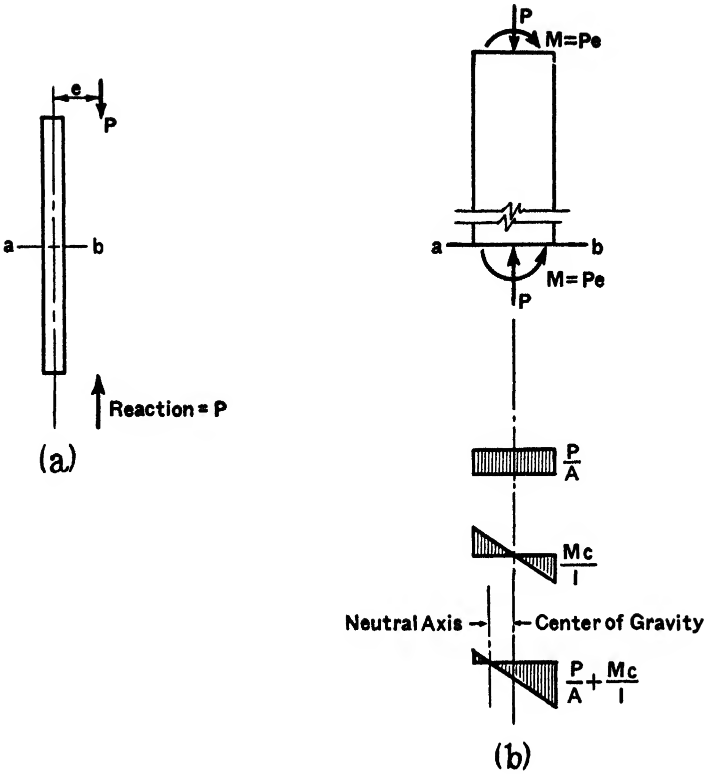


FIG. 4–33.

Consider a section through a member shown in Fig. 4–33(a). The eccentric load P can be replaced by an axial load of the same amount and a bending moment P_e, as shown in Fig. 4–33(b). At any section, such as at a–b, equilibrium of the section is maintained by an internal force and an internal moment equal to, but in opposite direction to, the external force and moment, respectively. At any point on the section, the total stress is the sum of the stresses caused by axial load and by bending moment.

At point a,

$$f = \frac{P}{A} - \frac{Mc}{I},$$

and at point b,

$$f = \frac{P}{A} + \frac{Mc}{I};$$

or for any point at that section,

$$f = \frac{P}{A} \pm \frac{My}{I}.$$

The conditions would prevail provided neither the yield strength nor the critical buckling stress had been reached.

For a member in which the allowable bending stress is equal to the allowable axial load stress, the choice of a safe section consists of finding one for which the maximum combined stress $P/A \pm Mc/I$ does not exceed the allowable. Because of the number of variables, direct choice of section is not feasible, except for sections of regular shape, such as round, square, hexagonal, etc. For the usual case, successive trials must be made to determine the most economical safe section.

If the allowable stress in bending is not equal to that for axial load, to what value should the total or combined stress be limited? It is certainly conservative to require that the combined stress be no larger than the smaller of the allowable stresses in bending or axial load. Some specifications do require that the combined stress be so limited.

A more popular method is one in which the combined stress is a compromise between the bending and axial allowables. By this method, if the majority of the combined stress is due to axial load, the allowable combined stress lies between the bending and axial load allowables, but closer to that for axial load; if bending causes the larger part of the combined stress, the allowable value lies closer to that for bending.

The AISC specification for combined stresses due to axial and bending loads provides that

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

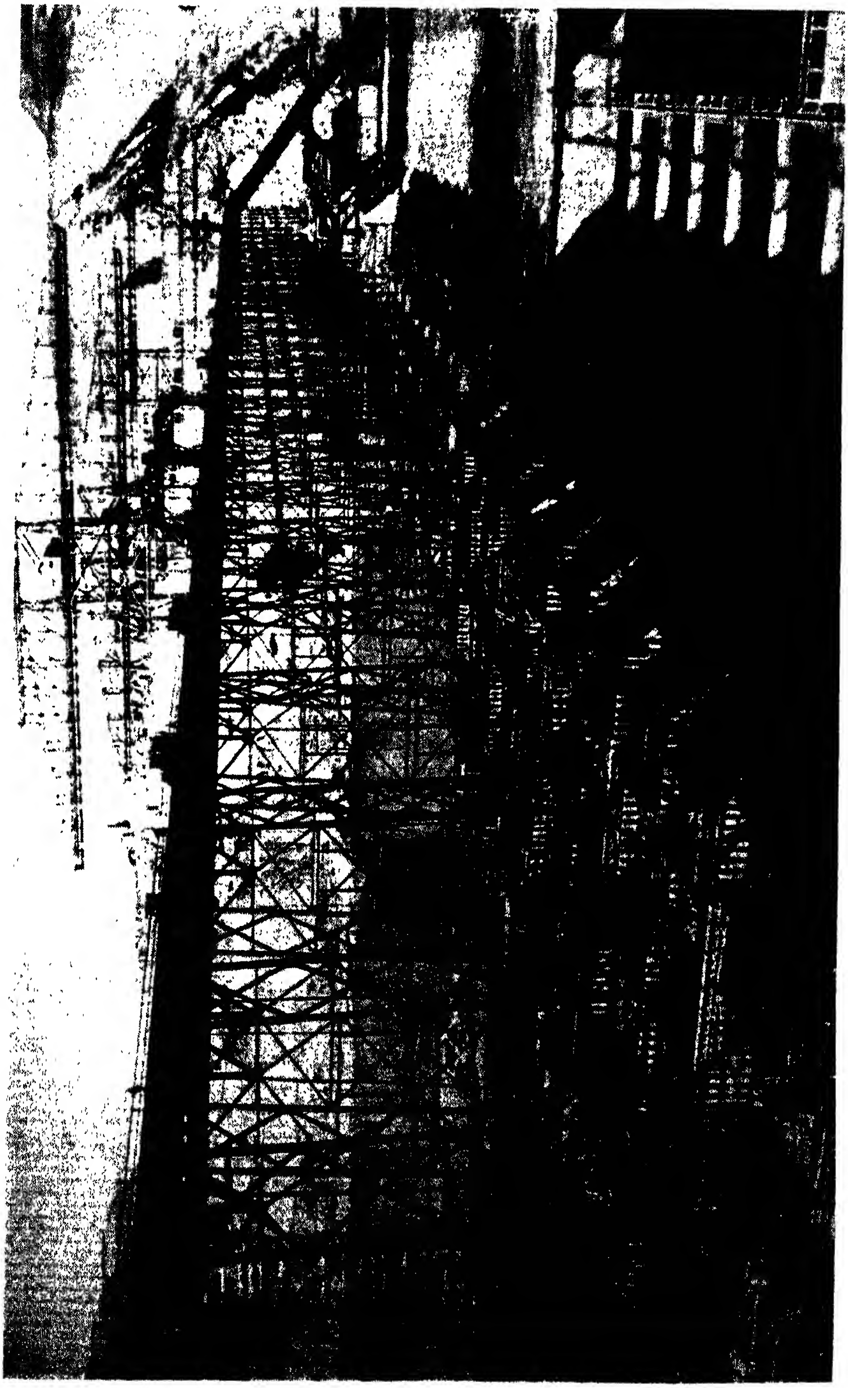
F_a = axial unit stress that would be permitted by this Specification if axial stress only existed,

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

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

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

This method of determining the limit of stress is based on the rationalization that, if a certain percentage of the value of the member has been used for axial load, the remaining percentage may be used for bending moment. It will be noted that if $f_b = 0$, $f_a = F_a$; if $f_a = 0$, $f_b = F_b$; if $f_a = \frac{1}{3}$ of F_a, f_b may be $\frac{2}{3}$ of F_b; etc. Since both F_a and F_b are based on empirical formulas, no derivation of the relationship seems possible. Indeed, there seems little reason to suspect that the yield point is not the limit of useful stress and that the Euler limit is not the limit of axial load, if the Euler load produces unit stress less than the yield-point stress. To be entirely consistent, an approximate factor of safety should be maintained with respect to the Euler load, and the combined unit stress $f_a + f_b$



Bethlehem Steel Co.

GRAND COULEE DAM, WASHINGTON.

should have a factor of safety with respect to the yield point. The development of the Euler load for the flange involves a loading which comes from the axial load as well as from the increments added along the length of the member due to bending; this development is outside of the scope of this text.

EXAMPLE 4–25. A structural steel bar, 1 in. wide, 4 in. deep, and 20 ft long, lies in a horizontal position with the 4-in. dimension vertical. The ends of the bar are connected by ¾-in. rivets in such a manner that no end moments occur. An axial load of 62.5 kips tension is applied. If the allowable stresses in bending and in axial load are each 20,000 psi, is the member satisfactory?

The member in this example must be investigated at two sections—the end, at which there is axial load on a section reduced by holes; and the center, where both axial load and bending moment occur, but where the entire section is effective.

At the ends,

$$A \text{ (net)} = (4 \times 1) - (\tfrac{7}{8} \times 1) = 3.125 \text{ in.}^2,$$

and

$$f = \frac{P}{A} = \frac{62{,}500}{3.125} = 20{,}000 \text{ psi.}$$

Since f does not exceed 20,000 psi, the member is satisfactory at this point.

At the center, the bending moment due to dead load equals $wL^2/8$. The weight per linear foot equals $4 \times 3.4 = 13.6$ lb; and

$$M = \frac{13.6 \times 20 \times 20}{8} = 680 \text{ ft-lb},$$

$$I = \frac{1 \times (4)^3}{12} = \frac{16}{3} \text{ in.}^4,$$

$$f_a = \frac{P}{A} = \frac{62{,}500}{4} = 15{,}625 \text{ psi},$$

$$f_b = \frac{Mc}{I} = 8{,}160 \times 2 \times \frac{3}{16} = 3{,}060 \text{ psi},$$

$$f = 15{,}625 + 3{,}060 = 18{,}685 \text{ psi.}$$

Since the combined f does not exceed the allowable stress of 20,000 psi, the member is safe at the midsection. It should be noticed that the critical section is at the end where the section is reduced. This is so often true that, for short tension members, analysis for the effect of dead load may usually be omitted. It should be noted by the student that the tension in the member reduces the moment in the bar. The amount of reduction is $P\Delta$, where Δ is the deflection at any point. Since the critical section is at the end, no further analysis is required.

This example deals with a member having its ends simply supported. However, end conditions are sometimes continuous, and so the designer may need to compute end moments as well as the moment at midspan. Analysis at the end section then involves not only axial load but also bending on the net section.

EXAMPLE 4–26. Select a W section for a building column to resist a vertical load of 105 kips and a maximum bending moment of 630 in.-kips about the x–x

axis. The length of the column is 24 ft, and lateral support is provided only at the ends. Use the AISC specification.

Although selection of this section is a trial-and-error process, it is often possible to estimate closely the size required so that a final choice may be made after only one or two trials. To aid in making an estimate on which to start computations, assume that the member will act at 50 per cent of its capacity in bending and at 50 per cent of its capacity in axial load. For a member with a 24-ft unsupported length, both F_a and F_b will probably be reduced to prevent buckling. If F_b is 17,000 and F_a is 14,000 (still assumed values), the section would require

$$A = \frac{P}{0.5F_a} = \frac{105{,}000}{0.5 \times 14{,}000} = 15.0 \text{ in.}^2;$$

$$S = \frac{M}{0.5F_b} = \frac{630{,}000}{0.5 \times 17{,}000} = 74.1 \text{ in.}^3$$

In order not to exceed a value of 120 for the ratio L/r, the section should have a minimum radius of gyration of not less than $(24 \times 12)/120 = 2.4$ in.

It should be emphasized that the preceding figures are just an aid in estimating the original trial section. They are not a part of the necessary design calculations. If the designer prefers, he may use any other type of aid or may even guess; and if his guess is based on experience and good judgment, it should yield as good a starting point as any other method. If the first trial section is far from the correct answer, no harm is done except that one or two extra trials may be needed in order to arrive at the final section.

Using the estimated requirements for r, A, and S as a guide, try a 12 WF 53. For this section the minimum value of $r = 2.48$ in., $A = 15.59$ in.^{2}, $S = 70.7$ in.^{3}, $b = 10.0$ in., $t = 0.576$ in., and $d = 12.06$ in. Therefore,

$$f_a = \frac{105{,}000}{15.59} = 6{,}740 \text{ psi},$$

$$f_b = \frac{105{,}000 \times 6}{70.7} = 8{,}910 \text{ psi},$$

$$\frac{L}{r} = \frac{12 \times 24}{2.48} = 116,$$

$$F_a = 17{,}000 - 0.485(116)^2 = 10{,}470 \text{ psi},$$

$$F_b = \frac{12{,}000{,}000 \times 10.0 \times 0.576}{12 \times 24 \times 12.06} = 19{,}900 \text{ psi}.$$

Then

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{6{,}740}{10{,}470} + \frac{8{,}910}{19{,}900} = 1.092,$$

which is greater than unity; hence, the 12 WF 53 does not satisfy the specification.

Since the unit stress in the first trial section was approximately 10 per cent too high, a second trial section with approximately 10 per cent greater area will be chosen.

Try a 12 WF 58, for which the minimum $r = 2.51$ in., $A = 17.06$ in.^{2}, $S = 78.1$ in.^{3}, $b = 10.01$ in., $t = 0.641$ in., and $d = 12.19$ in. Repeating the computations,

$$\frac{L}{r} = 115,$$

$$f_a = 6{,}150 \text{ psi},$$
$$f_b = 8{,}060 \text{ psi},$$
$$F_a = 10{,}590 \text{ psi},$$
$$F_b = 20{,}000 \text{ psi}.$$

Then

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = 0.581 + 0.403 = 0.984.$$

The calculation shows that the 12 WF 58 is satisfactory. Inspection of the list of available WF sections in the handbook shows no lighter satisfactory WF section. Then, provided sufficient space exists to permit its use, a 12 WF 58 will be used.

If a space or architectural requirement indicates a depth of section other than 12 in., a suitable section can likely be provided, but it will be heavier and less economical than the 12 WF 58.

4–32. Effect of Deflection. Transverse deflection of members subject to axial load and bending moment causes changes of stress. The effect of deflection is sometimes to increase stress and sometimes to decrease it. The amount of the effect may be appreciable or it may be negligible.

As a purely qualitative example, consider the members shown in Fig. 4–34. In (a) is shown a member having a tensile load P_1 and a transverse

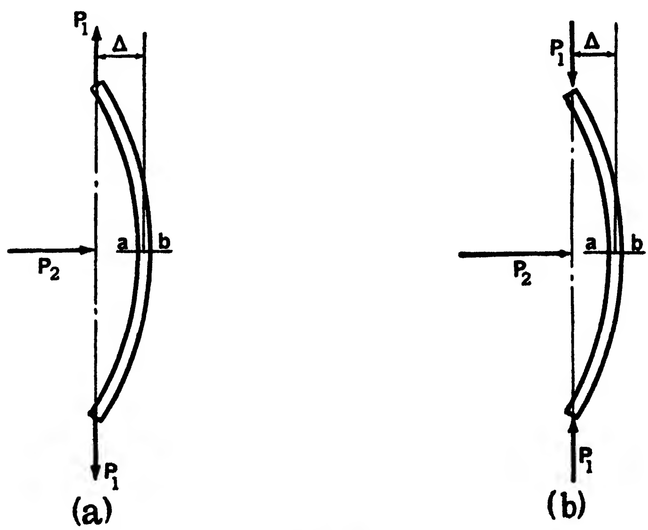


FIG. 4–34.

load P_2. Without deflection, the section at a–b must resist the direct tension P_1 and a bending moment M caused by the transverse load. At point a the total stress is

$$f = \frac{P_1}{A} - \frac{Mc}{I};$$

at b,

$$f = \frac{P_1}{A} + \frac{Mc}{I}.$$

If a deflection occurs because of the load P_2, an additional bending moment results from the eccentricity of the load P_1 with respect to the center of gravity of section a–b. This moment is equal to $P_1\Delta$ but it is of opposite sign to the bending moment caused by P_2. The resultant stresses at point a are now

$$f = \frac{P_1}{A} - \frac{Mc}{I} + \frac{P_1\Delta c}{I};$$

and at point b,

$$f = \frac{P_1}{A} + \frac{Mc}{I} - \frac{P_1\Delta c}{I}.$$

Deflection, in this case, has caused a decrease in the maximum stress (point b) and an increase in the minimum stress (point a).

By similar analysis, deflection of the compression member with transverse load in Fig. 4–34(b) causes an increase of the maximum stress (point a) and a decrease of the minimum (point b).

Thus for symmetrical sections, the effect of deflection on tension members may usually be neglected, since it seldom causes sufficient change of stress to permit reduction of the section. For compression members, deflection must occasionally be considered, because its effect is to increase the maximum stress.

If it is desired to know the effect of deflection on stress, it may be obtained by the following procedure:

1. Compute Δ, the deflection caused by the transverse load (P_2) or by the applied bending moment.
2. Compute the bending moment ($P\Delta$) and the amount of the stress $[(P\Delta c)/I]$ caused by this moment at the extreme fibers.
3. Add, algebraically, the stress caused by the bending moment $P\Delta$ to the stresses caused by the applied loads.

The change $P\Delta$ of bending moment causes a further change in deflection. However, except for very slender columns, the first determination of the amount of the deflection will be sufficiently accurate. For a complete calculation of the amount of the deflection, integrate the expression

$$EI\frac{d^2y}{dx^2} = M,$$

which may be accomplished by computation of successive increments of deflection.*

* C. D. Williams, *Analysis of Statically Indeterminate Structures* (Scranton, Pa.: International Textbook Co., 3d Edition, 1952), Chap. 9.

Built-up Members

4–33. Built-up Sections. Built-up sections are those in which the flanges, webs, and other elements are each formed separately and then assembled by riveting, bolting, or welding to form the desired section. The three main reasons for using built-up sections are as follows:

1. A single section having the necessary dimensions and properties may not be available.
2. A built-up section may be less expensive than a suitable single section.
3. The weight of a built-up section may be less than that of a single section. For aircraft structures and some bridges, this may be sufficient reason to use a built-up section even though the cost may be increased.

Built-up sections may consist of one material only; or they may include a combination of metals, such as steel and an aluminum alloy, in the same member. Protection against galvanic action may be required in such cases.

A common example of a built-up section is the plate girder, which is considered in Chap. 6. Other examples, some of which are discussed in this chapter, are reinforced beams, built-up columns, truss members, stiffened sheet panels, and cellular sections.

4–34. Reinforced Beams. A common built-up flexural member is the reinforced beam, examples of which are shown in Fig. 4–35.

The usual reason for reinforcing a beam is to obtain greater strength through an increased section modulus or an increased allowable stress for the compression flange or both. A second reason is to provide increased resistance to lateral loads. A third reason is to provide detail shape and dimensions to suit the function of the member.

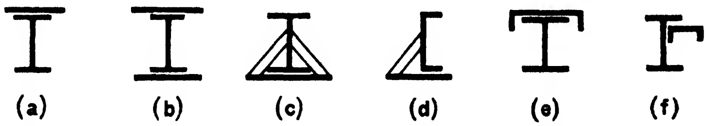


FIG. 4–35.

Reinforced beams are commonly used for lintels (beams supporting masonry walls over opening), spandrel beams (exterior beams supporting walls), crane-bridge beams, crane-runway beams, monorail beams, and many other types.

The design of reinforced sections that are symmetrical about the plane of the loads is relatively simple. If such symmetry does not occur, then torsional stresses are caused. In some cases the simplifying assumption of Art. 4–30 and Ex. 4–28 is used. In other cases, the torsion may be great and should be considered. (The analysis of torsion is complex and is not covered in this text.)

1. Select a trial section consisting of a beam and its reinforcement. Except for sections that are symmetrical about both axes, such as that of Fig. 4–35(*b*), a direct solution is not possible. The first selection must be an estimate.

2. Locate the gravity axis of the combined section and compute the moment of inertia about that axis.

3. Compute the maximum tensile and compressive unit stresses for the combined section and compare them with the stresses permitted by the specification.

4. Repeat steps (1), (2), and (3) as required to determine a satisfactory section.

5. If it is desired to use reinforcement only at the points where the section modulus of the unreinforced beam is insufficient, determine the points of cut-off for the reinforcement. (The same procedure is used as for the cover plates of plate girders. See Chap. 6.) Since the flange dimensions affect the allowable unit compressive flexural stress, a beam with reinforcement extending over less than the entire span might have a different allowable stress than one with full-length reinforcement. Specifications do not provide for the determination of the allowable stress for such members. A value of the allowable compressive stress for such a beam may be approximated by a procedure recommended by C. D. Williams* for determining the Euler limit of variable-section beams. Otherwise, an approximate value of the allowable compressive stress can be obtained by using a ratio of L/b (or Ld/bt) intermediate between that of the unreinforced beam and one with full-length reinforcement. The effective ratio used for solving problems such as this should be a weighted average value which considers the relative lengths of reinforced and unreinforced sections.

6. Check for resistance to shear failure, web crippling, and web buckling, as required.

7. Determine the longitudinal shear at the connection of the reinforcement to the beam and design the welded, riveted, or bolted connections of the reinforcement to the beam.

Example 4–27. What is the maximum bending moment that can be applied to the section shown in Fig. 4–36 without exceeding the flexural stresses allowed by the AREA specification?

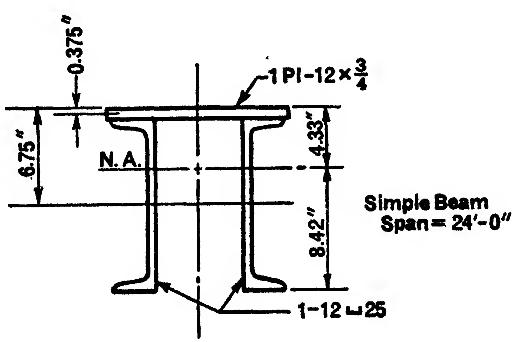
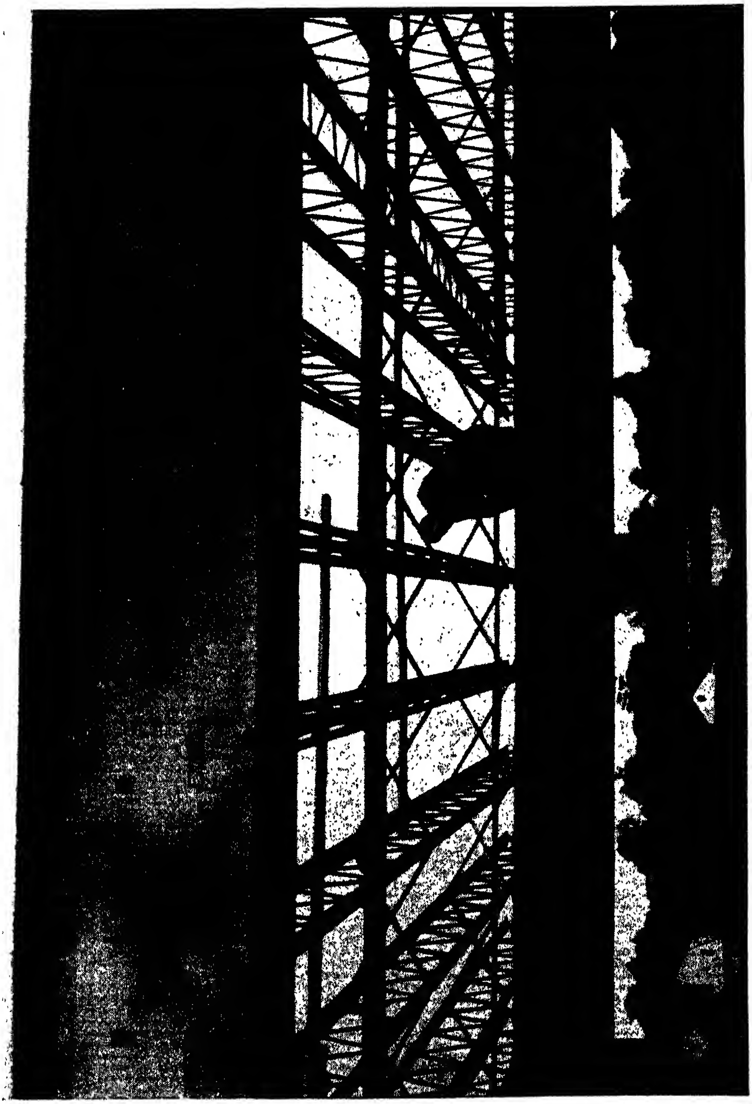


Fig. 4–36.

* See Williams, *op. cit.*, Chap. 9.



Bethlehem Steel Co.

ERECTION OF STEEL JOIST BEAMS

The properties of the section are computed as follows:

Section	A	y (from Top)	Ay	Ay^2	I_{0-0}
1—12 × ¾ Pl.	9.00	0.375	3.4	1.3	0.0
2—12 [25.	14.64	6.75	98.8	667.0	287.0
Total.	23.64		102.2	668.3	287.0

Then

$$\bar{y} = \frac{102.2}{23.6} = 4.33 \text{ in.},$$

$$I_{na} = 668.3 + 287.0 - (102.2 \times 4.33) = 512.8 \text{ in.}^4,$$

$$\frac{L}{b} \text{ for the compression flange} = \frac{24 \times 12}{12} = 24,$$

$$F_b \text{ (allowable flexural stress)} = 18{,}000 - 5\left(\frac{L}{b}\right)^2 = 15{,}120 \text{ psi compression, or } 18{,}000 \text{ psi tension},$$

$$M \text{ (maximum allowable bending moment)} = \frac{F_b I}{c}.$$

Based on 15,120 psi compression,

$$M = \frac{15.12 \times 512.8}{4.33 \times 12} = 149 \text{ ft-kips}.$$

Based on 18,000 psi tension,

$$M = \frac{18 \times 512.8}{8.42 \times 12} = 91.5 \text{ ft-kips}.$$

The maximum allowable bending moment is 91.5 ft-kips. The fact that such a great reserve of strength exists in the compression flange of this section might in some cases indicate to the designer that a more economical design is possible. If functional requirements did not limit the section to the one just computed, further trials might be made with other sizes of material.

Example 4–28. Figure 4–37(a) shows a typical overhead crane with its trolley in the extreme right end position so as to cause the maximum load on the right runway. The maximum reaction of the crane on one runway girder is 18 kips, distributed between the two wheels of the crane end-truck. The wheel loads are each increased by 25 per cent for impact, in accordance with the AISC specification. Each wheel load is then 11.25 kips. The wheels are fixed at 4 ft center-to-center. The pair of wheels is located on Fig. 4–37(b) so as to produce the maximum bending moment. In accordance with the specification, a lateral load is applied to each runway, at the top of the rail, equal to 10 per cent of the sum of the lifted load plus the weights of the hoist and the trolley. This lateral load amounts to 0.80 kip at each wheel. Using the AISC allowable stresses, is the section satisfactory?

The analysis of the stresses caused by the vertical loads is similar to that of Ex. 4–26. Exact analysis of the effect of the lateral load is more difficult for two reasons: the horizontal load is applied eccentrically, and the section has no horizontal axis of symmetry. It is considered by many engineers to be sufficiently accurate (if the end connections provide for horizontal reactions at the top-flange level) to assume that the lateral load acts at the top-flange level and that it is resisted entirely by the top flange. For shallow girders, however, or for those in which the lateral loads are a larger percentage of the vertical loads, analysis by more exact methods is desirable. In this example the simplifying assumption will be used:

$$M \text{ (vertical)} = \frac{2 \times 11.25 \times 9 \times 9}{20} + \frac{0.051 \times 9 \times 11}{2} = 93.6 \text{ ft-kips},$$

$$M \text{ (horizontal)} = \frac{2 \times 0.80 \times 9 \times 9}{20} = 6.48 \text{ ft-kips}.$$

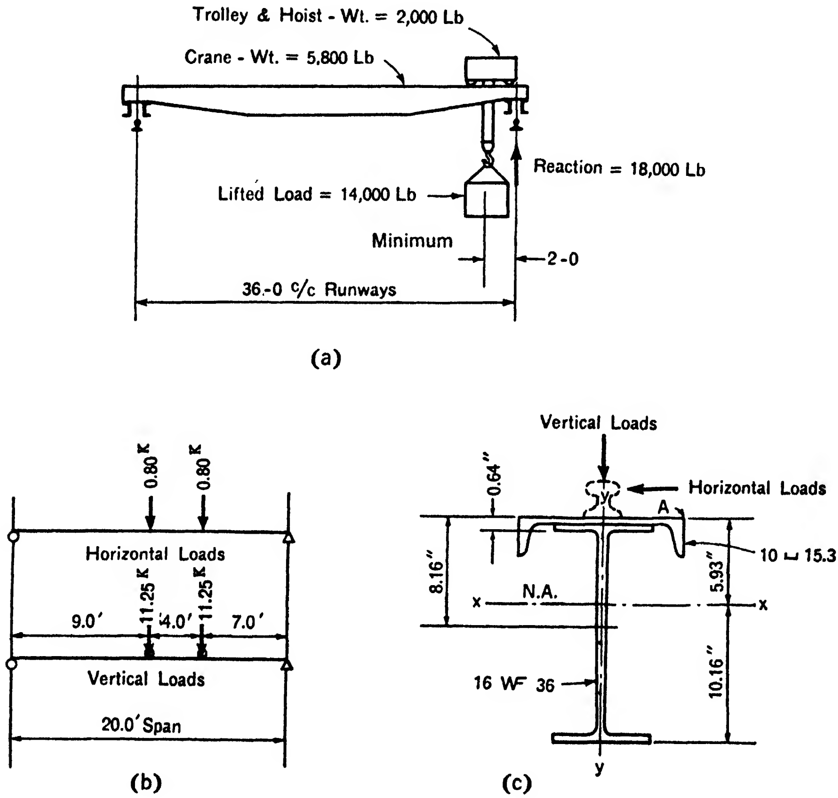


FIG. 4–37.

The section properties are:

Section	A	y (from Top)	Ay	Ay^2	I_{0-0}
16 WF 36	10.59	8.16	86.4	705.0	446.3
10 ⊔ 15.3	4.47	0.64	2.9	1.8	2.3
Total	15.06		89.3	706.8	448.6

Then

$$\bar{y} = \frac{89.3}{15.06} = 5.93 \text{ in.},$$

$$I_x = 706.8 + 448.6 - (89.3 \times 5.93) = 625.4 \text{ in.}^4$$

Using the channel and top flange only,

$$I_y = 66.9 + \left(\frac{0.428 \times \overline{6.99}^3}{12}\right) = 79.1 \text{ in.}^4$$

The AISC specification does not state how the lateral buckling of a section such as this should be considered. The AISC formula was designed for use on I-shaped, unreinforced sections. In it, the product bt is the area of the compressive flange. Karl de Vries,* in his paper which led to the adoption of the AISC formula, recommends that for a plate girder flange consisting of angles and cover plates riveted together, a value of three-fourths of the combined area of the cover plates and the outstanding legs of the angles of the compression flange should be substituted for the term bt. Applying the recommendation to this example, bt is taken to be equal to three-fourths of the combined area of the top flange and the reinforcing channel. Then

$$A \text{ (for the channel and top flange)} = 7.47 \text{ in.}^2,$$

$$bt \text{ (effective)} = \tfrac{3}{4} \times 7.47 = 5.6,$$

$$F_b = \frac{12{,}000{,}000}{Ld/bt} = \frac{12{,}000{,}000 \times 5.6}{12 \times 20 \times 16.09} = 17{,}400 \text{ psi compression},$$

or 20,000 psi tension.

Computing the actual stresses,

$$f_b \text{ (bottom flange)} = \frac{93.6 \times 12{,}000 \times 10.16}{625.4} = 18{,}300 \text{ psi},$$

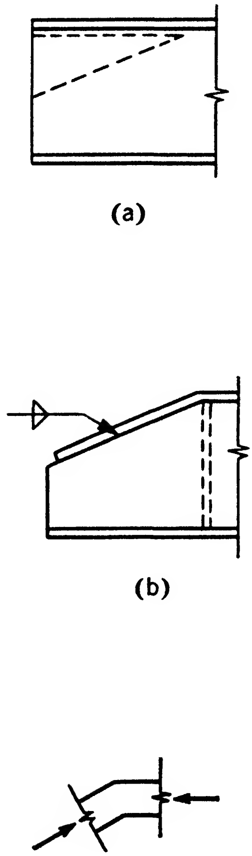
$$f_b \text{ (point } A \text{ of top flange)} = \frac{93.6 \times 12{,}000 \times 5.93}{625.4} + \frac{6.48 \times 12{,}000 \times 5}{79.1} = 15{,}600.$$

Flexurally, the section satisfies the AISC requirements.

The shear value of the web of the 16 WF 36 is 13,000 × 15.85 × 0.299, or 61,600 lb. The maximum possible reaction is obviously not this great; therefore, the section is satisfactory.

4–35. Beams of Variable Depth. Beams of variable depth are used for improving appearance, satisfying limited space requirements, controlling the stiffness, reducing the cost, or minimizing the weight of the member. Common examples of variable-depth beams are crane-bridge beams and the longitudinal girders of railroad flat cars, which are both deeper at the center than at the ends. Another example is the wing spar of an airplane which tapers from a small size at the wing tip to a maximum depth near the fuselage.

* Karl de Vries, "Strength of Beams as Determined by Lateral Buckling," *Proc. ASCE*, Vol. 72 (September 1946), No. 7, p. 985.



(c)

FIG. 4–38.

A variable-depth section is usually produced by connecting the flange elements to a separately manufactured web which has been cut to the desired shape. The flange material is bent to the outline desired and is connected to the web by rivets, bolts, or welds. Another method of producing the variable depth is illustrated in Fig. 4–38. The web is cut as indicated by the dotted lines on the original rolled section. After removal of the wedge-shaped piece of web material, the flange is bent to touch the remaining web and is welded as shown. If it is desired, the depth of the original section can be increased instead of decreased. This is done by cutting the web on one line only, then spreading the flanges farther apart, and welding an insert of material to the web.

It should be noted that the flange near the sharp bend is subject to unbalanced flexural stresses, as shown in Fig. 4–38(*c*). These stresses, if compressive, tend to bend the outer edges of the flange upward, away from the rest of the beam. If the stresses are tensile, the opposite distortion may occur. In either case, should the distortion occur, the stresses in the distorted part will be partially relieved, while those on the rest of the section increase to more than the computed amount. The distortion can be prevented by adding a welded stiffener which can provide the force needed as an equilibrant to the two stresses shown in Fig. 4–38(*c*). Such a stiffener is shown dotted in Fig. 4–38(*b*).

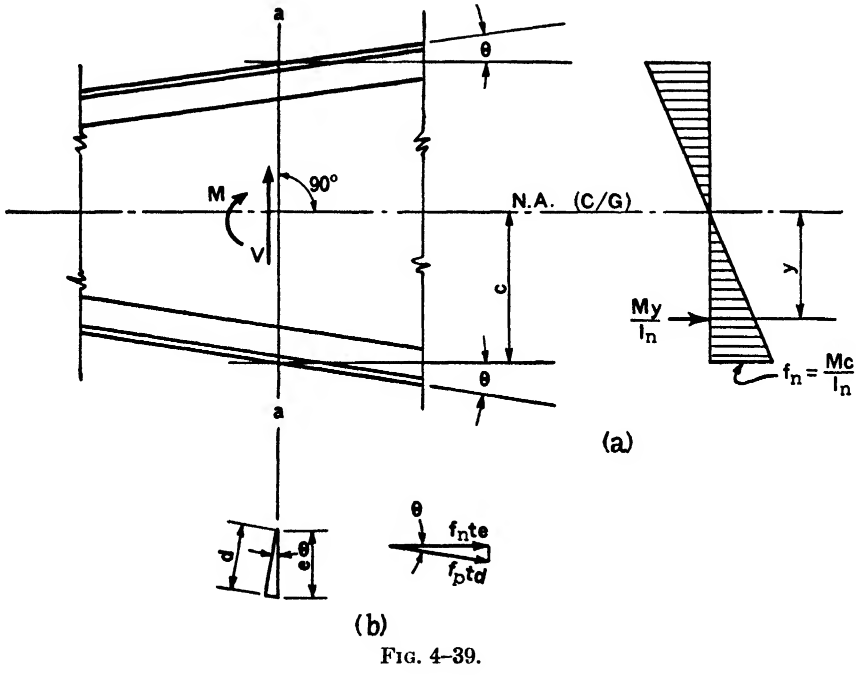


FIG. 4–39.

In Fig. 4–39(*a*) is shown a portion of a beam of variable depth. Both flanges are inclined at an angle θ to the neutral axis. To make an analysis of the maximum flexural and shearing stresses, consider the beam cut at section *a–a*. Section *a–a* is taken perpendicular to the neutral axis of the section. Projected to the right of the section is a diagram showing the flexural stresses f_n normal to section *a–a*. The moment of inertia I_n used in computing f_n is that of the section as measured along line *a–a*.

The maximum principal stress f_p at the extreme fiber is parallel to the edge of the material. The relationship between f_n and f_p is illustrated by part (*b*) of Fig. 4–39. A small element of flange material is shown having side lengths d and e and thickness t. The force normal to the vertical face of the element (parallel to the neutral axis of the beam) is $f_n te$. This force is a component of the force $f_p td$, which is parallel to the edge of the beam. Then

$$f_p td = f_n te \sec \theta,$$

$$e = d \sec \theta.$$

Substituting for e,

$$f_p = f_n \sec^2 \theta.$$

If the section has a concentration of material in the flanges, most of the term I_n is due to the flange areas. In view of this fact, the equation just developed can be simplified.

Let I be the moment of inertia computed, using the over-all depth measured along line a–a, but using the *right* cross-sectional properties (area and moment of inertia) of the flanges themselves rather than the properties of the section cut through the flange at line a–a.

The *right* flange area is equal to the flange area as measured parallel to a–a times $\cos \theta$. Therefore,

$$I_n = I \sec \theta \text{ (approximately)},$$

$$f_n = \frac{Mc}{I \sec \theta},$$

$$f_p = \frac{Mc}{I} \times \sec \theta.$$

Application of this equation is simple because, in computing I, use is made of the right cross-sectional areas of the flange elements.

The shear V is measured parallel to the line a–a. As shown in Fig. 4–40, the total shear may be considered to be resisted by the web shear and by the vertical components of the flexural stresses neglecting the small part of the shear taken by the flanges. Since the bulk of the flexural stress occurs in the flange, and since the direction of the flexural stress in the web is more nearly parallel to the neutral axis, the shear resisted by the web can be approximated as follows:

$$C = A_c \text{ (area of flange)} \times \frac{M\bar{y} \sec \theta}{I},$$

where $\bar{y}$ is the distance from the center of gravity of the flange to the neutral axis of the beam, and

$$V_c \text{ (shear resisted by the top flange)} = C \sin \theta = \frac{M\bar{y}}{I} \times A_c \tan \theta.$$

Similarly,

$$V_t = \frac{M\bar{y}}{I} \times A_t \tan \theta,$$

$$V_w \text{ (shear resisted by web)} = V - V_c - V_t.$$

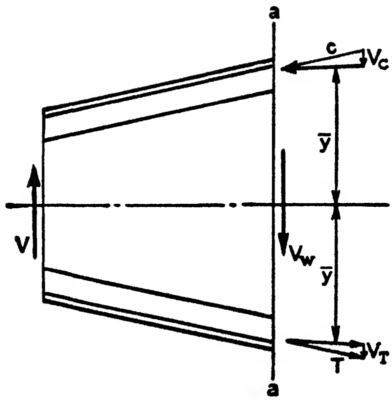


FIG. 4-40.

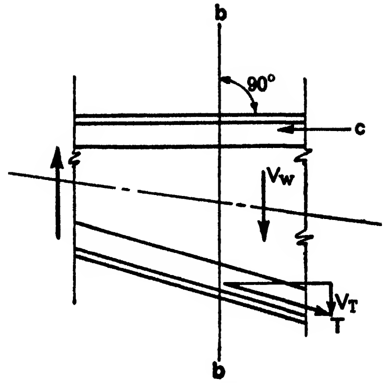


FIG. 4-41.

The values of flange stress, and consequently web shear, are affected by the action of direct load on the section at line a–a. If the resultant of the loads on one side of a–a is not parallel to a–a, its normal component is treated as a direct load and is divided uniformly over the entire cross-sectional area. The flange stress thus computed is normal to a–a and must be multiplied by $\sec^2 \theta$ to obtain the principal stress due to direct load.

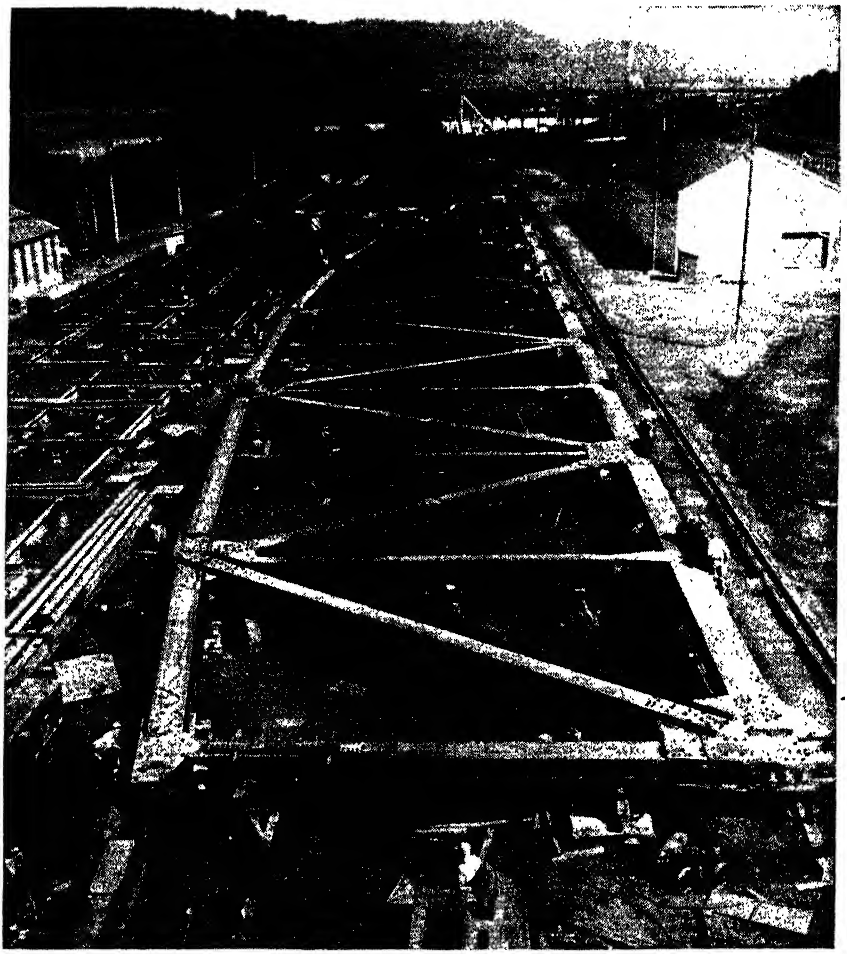
While it is always more accurate to compute the stresses on a section taken at right angles to the neutral axis for bending, it is often simpler to use a section taken perpendicular to one of the flanges. If the slope of the flange is slight, the results obtained are accurate enough for most types of structure.

A particular case for which this simplified procedure is useful is that of a beam having only one flange inclined, as illustrated in Fig. 4–41. For the section along line b–b, I is computed using the right sectional properties of each flange. The stress f_n is computed for each extreme fiber. For the horizontal flange, f_n is the actual fiber stress. For the inclined flange, f_p is approximately equal to $f_n \sec \theta$. The shear V_w resisted by the web is $V - V_t$, where V_t is approximately $(M\bar{y}/I) \times A_t \tan \theta$.

4–36. Built-up Compression Members. Compression members are built up from smaller rolled or extruded shapes for one of the following reasons:

1. To provide a larger cross-sectional area than is available with a single section.
2. To control the maximum slenderness (L/r ratio) of the member by providing a greater radius of gyration than is available in a single section.
3. To produce a member having approximately equal L/r ratios about the two principal axes.

Several examples of built-up compression members are shown in Fig. 4–8 and are discussed in Art. 4–10.



American Bridge Co.

Shop Assembly of One of Four 340-Foot Trusses for Vertical Lift Railroad Bridge, Harlem River, New York City.

4–37. Design of Built-up Compression Members. The selection of a built-up compression member is performed by successive trials in the same manner as for a single section. A little more work is involved, however, because for each trial section it is generally necessary to compute the properties (area and radii of gyration). Tables for many types of built-up sections are available, but such tables cannot cover the entire range and variety of possible sections. For larger structures (or for more costly ones) it may be necessary not only to determine the most economical section of one particular type but also to compare the chosen sections of several different types to determine the most desirable section.

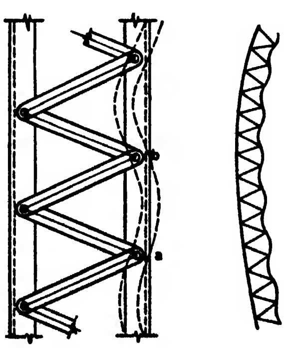
The allowable unit compressive stress P/A is limited by the L/r ratio of the entire member. If the L/r values about various axes are not equal, the largest ratio is used. The area and radius of gyration are based on the entire area in the usual case, although in some instances a portion of the area is dropped from the computation of both area and radius of gyration. (See Exs. 4–7 and 4–10.)

After the section has been selected, provisions must be made to join the elements so that they will act as a unit. To act as a unit, they must be connected together in such manner that the buckling resistance of the individual element is at least as great as that of the member as a whole. For example, if the critical buckling stress for the section of channel between points *a* and *b* of Fig. 4–42(*a*) is less than that of the entire member, failure will occur through local buckling (dotted line). As shown by Fig. 4–42(*b*), the local failure could easily lead to a general failure with no further increase of load.

To prevent this type of failure and to ensure that the strength of the column can be measured by L/r for the complete section, it is necessary to do three things:

1. Limit the ratio of width to thickness of outstanding leg or other unconnected element as required by the specification, or according to experimental data giving the relationship of that ratio to the critical local buckling stress.
2. Limit the ratio of transverse rivet or weld spacing to material thickness as required by the specification, or according to experimental data.
3. Limit the longitudinal distance between rivets, welds, or lacing bars according to the specification, or so that the L/r ratio for no element exceeds that for the entire member.

Since the form of a buckled portion of the member of Fig. 4–42(*b*) is that of a true pin-connected member, and since the member as a whole usually has some end restraint, most specifications limit the L/r of a



(a) (b)

FIG. 4–42.

longitudinal portion of an element between lacing. For buildings, the AISC specification limits the ratio for the element to not more than three-fourths of L/r for the entire member. For bridges, the AREA specification limits the ratio for the element to two-thirds of that for the entire member, but not more than 40. The longitudinal spacing of rivets or welds connecting nonlaced elements is also limited by most specifications.

A column which is deflected laterally has a bending moment $P\Delta$ varying from a maximum at the point of greatest deflection Δ to zero at the points which have not deflected. If the bending moment changes from point to point, shear must be present in the member. The amount of shear is assumed by specifications to be a definite portion (usually from 2.0 to 3.5 per cent of the axial load) or a variable percentage of the axial load, depending on the L/r ratio of the member. The amounts used are based on estimates and computations of the curves of deflected actual and ideal columns. Of the shear thus determined, a portion due to initial crookedness might occur at working loads, but the entire amount should not be realized until the ultimate column load is reached.

The lacing and its connections to the elements of the section are designed to resist the entire amount of this assumed shear plus any shear known to exist because of the eccentric loads or lateral loads. Each lacing bar is designed to resist both tension and compression. When a lacing bar acts as a compression member, its resistance to buckling is a function of the L/r ratio for the bar. The design of the bar is often controlled further by definite specification requirements for the thickness of the bar and for its angle of inclination to the axis of the member.

Figure 4–43 shows three examples of lacing. Parts (*a*) and (*b*) show bars whose centerlines intersect at a point on the gage line of the element being laced. The bars of part (*c*) do not so intersect. The intersecting type of

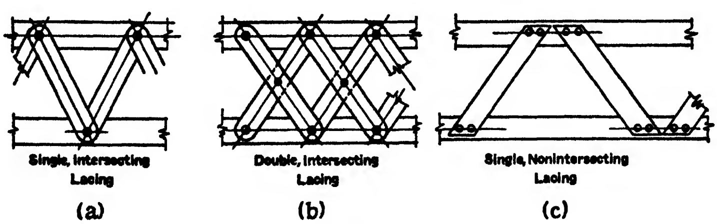


Fig. 4–43.

lacing is generally considered more desirable than the nonintersecting type because axial loads in the nonintersecting lacing bars cause bending moment in the elements being braced. The nonintersecting type should be limited in use to secondary members whose size is controlled by factors other than strength.

The question of whether to use single lacing (*a*) or double lacing (*b*) is often answered by the specification. If it is not, then a comparative design to determine the relative costs of the two types may be desirable.

In a member of the type shown by parts (*n*) or (*o*) of Fig. 4–8, the total design shear is assumed to be equally divided between the top cover plate and the bottom lacing. In the types having two parallel planes of lacing, the shear is assumed to be equally divided between the two planes.

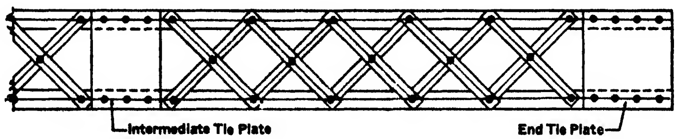


Fig. 4–44.

At the ends of the member and at other points at which the lacing is interrupted, tie plates are employed. A typical tie plate is illustrated by Fig. 4–44. The function of the tie plate is to hold the elements of the member parallel and at the correct distance apart, and to correct any unequal distribution of axial load between the elements. The dimensions and connections required for tie plates are defined by most specifications.

When possible, the end lacing bars should overlap the tie plates, using a rivet common to both. This may lead to detail difficulties, however, so that the solution shown by Fig. 4–44 is usually used. The lacing bars should be terminated as close as possible to the tie plate so as to minimize secondary bending in the element being laced.

Tie plates spaced at intervals along the member may be used in place of lacing, provided the elements connected are capable of resisting the shear for which the lacing would be designed.

Some specifications permit the use of perforated cover plates in place of lacing. The "perforations" are merely large holes cut in the plate so as to

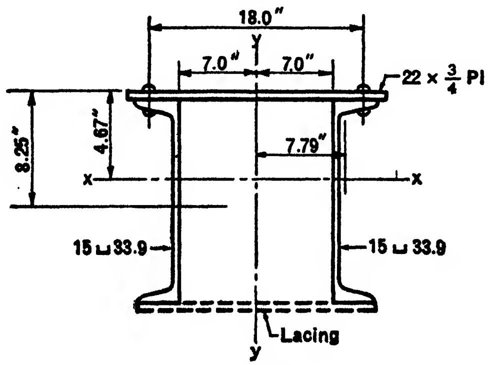


Fig. 4–45.

lighten it, and in some cases, to provide access. (See AASHO Specification, Section 3.6.36.)

EXAMPLE 4–29. A built-up compression member with riveted ends is shown by Fig. 4–45. Determine the allowable compressive axial load on the member and design the lacing. Use the AREA specification. The distance between lateral supports in a vertical plane is 15 ft and in a horizontal plane 30 ft. Use ¾-in. rivets in working out this problem.

The properties of the section, computed in a tabular form, can be presented as follows:

Section	A	y (to Top)	Ay	Ay^2	I_{x-x}	x	Ax^2	I_{y-y}
2—15 ⊔ 33.9.....	19.8	8.25	163.4	1,348	625.2	7.79	1,200	16.4
1—22 × ¾ Pl....	16.5	0.375	6.2	2	0.8	0.0	0	665.5
Total......	36.3		169.6	1,350	626.0		1,200	681.9

Then

$$\bar{y} = \frac{169.6}{36.3} = 4.67 \text{ in.},$$

$$I_x = 1{,}350 + 626 - (169.6 \times 4.67) = 1{,}184 \text{ in.}^4,$$

$$r_x = \sqrt{\frac{1{,}184}{36.3}} = 5.71 \text{ in.},$$

$$I_y = 1{,}200 + 682 = 1{,}882 \text{ in.}^4,$$

$$r_y = \sqrt{\frac{1{,}882}{36.3}} = 7.20 \text{ in.},$$

$$\frac{L}{r} \text{ (for axis } x) = \frac{12 \times 15}{5.71} = 31.5,$$

$$\frac{L}{r} \text{ (for axis } y) = \frac{12 \times 30}{7.20} = 50.0, \qquad \textit{Controls}$$

$$F_a \text{ (allowable compressive stress)} = 15{,}000 - (\tfrac{1}{4} \times \overline{50}^2) = 14{,}375 \text{ psi.}$$

The allowable compressive axial load P is $A \times F_a$, or 522 kips.

The shear to be assumed is given by the specification as

$$V = \frac{P}{100}\left(\frac{100}{(L/r) + 10} + \frac{L/r}{100}\right),$$

in which L/r is that for the axis perpendicular to the plane of the lacing. Substituting for P and L/r, a value of 11.3 kips is obtained for V. The shear is assumed to be resisted equally by the cover plate and the lacing. The lacing must be designed, therefore, to resist a shear of 5.65 kips.

The maximum spacing between lacing points on one flange is that which causes L/r for the one channel to be ⅔ × 50, or 33.3. Then

$$\frac{\text{Spacing}}{r} = 33.3$$

and

$$\text{Spacing} = 33.3 \times 0.91 = 30.3 \text{ in. max.}$$

The AREA specification requires that double lacing be used whenever the transverse distance between the rows of rivets connecting the lacing to the flanges exceeds 15 in. and when only one rivet is used at each end of a lattice bar. Double lacing will be used with the bars inclined at 45° to the longitudinal axis of the member. The distance between lacing points will be 18 in., which is below the maximum permissible, as just computed. If the shear resisted by the lacing is assumed to be equally resisted by the two bars at any section, the load per bar is

$$\frac{5.65}{2} \times \sqrt{2}, \quad \text{or} \quad 4.00 \text{ kips, tension or compression.}$$

The distance between the end rivets, measured along a lacing bar, is 18 $\sqrt{2}$, or 25.5 in.

The thickness of a double-lacing lattice bar is required by the specification to be not less than one-sixtieth of the distance between end rivets, or 0.425 in. The width may not be less than three times the nominal rivet diameter, or $2\frac{1}{4}$ in. A width of $2\frac{1}{2}$ in. would be practical. The tensile and compressive strengths of a $2\frac{1}{2} \times \frac{7}{16}$ bar are now computed. Thus,

$$A \text{ (net)} = \tfrac{7}{16}\,(2.50 - 0.88) = 0.71 \text{ in.}^2,$$

$$\text{Allowable } P \text{ (tensile)} = 0.71 \times 18 = 12.8 \text{ kips.}$$

The effective L in L/r for double lacing is assumed by the AREA specification to be 0.7 times the actual L between end rivets. Then

$$\frac{L}{r} = \frac{25.5 \times 0.7}{0.289 \times \frac{7}{16}} = 141,$$

F_a (allowable compressive stress) $\doteq 15{,}000 - (\frac{1}{4} \times \overline{141}^2) = 10{,}030$ psi.

Since L/r exceeds 140, the secant formula given by Appendix A of the AREA specification should be used. If the above F_a value is checked using the secant formula, it is found to be sufficiently accurate. Then

$$\text{Allowable } P \text{ (compressive)} = 2.5 \times \tfrac{7}{16} \times 10.03 = 11.0 \text{ kips.}$$

Since both the allowable tension and the allowable compression per lacing bar exceed the load of 4.00 kips per bar, double lacing consisting of $2\frac{1}{2} \times \frac{7}{16}$ in. bars is satisfactory.

The resultant load from two bars on one end rivet is

$$2 \times \frac{4.00}{\sqrt{2}}, \quad \text{or} \quad 5.65 \text{ kips,}$$

parallel to the longitudinal axis of the column.

The shear value of a $\frac{3}{4}$-in. rivet, using the AREA allowable stresses, is 5.96 kips. The bearing value in the channel flange ($\frac{5}{8}$ in. thick) is 12.7 kips and in the lacing bar 8.85 kips. A single rivet through the ends of two intersecting bars is sufficient. Rivets should be installed also at the center intersections.

Tie plates or stay plates must be provided. The end tie-plate dimensions are controlled by the specifications, as follows:

Thickness: not less than 1/50 of the transverse distance between rivets;

Length: not less than $1\frac{1}{4}$ times the transverse distance between rivets.

A plate measuring 22 x $\frac{3}{8}$ x 1—$10\frac{1}{2}$ will satisfy these requirements. One plate should be used at each end of the member. Each plate should be connected to each flange by at least eight rivets. [See Art. 47(*d*), AREA.]

If the lacing is interrupted at the intermediate panel point, intermediate tie plates must be used. The length of these plates must be at least three-fourths of the transverse distance between rivets, or 1 ft $1\frac{1}{2}$ in. The required thickness is the same as that for the end tie plates. At least three rivets are required in the connection to each flange [see Art. 47(*b*), AREA].

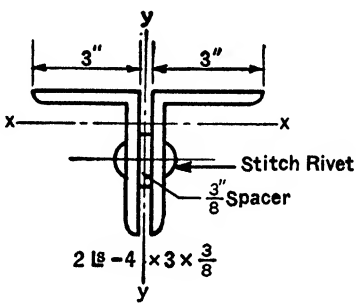


FIG. 4–46.

EXAMPLE 4–30. A building truss has a top chord consisting of two 4 x 3 x $\frac{3}{8}$ angles placed as illustrated by Fig. 4–46. It is laterally supported both vertically and horizontally at the panel points. The spacing of the panel points is 4 ft 3 in. The angles are separated by $\frac{3}{8}$ in., the thickness of the gusset plates. To prevent local buckling and to ensure action of the two angles as a single member, spacers and stitch rivets are installed at regular intervals along the member. What is the maximum allowable spacing of the spacers and rivets, according to the AISC specification?

The properties of the combined section are:

r_x = 1.26 in. (taken from *Steel Construction*, AISC),

r_y = 1.31 in. (taken from *Steel Construction*, AISC),

$$\frac{L}{r}\text{ (max)} = \frac{51}{1.26} = 40.5.$$

The specification requires that compression members of this type have stitch rivets at not more than 24 in. spacing or that spacing for which L/r of one angle is equal to $\frac{3}{4}$ of L/r for the entire member. Thus, for one angle,

$$r_z = 0.64 \text{ in.},$$

$$\text{Maximum permissible } \frac{L}{r} \text{ for one angle} = 0.75 \times 40.5,$$

$$L \text{ (spacing)} = 0.75 \times 40.5 \times 0.64 = 19.4 \text{ in.}$$

Spacers and stitch rivets would probably be called for on the details of the truss chord at approximately 18 in. center-to-center.

4–38. Built-up Tension Members. Tension members can be built up in the same manner as compression members. Some types are illustrated by Fig. 4–1. Many of the built-up compression sections of Fig. 4–8 can be used as tension sections also. The allowable unit tensile stress is, of course, not a function of the slenderness of the member. The choice of section is based primarily on the required net area and on the limiting L/r values.

The elements assembled to form a built-up tension member are connected to each other for the following reasons:

1. To limit the L/r ratio for the individual element to that permitted by the specification.
2. To cause the elements to act together as a single member. The connection of elements is assumed to transfer stress from one element to the other as required to produce equal unit stress on the gross area of all the elements.
3. To permit easier handling of the member. The handling of a long slender part may be difficult, whereas a number of such parts, connected to form a single member of greater stiffness, may often be handled more easily and without damage.

The size of tie plates is given by most specifications. The spacing, however, is usually not specified. Tie plates should be used at the ends of the member and at interior points as required by reasons (1) and (3). Experiences with similar existing structures and the desires of the fabrication shop may serve as useful guides for determining the tie plate spacing.

4–39. Members of Two or More Types of Metal. Built-up members with two or more different types of metal in the same section are frequently

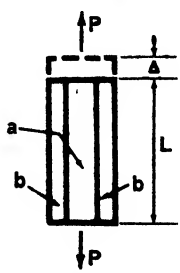


FIG. 4–47.

used in aircraft. For example, built-up frames or bulkheads may have the web and one flange made of a lightweight alloy and the other flange of steel.

If the materials used in the section have equal values for the modulus of elasticity, the analysis of stresses is no different from that of a member

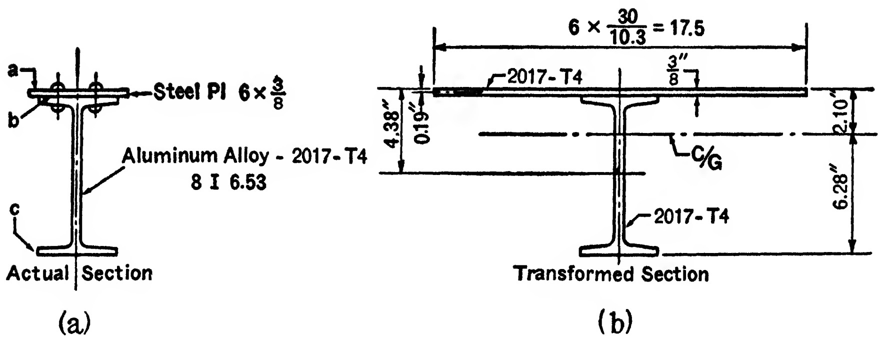


FIG. 4–48.

made from a single material. In determining the suitability of the section, however, allowance must be made for possible differences between the ultimate strengths of the materials used.

If the materials have different values for the modulus of elasticity, the analysis of stresses is modified by the use of a transformed section. Fig. 4–47 shows a tension member consisting of a strip of metal of one type flanked by strips of another type. The axial load P is resisted by all three strips. For this presentation the metals are referred to as metals a and b. The modulus of elasticity is referred to as E_a or E_b, the unit stress as f_a. or f_b, and the total area of each material as A_a or A_b.

For stresses not exceeding the elastic limit, the following equations may be written:

$$f_a = E_a\Delta/L,$$

$$f_b = E_b\Delta/L.$$

Since the metals act together as a single member, the unit elongation Δ/L is the same for metal a as for metal b. Solving the above equations for Δ/L and equating them,

$$f_b = f_aE_b/E_a,$$

$$P = A_af_a + A_bf_b = f_a\left(A_a + A_b\frac{E_b}{E_a}\right).$$

The quantity

$$\left(A_a + A_b\frac{E_b}{E_a}\right)$$

is called the *area of the transformed section.* The transformed section is an imaginary one in which the metal b has been replaced by an amount of metal a having the same stiffness (resistance to elongation) as the original strip of material b.

Any composite member, whether in tension, compression, or flexure, may be analyzed by means of a transformed section. The same procedure is used also in the analysis of combinations of metal with other materials; for example, reinforced concrete.

EXAMPLE 4–31. Compute the flexural stresses at points a, b, and c of the section shown in Fig. 4–48(a). The section is subject to a bending moment of 22 ft-kips (compression above the neutral axis). The modulus of elasticity for 2017-T4 is 10,500,000 psi; for steel, 30,000,000 psi.

The properties are computed for a transformed section consisting entirely of 2017-T4 material. The steel plate is replaced by an area A' of 2017-T4, called the *equivalent area*, and is equal to

$$6 \times \frac{3}{8} \times \frac{30,000,000}{10,500,000} = 6.43 \text{ in.}^2$$

This equivalent area is assumed to be concentrated at the same location vertically as the actual steel plate. (See part (b) of Fig. 4–48.)

The properties of the section area:

Item	A	A' (Equiv. Area)	y	$A'y$	$A'y^2$	I_{o-o}
I-beam	5.40	5.40	4.38	23.65	103.60	57.55
Plate	2.25	6.43	0.19	1.22	0.23	0.08
Total		11.83		24.87	103.83	57.63

Then

$$\bar{y} = \frac{24.87}{11.83} = 2.10 \text{ in.},$$

$I = 103.83 + 57.63 - (2.10 \times 24.87) = 109.2 \text{ in.}^4$ (Moment of inertia of the transformed section about its center of gravity.)

At point a,

$$f \text{ (on transformed section)} = \frac{12 \times 22,000 \times 2.10}{109.2} = 5,070 \text{ psi compression;}$$

$$f \text{ (on actual steel plate)} = 5,070 \times \frac{30}{10.5} = 14,500 \text{ psi compression.}$$

At point b,

$$f = \frac{12 \times 22,000 \times 1.72}{109.2} = 4,150 \text{ psi compression.}$$

At point c,

$$f = \frac{12 \times 22,000 \times 6.28}{109.2} = 15,200 \text{ psi tension.}$$

PROBLEMS

NOTE: For each problem in which a section is to be selected, it is intended that the most economical practical section be chosen. Unless it is otherwise noted, the member is to be considered as a "main member," and is made of structural steel.

4–1. Choose a round steel bar, threaded but with ends upset, to resist an axial tension of 14,000 lb. Use the latest AISC specification.

4–2. Same as Prob. 4–1, but with threaded ends, not upset.

4–3. What is the most economical size of steel angle to resist an axial tensile load of 26 kips? Assume the angle to be connected at its ends by welding and without assembly holes or other holes. Use the AISC specification.

4–4. Under the AISC specification, what is the permissible tensile load for a 4 x 3 x 3/8 angle connected at its ends by a single row of 3/4-in. rivets through the 4-in. leg?

4–5. Same as Prob. 4–4, but using the AREA specification.

4–6. Select an angle to resist an axial tension of 30 kips when connected by a single row of 7/8-in. rivets. Use the AISC specification.

4–7. Same as Prob. 4–6, but using the AREA specification.

4–8. A 1/8 in. thick plate of aluminum alloy Alclad 2024-T86 has a row of 3/8-in. cold-driven rivets spaced at 1½ in. The holes are drilled. What is the probable ultimate tensile strength of the sheet per foot of width? Refer to the *Alcoa Structural Handbook.*

4–9. Determine the minimum net section of the 14 x 3/8 Pl shown by Fig. 4–49. Show where the critical section is located. Rivets are 3/4 in. diam. Use the AISC specification.

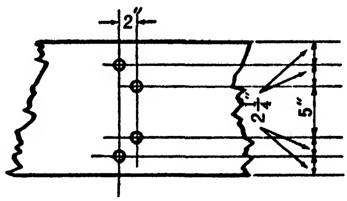


FIG. 4–49.

4–10. An equal-leg angle is to resist a total tension of 44 kips. It is connected to gusset plates by 3/4-in. rivets through both legs. The pitch on each leg is 3 in., and the rivets in one leg are staggered 1½ in. with respect to those in the other. Using the usual gages and the AISC specification, select the most economical 3/8 in. thick equal-leg angle for this application.

4–11. Two angles, 6 x 3½ x 3/8, are placed with long legs back-to-back and are separated at frequent intervals by 3/8-in. fillers. The member is connected at its ends by a single row of 7/8-in. rivets (through the long legs) to a 3/8-in. gusset plate. If the length between gusset plates is 28 ft, is the member suitable for a tension member under the AISC specification? (Check L/r about each axis.) What is the maximum allowable tensile load?

4–12. Using the AISC specification, select a single, equal-leg angle to resist an axial load of 21 kips tension. The laterally unsupported length of the member is 9 ft. Use one row of 3/4-in. rivets.

4–13. A 6 x 4 angle is riveted by two rows of ⅞-in. rivets through the 6-in. leg. The gages on this leg are 2 in. from the back of the angle to the first row and 2½ in. between rows. The pitch is 3 in. in each row. The stagger is 1½ in. Use the AISC specification and select a thickness for the angle. The tensile load is specified as 75 kips.

4–14. A 10 WF 33 of 16 ft laterally unsupported length is used as a building column. What axial compressive load is permitted under the AISC specification? Compute and then check results by handbook tables.

4–15. If the column of Prob. 4–14 is assumed to have rounded ends, what is its critical or buckling load as indicated by the Euler equation? What factor of safety does the AISC specification provide against failure for this particular column? Use $E = 30{,}000{,}000$.

4–16. Select the most economical steel WF section to resist an axial compression of 60 kips. The length between supports is 16 ft. Use the allowable stresses of AISC. Solve without reference to handbook tables of allowable loads. Check selection by handbook tables.

4–17. Select a steel angle, 6 ft long, to resist an axial compresion of 20 kips. Use the AISC specification.

4–18. A column consists of an 8 WF 24, 20 ft. long, laterally supported in all directions at the ends, but at the center in a direction perpendicular to the web only. Under the AISC specification, what is the allowable load?

4–19. What is the lightest WF section permitted by the AISC specification for a column to resist an axial compression of 80 kips when the column is 32 ft long and laterally restrained both ways at the ends and in one plane only at its midpoint?

4–20. The top chord of a truss consists of two 4 x 3 x ⅜ angles with the short legs back-to-back but separated ⅜ in. by fillers and gusset plates. Lateral support is provided parallel to the short legs at intervals of 5 ft and parallel to the long legs at intervals of 10 ft. What is the maximum axial compression allowed by the AISC specification?

4–21. (*a*) If the member of Prob. 4–20 is made of aluminum alloy 2014-T6, what is the maximum allowable load if a factor of safety of 2.5 is maintained? Use the values of the critical buckling stress given by Table 4 of the *Alcoa Structural Handbook*. Assume K for the end support condition to be 0.8. (*b*) Compute the allowable load according to the aluminum specifications. (See Appendix E.)

4–22. If the distance between lateral supports of the member of Prob. 4–21 is reduced to 2 ft 4 in. and 4 ft 8 in., instead of 5 ft and 10 ft, respectively, what is its allowable load according to the aluminum specifications? Note that local buckling must be considered.

4–23. Select a single angle of aluminum alloy 2014-T6 to resist an axial compression of 30,000 lb. The laterally unsupported length is 8 ft. Assume K for riveted ends to be 0.75. Use the aluminum specification.

4–24. Select the most economical double-angle truss compression member to carry a total direct stress of 84 kips. Gusset plates are ⅜ in. thick. The member is supported in the vertical plane at 6-ft centers and in the horizontal plane at 12-ft centers. Use the AISC specification.

4–25. Select a double-angle section for a strut of 6061-T6 aluminum alloy to resist axial compression of 60,000 lb. The length is 12 ft. The angles are to be riveted to opposite sides of a ⅜-in. gusset plate. Use $K = 0.75$ and a factor of safety of 3. For local buckling, assume that the support conditions of a double-angle strut are as given in Case *B*, Table 19(*a*), *Alcoa Structural Handbook.*

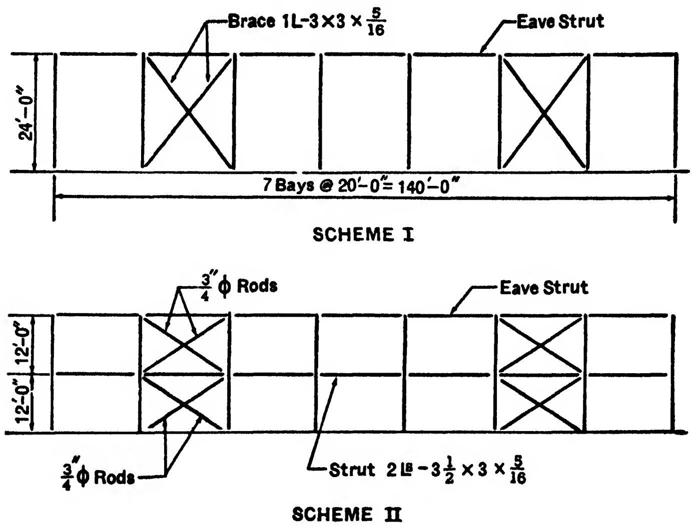


FIG. 4–50.

4–26. Fig. 4–50 shows two schemes for the arrangement of columns and bracing in the side elevation of a building. Compare the two schemes for economy. In scheme I, the columns are braced at the top and bottom only. In scheme II, the columns are also braced at the midpoint in one plane only. The load per column for either scheme is 90 kips. The eave struts are the same for either scheme, so that their weight may be neglected in the comparison. Ten per cent should be added to the computed total weights to cover connection, details, rivet heads, etc. Use standard WF sections. Use the AISC specification.

4–27. A steel column in a two-story building receives a load of 20,000 lb from the roof and 85,000 lb from the second floor, the load in each case being centrally applied. If the column is supported laterally in all directions at the roof and second-floor levels, suggest a section for the column which supports the second floor and roof. Story heights are 12 ft. Use the AISC specification.

4–28. Find the allowable axial load for a column made of four 5 x 3½ x ⅜ angles and a 12 x ⅜ web, as an I-shaped section 12½ in. deep. The 5-in. legs are outstanding, and the unsupported height is 17 ft. Use the AISC specification.

4–29. Select a WF section for a column with 30 kips axial load. The bottom is fixed, but the top is *not* laterally restrained. The length from bottom to top is 15 ft. (See Art. 4–9.) Use the AISC specification.

4–30. Compute the axial compressive load allowed by the AREA specification for the section of Fig. 4–51. The laterally unsupported length is 26 ft 6 in.

4–31. What is the allowable axial compressive load for an 8 x 8 x 7⁄16 angle, 15 ft long, laterally supported at the ends only, and used as a main member? Use the AISC specification.

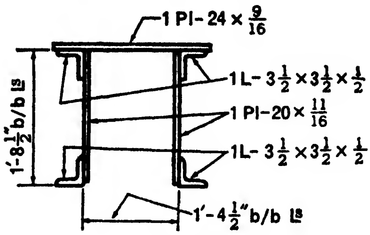


FIG. 4-51.

4-32. Find the maximum tensile and compressive flexural stresses for the member shown in Fig. 4-52.

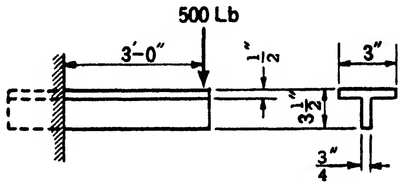


FIG. 4-52.

4-33. Two angles, 6 x 4 x ⅜, are placed with the long legs pointing down and back-to-back. They span between simple supports a distance of 8 ft. If the allowable flexural unit stress is 10,000 psi, what is the maximum uniform load in pounds per foot of beam that the member may carry?

4-34. Select a WF section of structural steel for the following conditions: load per linear foot 2,400 lb, *not* including the weight of the beam itself; simple span 32 ft, laterally supported over its full length. Use the AISC specification.

4-35. A crane-runway beam is subject to two wheel loads of 8,000 lb each, spaced at 6 ft center-to-center. The span is 20 ft. Select the lightest WF section to resist the flexural stresses. Assume lateral support over the full length, and use the latest AISC specification. The wheel loads given are live load only. To these loads must be added the effect of impact. Assume that the lateral supports provide resistance to any horizontal forces caused by movement of the crane trolley and that such forces do not stress the beam.

4-36. By use of tables in the AISC handbook, select the lightest WF section to resist in bending a uniform load of 1,000 lb per ft on a span of 24 ft. The load given does not include the weight of the beam itself. Assume lateral support at all points. For the section chosen, compute the maximum unit flexural stress. Note how the value of this stress may be obtained from the tables by direct proportion.

4-37. (*a*) A 14 WF 30 is used for a span of 26 ft without intermediate lateral support. What compressive flexural stress is permitted by the latest AISC specification? What is the amount of the largest concentrated load that can be placed at the center of the span without exceeding this allowable stress? (*b*) Same as (*a*) but using the AREA specification.

4–38. Select the lightest WF section suitable flexurally to resist a concentrated load of 10 kips on a span of 18 ft, without lateral support. Use the AISC specification.

4–39. Same as Prob. 4–38, but using the AREA specification.

4–40. An American standard section (I-beam) is to carry a uniform load of 2,000 lb per ft, including its own weight, over a span of 15 ft, with intermediate lateral support at the middle of the span only. Select the most economical section permitted by the AREA specification.

4–41. A 9-in. brick wall is to be carried by a steel lintel over an opening 18 ft wide. Assume that there is ample width of wall at each end of the span to resist thrust loads. The load on the lintel may then be assumed triangular, with a center height of half the span length. To limit deflection, use a depth of section of not less than 1/25 of the span. Assume a 12-in. bearing length at each end and full lateral support. The section is exposed to the weather on the lower and outer surfaces. Select a section consisting of two channels, 9 in. back-to-back, flanges pointing inward, webs vertical, and having pipe separators and bolts. Use the AISC specification.

4–42. Compute the allowable uniform load per foot for a 10 I 25.4 acting as a fixed-end beam on a span of 23 ft, without intermediate lateral support. Use the AISC specification. (The location of the points of contraflexure may be found in the handbook, *Steel Construction.*)

4–43. Using the AISC specification, select a section for the beam of Fig. 4–53. Lateral support is given only at the reaction points, *A* and *B*. (Note that in com-

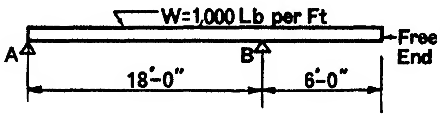


FIG. 4–53.

puting the allowable flexural compressive stress for the free-ended overhang, the L used is *twice* the length of the overhang.)

4–44. Select a beam section, using the AREA specification, to support two concentrated loads of 27 kips each, placed at the third-points of the span. The span is 36 ft. Lateral support is provided at the ends and center only.

4–45. What is the largest concentrated load that can be applied without exceeding the allowable unit flexural stresses to an aluminum alloy 2014-T6 beam (12-in. standard I-beam weighing 11.31 lb per ft) on a 15-ft simple span without lateral support? Use the aluminum specifications.

4–46. Using the values in the *Alcoa Structural Handbook*, what is the ultimate load for the beam of Prob. 4–45? For a factor of safety of 2.5, what is the allowable load? Compare this to the result obtained by using the aluminum specifications.

4–47. If the beam of Prob. 4–46 is loaded with the allowable load computed in that problem, what is the approximate factor of safety with respect to local buckling? As the support condition of the flange is between that of simply supported and that of built-in, obtain the value of KL/r after interpolation in Table 19(*a*) of the *Alcoa Structural Handbook.*

4-48. A 21 WF 62 has an end shear of 60 kips. What is the average unit shearing stress (using the over-all depth of the section)? What is the actual maximum unit shearing stress?

4-49. Using the AISC specification, compute the maximum uniform load that can be placed on a 14 WF 34 beam with a span of 6 ft 6 in. Give answer in pounds per foot.

4-50. Compute the length of end bearing required to prevent web crippling for the beam and load of Prob. 4-49.

4-51. A 21 WF 62 beam supports an 80-kip concentrated load at midspan. Using the latest AISC specification, compute the bearing length required at the load and at the end reactions to prevent web crippling.

4-52. Compute the bearing length required to maintain a factor of safety of 2 against web crippling of a 10 I 9.01 of aluminum alloy 6061-T6 having an end reaction of 22 kips.

4-53. Compute the bearing length required to maintain a factor of safety of 2 with respect to vertical buckling of the web of the beam of Prob. 4-52.

4-54. A 15 I 42.9 subject to a uniform load of 1,500 lb per ft (total) on a span of 20 ft has holes for ¾-in. rivets in each side of each flange. Compute the tensile and compressive flexural stresses at the section containing the holes by the following methods:

(a) Assuming a shift of the neutral axis (theoretical method), (b) AREA, (c) AISC.

4-55. Select a WF section for a laterally unsupported beam to span 18 ft and to support a moving concentrated load of 20 kips. Two holes for ⅞-in. bolts occur in the tension flange only, at the middle of the span. Use the AISC specification. Assume that the neutral axis shifts.

4-56. Select a WF beam section of 20-ft span to resist a load of 10 kips, applied as shown by Fig. 4-32(a), with the angle θ equal to 20°. Lateral support is provided at the ends only. Use the AISC specification.

4-57. Select the most economical section for the roof purlins shown in Fig. 4-54. Use the AISC specification. Assume that the load components parallel to the roof

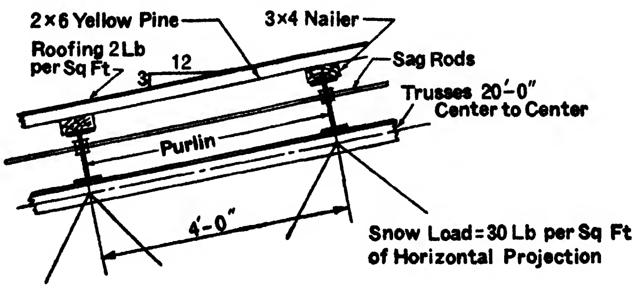


FIG. 4-54.

are transferred by the sag rods to the ridge of the roof. The sag rods are at the third-points of the span of the purlins. Lateral bending occurs on 6 ft 8 in. spans.

4-58. For the jib crane shown by Fig. 4-55, select an American standard beam section for the boom and a WF section for the mast. In both cases, select the most

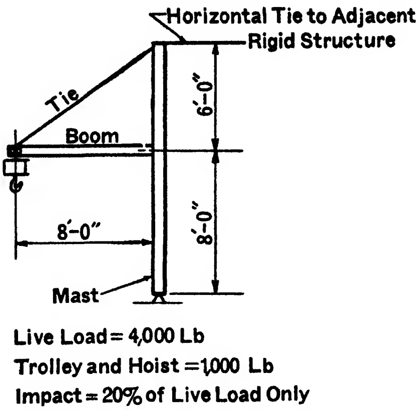


FIG. 4–55.

economical section, using the AISC specification. For the boom design, place the moving loads at midspan. For the mast design, place the moving loads at the outer end of the boom. Design the boom first, estimating the boom weight to start the design. Then design the mast, using the corrected value of boom weight. Assume the entire vertical effect of the boom weight to be applied to the mast at its upper end.

4–59. Choose a section for the column shown by Fig. 4–56. Assume the beams to be laterally supported and fully loaded. Use the safe load tables for beams (AISC handbook) to obtain the beam loads. In obtaining the beam loads, assume the span to be the distance center-to-center of columns. In designing the columns, assume the loads to be applied at the column face. Use the AISC specification.

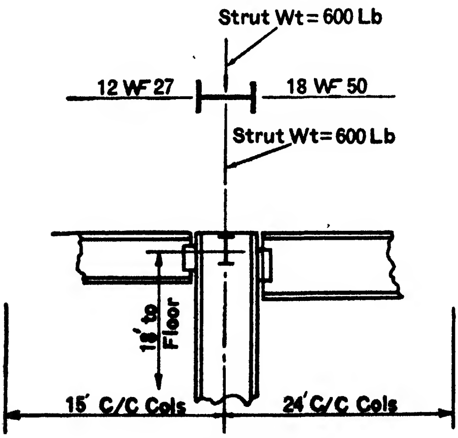


FIG. 4–56.

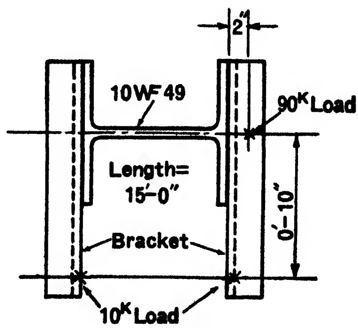


FIG. 4–57.

4–60. Figure 4–57 shows the plan of a building column, the loads, and the point of application of those loads. The column is laterally supported at the ends only. Does it satisfy the requirements of the AISC specification?

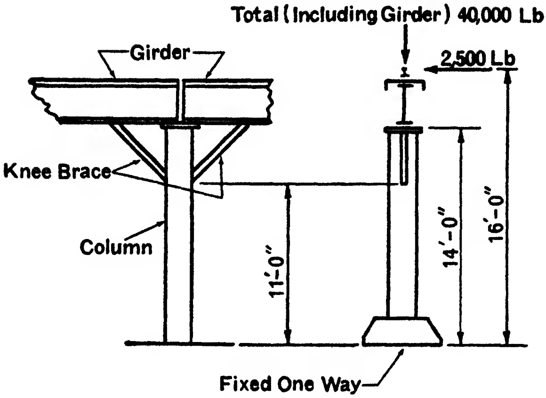


FIG. 4-58.

4-61. Select a section for the crane columns shown by Fig. 4-58. Note that in one direction, the column must be assumed fixed at one end and entirely free at the other (effective $L = 28$ ft), while in the other direction, lateral support is given by the base detail and by the knee brace.

4-62. For the reinforced beam of Fig. 4-59, compute the allowable bending moment for the following conditions: (*a*) Laterally supported, using the AISC specification. (*b*) Laterally unsupported over a 20-ft length, using the AISC specification. (Use three-fourths of the area of the top flange and cover plate as the equivalent of *bt*.) (*c*) Laterally unsupported over a 20-ft length, using the AREA specification.

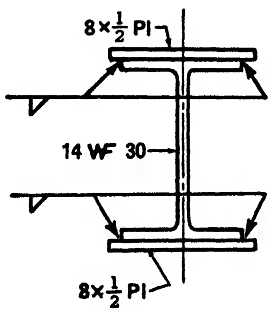


FIG. 4-59.

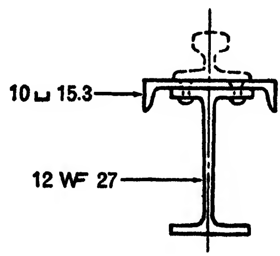


FIG. 4-60.

4-63. Compute the maximum permissible concentrated live load for the girder shown by Fig. 4-60 if laterally unsupported on a simple span of 18 ft. The total load consists of dead load plus live load plus impact (20 per cent of live load). Use the AISC specification and assume three-fourths of channel and top flange as the equivalent of *bt*.

4-64. Select a section to carry the moving loads of Fig. 4-61 on a span of 20 ft. Use an American standard section for the main section, reinforced with a top plate or channel as required. Design for the most economical section. Use the AISC specification.

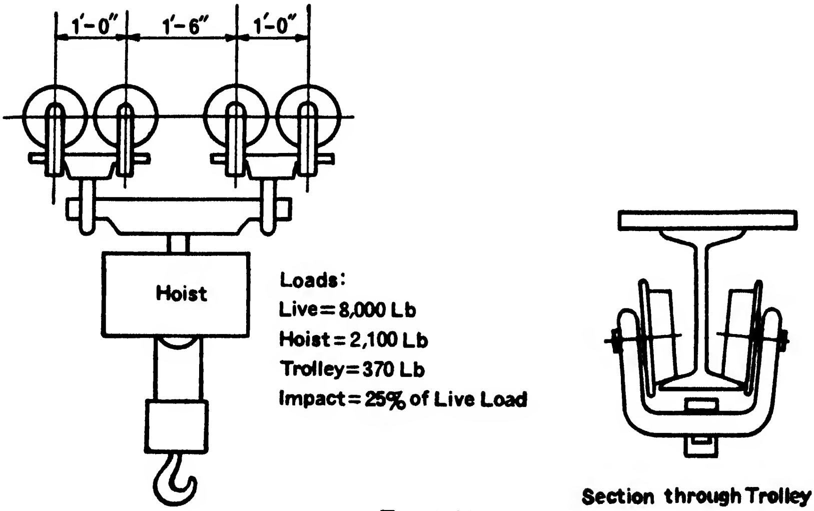


FIG. 4-61.

4-65. A crane end-truck consisting of two 10 $\overset{S}{\sqcup}$ 30 placed 8 in. back-to-back is cut and reassembled by welding to the outline shown by Fig. 4-62. What is the

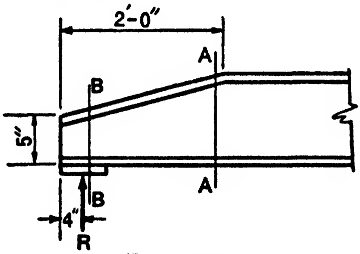


FIG. 4-62.

maximum reaction R that can be applied to the truck without exceeding the AISC flexural stresses at line A? With this reaction, what is the approximate average shear stress in the web at line B?

4-66. A beam is fabricated by welding, as shown by Fig. 4-63. Compute the depth of section required at various points for flexure. After obtaining the re-

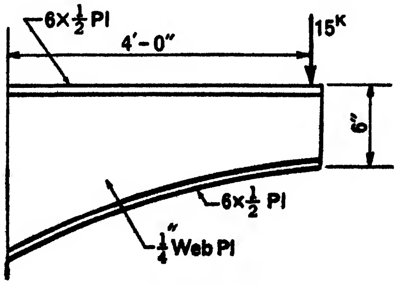


FIG. 4-63.

quired depth, suggest a practical outline for the member. Use the AISC allowable stresses and assume full lateral support.

4–67. A building-truss compression chord consists of two 15 $\underset{\smile}{\text{S}}$ 33.9 placed with toes outward and 15 in. back-to-back. The distance between lateral supports is 15 ft. Assuming the chord to be axially loaded to capacity, design the lacing and tie plates in accordance with the latest AISC specification. Use ¾-in. rivets.

4–68. Select the most economical two-angle section for a truss chord. The axial load P = 130 kips compression. Vertical lateral support is provided at 6-ft centers and horizontal lateral support at 12-ft centers. Select angles and determine stitch rivet spacing. Assume ⅜-in. gusset plates. Use the AISC specification.

4–69. Design lacing and tie plates for the compression member of Prob. 4–30. Use the AREA specification. Use ¾-in. rivets.

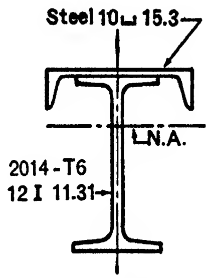


FIG. 4–64.

4–70. Compute the flexural stress at both extreme fibers of the member of Fig. 4–64. The section is subject to a bending moment of 7,500 ft-lb. E for 17S-T4 is 10,300,000 psi; for steel, 30,000,000 psi.

CHAPTER 5

CONNECTIONS

5–1. Introduction. It is important that structural members be selected with due regard to safety, economy, and appearance. It is equally important that careful consideration be given to designing the connections by which the members become part of the completed structure. The strength and safety of a structure is dependent on the connection details which join the members, transferring loads to them and providing reactive forces.

Many more structural failures have occurred because of faulty connection details than because of overstressed main members. Too frequently the structural engineer leaves the development of these details to the structural steel fabricator, considering that his job is completed with the selection of members. Such a practice is usually uneconomical and may be disastrous because the fabricator's detailer is not dealing with forces and loads; he is concerned with dimensions and rivet spacing. The general quality of structural engineering will improve when all structural designers recognize that the selection of main members is only the start of their job.

This chapter deals with connections made by riveting, bolting, or welding. It is recommended that, if the reader has not already done so, he read and understand Chaps. 2 and 3 before proceeding with the work of this chapter. Chapter 2 deals with the usual methods of making connections with rivets and bolts and the assumptions that are made in the use of these connectors. Chapter 3 contains a discussion of the process of welding.

5–2. Riveted and Bolted Connections in Shear. The usual riveted or bolted joint is a shear type connection, as shown by Fig. 5–1(*a*). There

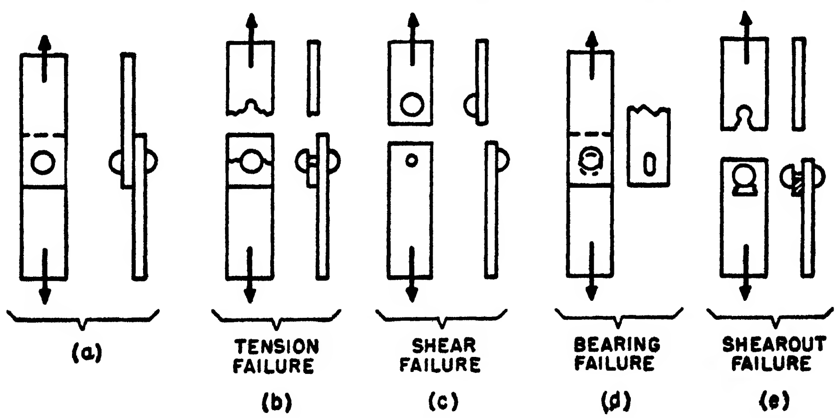


Fig. 5–1.

are four basic ways in which such a connection may fail. Those failures are illustrated by parts (*b*), (*c*), (*d*) and (*e*) of the figure.

1. Because of the reduction in cross section of the members at the hole, the plates may be weakened and one of them may have a tension failure, as shown in Fig. 5–1(*b*). The prevention of this type of failure was discussed in Chap. 4.

2. The rivet shank itself is broken in the shear failure of Fig. 5–1(*c*). The shearing stress is assumed to be uniform over the shank area. The allowable shearing load for one rivet is

$$R = \text{area (shank)} \times \text{allowable unit shear} = Af_s.$$

The allowable unit shearing stresses for hot-driven steel rivets are based on the area of the original shank before driving. Thus, although a ¾-in. rivet swells to 13/16-in. diam. when it is driven, the ¾-in. diam. is used when computing the strength.

3. The bearing failure of (*d*) is the result of compression of the rivet shank against the member material at the side of the hole. The failure consists of distortion of the rivet hole (or, rarely, of the rivet) by load. A shear failure is sudden and complete. The bearing failure, however, is gradual and progressive. Allowable bearing stress is usually conservative. There is no apparent amount of deformation that could be considered to be a failure. However, a bearing failure might be defined as a permanent increase of the hole diameter by some stated percentage of its original diameter. For example, the *Alcoa Structural Handbook* gives bearing values for rivets which are based on the assumption that a 2 per cent permanent elongation of the hole diameter constitutes a bearing yield failure.

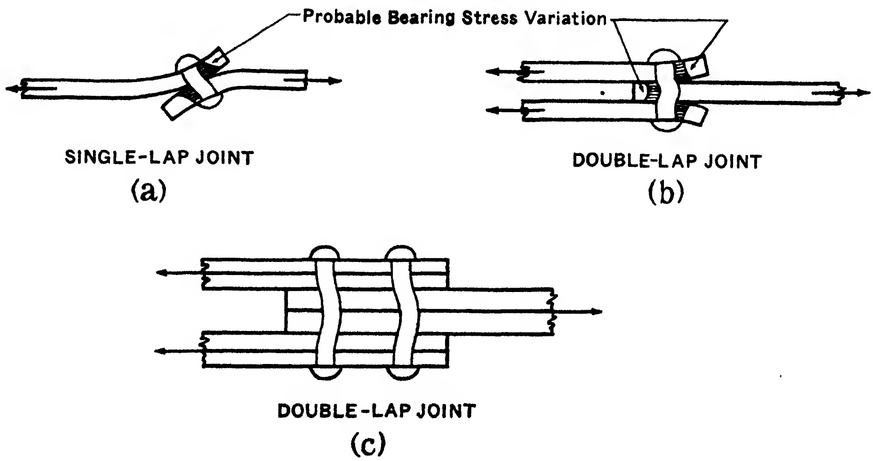


Fig. 5–2.

Figure 5–2 shows two types of joint, a single-lap and a double-lap. The intensity of the bearing stresses (compression) between the rivet shank

and the connected material is shown shaded. In the single-lap joint the nature of the shear transfer causes the connected plates to bend, resulting in higher-than-average bearing stress of one edge of each plate. In the double-lap joint, the outer plates are subject to similar but less severe action, while the inner plate should have nearly uniform bearing stress. In each type, the *average* bearing stress can be computed as

$$f_{br} = \frac{\text{rivet load transferred to material in question}}{\text{shank diameter (nominal)} \times \text{material thickness}}.$$

When f_{br} is the allowable bearing stress, this could be stated also,

$$\text{Allowable } R = tdf_{br}$$

In each of the above, f_{br} is the average bearing stress. For a single-lap joint, the average may be much lower than the maximum. For the inner plate of a double-lap joint there is less difference between maximum and average. Some specifications give one allowable bearing stress, set low enough for safe use with either the single- or the double-lap joint. The AISC specification, however, gives an allowable of 32,000 psi for a single-lap (single shear) connection, and 40,000 psi for the inner plate of the double-lap (double shear) connection. Sometimes the more descriptive term *enclosed bearing* value is used in place of the double-shear bearing value; and *nonenclosed bearing* value is used in place of single-shear bearing value. Thus, in Fig. 5–2, the 32,000-psi value would be used to determine the bearing capacity in either plate of the single-lap joint, or in the outer plates of the double-lap. The 40,000-psi value would be used with the inner plate of the double-lap.

4. Unless the rivet is placed far enough from the ends of the plate, the shear-out failure of Fig. 5–1(*e*) may occur. Prevention of this type of failure is discussed in Art. 5–4.

The allowable loads for bolts are computed in the same manner, but usually with different allowable unit stresses for unfinished bolts. Connections made with high-tension bolts are at present designed in the same manner; in effect, the high-tension bolts are merely substituted for rivets in a similar riveted connection. It is probable that design specifications may be developed which will permit more nearly rational design methods with these bolts.

Normally, bending in the rivets can be neglected. Bending is prevented to some extent by friction between the members being connected. Further, the allowable unit stresses for rivets are established by tests in which that same bending is present. Consequently, no harm is done by neglecting its effect in our computations for most riveted connections.

However, if the rivet has a large grip, as shown in Fig. 5–2(*c*), bending may be so great that its effect cannot be ignored. It is difficult to compute the amount of bending moment for two reasons. The exact amount of displacement of each layer of material cannot be predetermined, because it is limited by an uncertain amount of friction. If the displacement could be determined, the problem of computing the bending moment would still be statically indeterminate. Any analysis made to determine the exact effect of rivet bending would be too cumbersome to use for usual design and probably would be inaccurate.

A practical way of dealing with the problem of bending in long rivets is provided by most specifications, which require that, when the rivet grip exceeds a stated number times the diameter, extra rivets be added above the number ordinarily required. The following quotation from the AREA specifications (Art. 46, Appendix B) illustrates this requirement.

> If the grip of rivets carrying calculated stress exceeds four and one-half times the diameter, the number of rivets shall be increased at least 1 per cent for each additional 1⁄16 inch of grip. If the grip exceeds six times the diameter, the shanks shall be especially designed to fill the holes completely when driven.

Bending of rivets or bolts may occur also where fillers separate the members being joined. The prevention of this type of bending will be discussed in Art. 5–31.

5–3. Bolts and Rivets in Tension. All rivets have initial tensile forces when driven. The hot rivet is driven when in a plastic state. As it cools below the plastic state, the rivet contracts. If shortening is prevented, the rivet will have an initial tension. It can be shown that a considerable force is required to prevent the shortening of a rivet while it is cooling from the plastic state. The force required to prevent shortening would be equal to the force necessary to stretch the rivet from the unstressed length—an amount equal to the contraction. This force will be beyond the elastic limit of the rivet. Since the rate of deformation increases rapidly above the elastic limit, the limit of stress carried by the rivet is probably about equal to the elastic limit.

The increase in length due to tension is fL/E. The shortening due to temperature is $0.0000065tL$, in which t is the number of degrees drop in temperature. With the unit stress in the rivet equal to the yield point of the rivet material, or about 30,000 psi, the following equation may be written:

$$\frac{30{,}000L}{E} = 0.0000065tL,$$

or

$$t = 154\text{F}.$$

The yield point in the rivet material would then be developed with a drop in temperature of only 154F if the rivet were prevented from reducing in length. The drop in temperature from the plastic state is, of course, much greater than 154F. The material around the rivet undoubtedly compresses, reducing the rivet tension; but tensile values near the yield point have been measured.

Bolts required to take tension are usually given an initial tension by tightening the nuts. Aircraft bolts and high-tension bolts for steel structures are tightened with a specified torque to produce tension somewhat below the elastic limit of the material.

Initial tension in a bolt or rivet below the yield-point stress does not reduce its capacity to carry tension,* since little additional tensile stress is developed until the initial stress is exceeded.

Let Fig. 5-3(*a*) represent a fitting attached with a bolt under tension load P. There will be compressive forces between the fitting and the support totaling P in amount. Now let a force P be applied to the fitting as

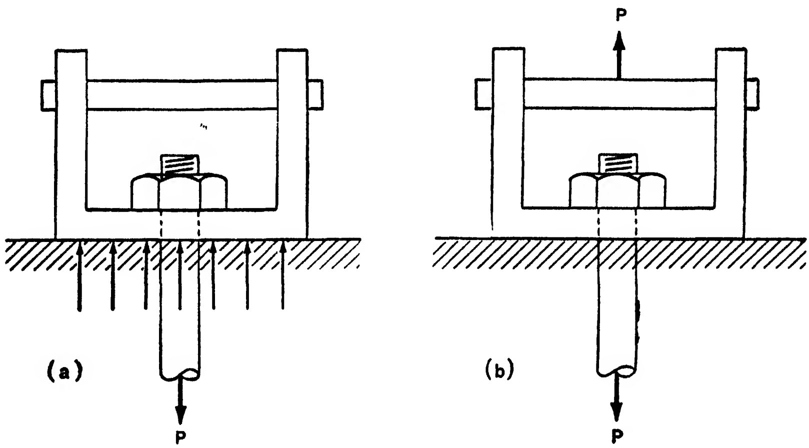


FIG. 5-3.

shown in Fig. 5-3(*b*), just removing the compression between the fitting and the support. The tension in the bolt is obviously equal to P.

The capacity of a bolt or rivet to take shear is reduced by its initial tension. Although rivets are capable of taking tension in an amount equal to their initial tension, specifications usually permit computed tension stresses at rather low values to protect the shearing capacity.

5-4. Edge Distance for Rivets and Bolts. The distance from the center of a rivet or bolt hole to the boundary of the material is called *edge distance.* A special case of edge distance is *end distance*; this is measured from the center of the hole to the boundary toward which the pressure of the rivet is directed.

All edge distances should be kept above certain minimum amounts so as to avoid bulging of the edge material under the pressure of punching the hole or of driving the rivet. Such minimums are specified for punched holes by Section 23(e) of the AISC specification. Where tables such as

* "Tension Tests in Rivets," *University of Illinois Engineering Experiment Station Bulletin,* No. 210 (1930).

this are not given, it is frequently the practice to use one and one-half times the nominal diameter of the rivet or bolt as a minimum value of edge distance.

End distances must be controlled further so as to prevent a shear-out failure, as in Fig. 5–1(e). In Fig. 5–4, the resistance of the piece to shear-out is probably equal to the shearing strength of two areas of length c and width t, where t is the thickness of the material. Occasionally, when available space is limited, this assumption is used to determine the required edge distance for pins. As the driving of a rivet, however, causes stresses of varying and uncertain magnitude in the metal surrounding the hole, this assumption as to required end distance might be unsafe.

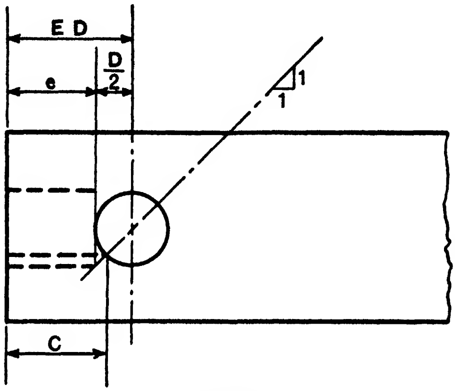


FIG. 5–4.

For rivets and bolts, the shear-out strength may be considered equal to the shearing strength of two areas having length e and width t. The dimension e is often called *net end distance*. Stated as an equation, the required end distance equals

$$\frac{R}{2tf_s} + \frac{D}{2},$$

in which R is the capacity of the rivet or bolt, D is the diameter of the hole, and f_s is the allowable unit shearing stress of the material in the member. Occasionally the use of this formula indicates a required end distance so low that bulging of the edge may occur during driving of the rivet. In such cases the minimum allowable edge distance controls.

For smaller rivets and bolts, especially in the aircraft industries, tables are available giving the required edge distance for various diameters of rivets or bolts in various thicknesses of material. Any one table is limited to a specific type of rivet (countersunk, dimpled, etc.) and to specified combinations of materials.

Many structural specifications state lower limits of edge distance for various sizes of rivets, irrespective of the material thickness. The AISC specification gives an empirical equation for determining the required end distance for rivets with computed stress and a table of minimum edge distances for punched holes. The empirical method gives answers which are generally more conservative than those of the rational method described above. If high tension bolts are used, and their higher shear strength considered in the design, it is recommended that the AISC end distance requirement be increased by 50 per cent.

A maximum allowable edge distance is given by many specifications to avoid undesirable gaps at the edges of parts being joined.

5–5. Spacing of Rivets and Bolts. Spacing is controlled by three considerations:

1. If the holes are close enough together, a shear-out failure occurs in which the material between holes is pushed toward one of the holes, the rivet or bolt in that hole pushing more material toward the next, and so on. The spacing must be sufficient to allow the shearing strength of two planes, each having an area equal to the thickness of the material times the *net* distance between holes, to be equal to or greater than the load on one rivet or bolt.
2. The spacing must also be sufficient to prevent the rivet heads, or the bolt heads or nuts, from interfering.
3. The spacing must be large enough to permit easy installation. If the spacing is too small, interference may result between the tool used and the adjacent nut or head.

Spacing is usually limited by the specification to a stated number times the nominal diameter. For example, the 1949 AISC specification states: "The minimum distance between centers of rivet holes shall preferably be not less than three times the diameter of the rivet."

To prevent gaps between the layers of material connected and to prevent local buckling of compression members, limiting maximum spacings are also given.

5–6. Riveted or Bolted Connections for Axially Loaded Members. If a joint is so arranged that its center of gravity is collinear with that of the member and if all rivets are the same size and material, the rivets may be assumed to be equally loaded. To design the connection it is necessary to first determine the controlling strength R of one rivet (its bearing or shearing strength, whichever is smaller). The number of rivets needed to resist the load P is then given by $n = P/R$.

A complete design of the joint includes locating the rivets or bolts. Consideration should be given to edge distance, spacing, installation clearances, and (for tension members) the effect on net area. It is usually desirable that the joint be as compact as possible.

EXAMPLE 5-1. A steel bar 5 x ¾ is to be connected by ⅞-in. rivets to a gusset plate ⅜ in. thick. The tensile load in the bar is 50 kips. Both the bar and the gusset plate have sheared edges. How many rivets are needed, and how must they be arranged to conform to the AREA specification?

$$R\text{ (shear)} = \frac{\pi D^2}{4} \times 13{,}500 = 8{,}120\text{ lb}, \qquad \textit{Controls}$$

$$R\text{ (bearing on } \tfrac{3}{8}\text{ Pl)} = 0.375 \times 0.875 \times 27{,}000 = 8{,}860\text{ lb},$$

$$n = \frac{50{,}000}{8{,}120} = 6.15. \qquad \textit{Use 7 rivets.}$$

According to the specification, the required minimum edge distance to a sheared edge is 1¾ × D, or 1.53 in., and the minimum permissible spacing is 3 × D, or 2⅝ in. We shall use for the connection an edge distance of not less than 1½ in. and a pitch of 3 in. The rivets will be placed in a single row along the centerline of the member.

EXAMPLE 5-2. Compute the maximum static tensile load allowed under the AISC specification for the double-angle member and connection shown in Fig. 5-5.

According to tables giving the properties of angles, the center of gravity of the member is 0.84 in. from the back of the angles. If the load acts along the center of gravity of the member, it is eccentric to the joint by an amount 1.75 − 0.84, or 0.91 in. If the load acts along the rivet gage line, then it is eccentric to the member by the same amount. The conditions which actually exist are probably intermediate between the two just described, so that slight eccentricities occur on both

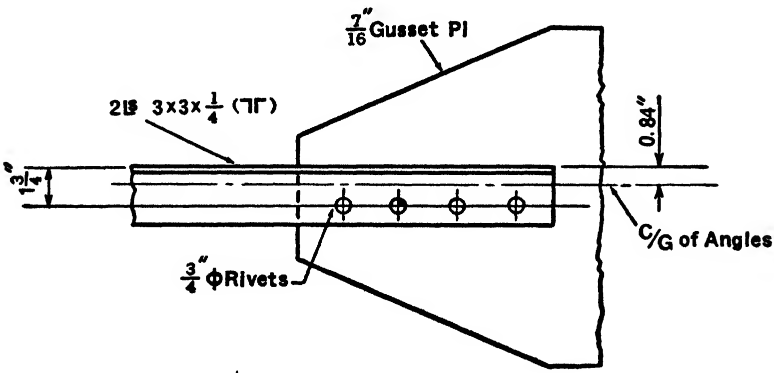


FIG. 5-5.

the member and the connection. The moments due to the eccentricities may cause slight bending of the member or slight rotation of the joint. The bending of the member would provide some relief of the eccentricity near the middle of the member length, and joint rotation would cause some relief of the eccentricity on the joint itself. Since these eccentricities are slight, and since the members are usually quite slender and the joints compact enough to permit some relief of eccentricity, it is customary in riveted work to disregard the eccentricity for angle members and their connections. However, that eccentricity should be kept as low as practical. For vibrating loads in which fatigue failure is probable, it would be best to eliminate the eccentricity entirely or to consider its effect, if it cannot be eliminated.

$$R \text{ (double shear)} = 13.25 \text{ kips}$$

$$R \text{ (enclosed bearing on } 7/16 \text{ Pl)} = 13.1 \text{ kips,}$$

$$R \text{ (bearing on two } 1/4 \text{ angles)} = 12.0 \text{ kips.}$$ *Controlling rivet strength*

The above values of R were taken from tables provided in the AISC handbook, *Steel Construction*. They can be computed in the same manner as those for Ex. 5–1, but with different allowable unit stresses.

The strength of the connection $P = nR = 4 \times 12$, or 48 kips.

The strength of the member is the product of net area and allowable unit tensile stress. Thus,

$$A_n \text{ for two angles} = 2[1.44 - (0.25 \times 0.875)] = 2.44 \text{ sq in.,}$$

$$P = 2.44 \times 20 = 48.8 \text{ kips.}$$

The maximum allowable load, controlled by the rivet group, is 48 kips. (It should be noted that the allowable unit bearing pressure for the enclosed plate is greater than for the outer plates. See Art. 5–2.)

5–7. Use of Lug Angles. To reduce the length of a connection for either single-angle or double-angle members, occasional use is made of lug angles. Figure 5–6 shows a single-angle member with a lug angle. There is some

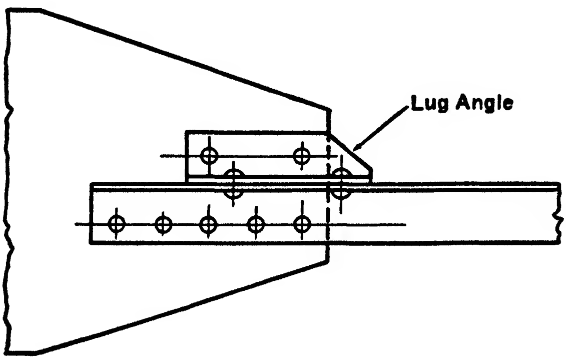


FIG. 5–6.

question as to its effectiveness. Unless the lug angle is very stiff, it will deform so that the rivets will not carry equal loads. For example, if the connection in Fig. 5–6 required exactly seven rivets, the two rivets in the lug angle might each receive less than one-seventh of the total load, and the remaining five rivets in the member itself would then be overstressed.

If it is necessary to use lug angles, certain precautions should be taken. The angle should be placed close to the end of the connection at which the stress in the member is highest and where greater movement will occur under load. The lug angle should be made as stiff as practical. It should contain as few rivets as necessary, but yet be as long as is practical so that greater stiffness will result. The total number of rivets through the gusset plate should be conservatively determined.

The presence of a lug angle on a single-angle tension member should not be construed as eliminating the bending which occurs in single-angle members connected by one leg only. Some specifications consider that angles connected to one side of a gusset plate are only partially effective. See Art. 45, Appendix B.

For tension members consisting of either one or two angles, the reduction of net area by the added row of holes must be considered. In general, the holes in a tension member should be staggered.

5-8. Welded Connections. The types of weld most frequently used for structural connections are the fillet weld, the single-vee butt weld, and the double-vee butt weld. Occasional use is made of plug welds and slot welds. The other types of weld shown in Chap. 3 are infrequently used. Calculation of their strength, however, involves no principles other than those used for the more common types.

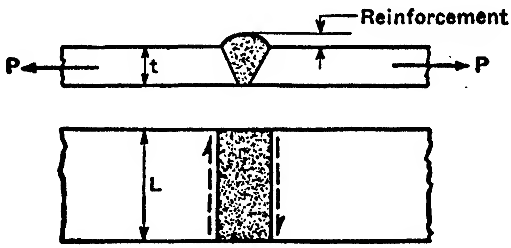


Fig. 5-7.

5-9. Butt Welds. A butt weld in shown in Fig. 5-7. The weld shown has "reinforcement," which is an additional deposit of weld metal causing the throat dimension to be greater than the thickness of the plate. Butt welds are commonly referred to as 100, 125, or 150 per cent butt welds, according to whether the throat dimension is equal to or is 25 or 50 per cent more than the material thickness.

Neglecting concentrations of stress, the tensile load shown may be assumed to be distributed uniformly over the cross section of the weld. The weakest point in the weld is at its thinnest point, so that the effective cross-sectional area is the product of the weld length L and the thickness t.

The tensile value of the butt weld in pounds per inch is equal to $f \times t$, where f is the allowable unit tensile stress for either the weld material or the member material, whichever is lower.

If the load is compressive, the value of the weld per inch is given by the same product, but with f equal to the allowable unit compressive stress for the weld or for the member material, whichever is lower.

If the joint is subject to shearing load (as shown dotted in Fig. 5–7) the weld strength per linear inch is not greater than the shear value of the plate per inch of width. The value of the weld in pounds per inch is equal to tf, in which f is the allowable unit shearing stress in either the weld material or the member material, whichever is weaker.

If strength is determined by the plate thickness, why bother to "reinforce" a butt weld?

The surface of a weld is not so smooth as that of the original material and may contain slag inclusions, etc. The probability that the strength of the joint will be affected by such irregularities is lessened by the deposit of extra metal on the joint. Further, it would require an excellent welder, indeed, to produce a flush joint having in no place a thickness less than that of the material being welded. It is much easier to make a butt weld

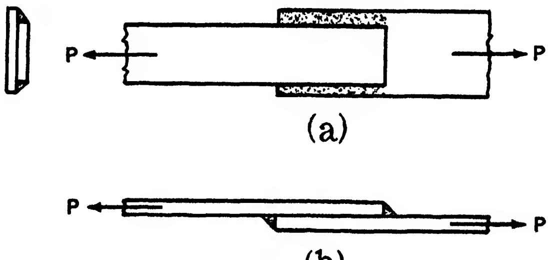


FIG. 5–8.

having reinforcement. For butt welds having static loads, or loads without vibration or frequent variation, some reinforcement is advantageous. For members having often repeated or vibrating loads, however, it is well to ensure full section by providing reinforcement and then grinding the weld flush so that its throat dimension is equal to the material thickness. If the reinforcement is not removed, a concentration of stress, which might become a nucleus for fatigue failure, will occur at the point where the bead meets the member.

EXAMPLE 5-3. Design a butt weld to join two 8 x ¾ Pls having a tensile load of 90 kips. Use the AWS allowable stresses for the weld and the AISC allowable stresses for the member.

The AWS specification states that, when used to connect structural steel for buildings, butt welds may be loaded to the same tensile, compressive, or shearing unit stress as allowed for the base (member) metal. The area of the weld is $8 \times 0.75 = 6.00$ sq in. The stress on the weld is then $90{,}000/6.00 = 15{,}000$ psi. This is less than the allowable 20,000 psi, and the connection is satisfactory. No smaller butt weld would be used. Unless it is specified otherwise on the detail drawings, the weld will have a slight reinforcement and will not be ground flush.

American Bridge Co.

BAR JOISTS FOR SUPPORT OF A FLOOR.

5-10. Fillet Welds. A fillet weld can be loaded in two ways. Figure 5-8(a) shows fillets having their axes parallel to the direction of the stress in the member. Each weld transfers load from one bar to the other by means of shearing stress parallel to its axis. Distribution of the shearing stress is assumed to be uniform over the length of the weld, although it is easily shown that uniform distribution cannot occur at stresses below the elastic limit. The maximum stress in the weld is a shearing stress at the throat and parallel to the axis. In failure, the weld should break on a longitudinal plane through the throat of the weld.

Figure 5-8(b) shows fillet welds transverse to the direction of stress. Welds of this type break through the throat as the result of combined shearing and tensile or compressive stresses. Computation of the principal stresses in the weld is complicated by many variable conditions (such as the degree of contact between the two members) in the connection. Tests show that the load-carrying capacity of a transverse fillet weld is about one-third greater than that of a longitudinal one of the same dimensions. For design purposes, it is usual to assume that the strength per linear inch of fillet is its shearing strength, regardless of the direction of load on the weld.

The design strength per linear inch of fillet weld is equal to the throat dimension multiplied by the unit shearing strength of the weld material. For a normal fillet weld the cross section is a right isosceles triangle, so that the throat dimension is the weld size W divided by $\sqrt{2}$, or $0.707W$.

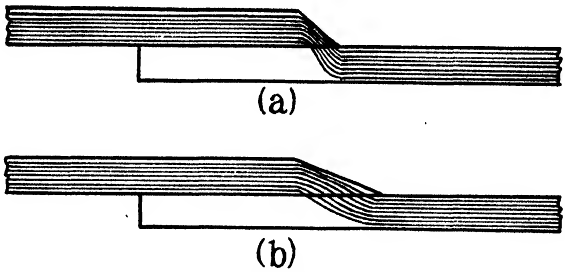


FIG. 5-9.

Using the AWS allowable unit shearing stress of 13,600 psi for weld metal on structural steel for buildings, the following allowable loads per inch of fillet weld are obtained:

(Load per inch = $13{,}600 \times 0.707W$)
For 3/16 fillet weld, 1,800 lb per in.
For 1/4 fillet weld, 2,400 lb per in.
For 5/16 fillet weld, 3,000 lb per in.
For 3/8 fillet weld, 3,600 lb per in.
For 1/2 fillet weld, 4,800 lb per in.

The student should note that with the AWS allowable stresses, the allowable load per linear inch of weld is equal to the number of sixteenths in the weld size, times 600. For example, the strength of a 3/8 weld is $6 \times 600 = 3{,}600$ lb. This relation permits the designer to remember the values.

In transferring stress from one member to another, a transverse fillet weld is subject to concentrations of stress, somewhat as shown in Fig. 5-9(a). If the load is vibratory or so often repeated that consideration of fatigue is necessary, the concentration may be reduced by use of a fillet having unequal legs, as shown in Fig. 5-9(b). In such a weld the transition from bar to fillet and from fillet to bar is more gradual, so that the stress distribution is more nearly uniform. The rough weld surface may be ground smooth to remove possible fatigue nuclei further. The designer

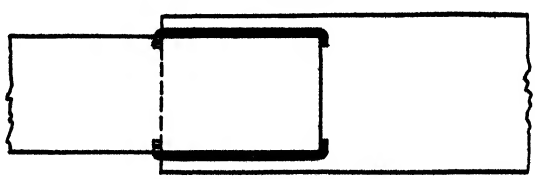


FIG. 5-10.

may call the attention of the fabricator to these precautionary measures by using appropriate standard welding symbols on the detail drawings.

The ends of a fillet weld are tapered; thus, the throat dimension at the ends is less than elsewhere. For this reason, twice the fillet size is added to the computed length of each fillet weld. The length given on detail drawings, however, is the net or computed length. The welder provides that length of full-sized section, and adds $2W$ to that length for end tapers.

Because of the previously mentioned unequal distribution of stress along a fillet placed parallel to the load, the ends of the weld are subject to more than the average shear. To reduce the stress at the ends, the fillet should, where possible, be continued for a short distance around the corner of the member, as shown in Fig. 5–10. The length of the "return" is usually not less than twice the weld size W.

5–11. Allowable and Effective Size of Fillet Weld. The heat generated by the welding process and the subsequent cooling cause expansion and shrinkage of both the original material and the weld material. If the amount of shrinkage in the two materials is unequal, initial or residual stresses occur in both the original material and the weld. Because the amount of expansion and shrinkage is controlled by the total amount of heat generated by the welding process, it is affected by the size of the fillet weld and by the thickness of the original material. Placing of a small weld on thick material does not generate enough heat to cause appreciable expansion of the original material. The weld tends to contract during cooling, but it is restrained from contracting fully by its attachment to the cooler thick material. Residual stress in the weld results. If the weld size is increased, the greater heat causes the expansion to be more nearly uniform, and less residual stress occurs.

In addition to this mechanical effect of the welding heat, there is a metallurgical effect. Steel, if very rapidly cooled from temperatures above 1500F, becomes brittle and loses its ductility by the formation of a hard crystalline substance called *martensite.* A small weld on very thick material is cooled, by conduction of heat away from the welded area through the thick base material, at a rate high enough to permit the formation of martensite. Such rapid cooling can be prevented by preheating the base material or by using a larger size of weld.

To avoid both the undesirable effects just described, the AWS building and bridge codes require that the size W of fillet welds be *not less* than those shown in Table 5–1.

The maximum size of weld is limited by three considerations. The first is the destruction of the original material by the heat generated in placing a large-sized weld with a single pass of the electrode. The maximum size weld that could be used without such destruction depends on the type of material being connected and the type of weld metal.

TABLE 5-1

Size of Fillet Weld (Inches)	Maximum Thickness of Part (Inches)
$\frac{3}{16}$	$\frac{1}{2}$
$\frac{1}{4}$	$\frac{3}{4}$
$\frac{5}{16}$	$1\frac{1}{4}$
$\frac{3}{8}$	2
$\frac{1}{2}$	6
$\frac{5}{8}$	Over 6

For structural steel, destruction of the original material is unlikely if the weld size is limited by the second consideration, efficiency. As the fillet size is increased, the amounts of weld material, current (or gas), and labor required to deposit the weld increase at a much greater rate than the strength per unit length of fillet. Therefore, from a cost standpoint, the most efficient fillet size is generally the smallest that is practical to use. (For strength purposes, welds as small as $\frac{3}{16}$ in. are permitted by AWS codes.)

To avoid excessive length of weld, larger fillets may be used, up to that size for which the strength of the welds is equal to that of the base material. An increase beyond this size is not effective, and the additional weld metal used is wasted.

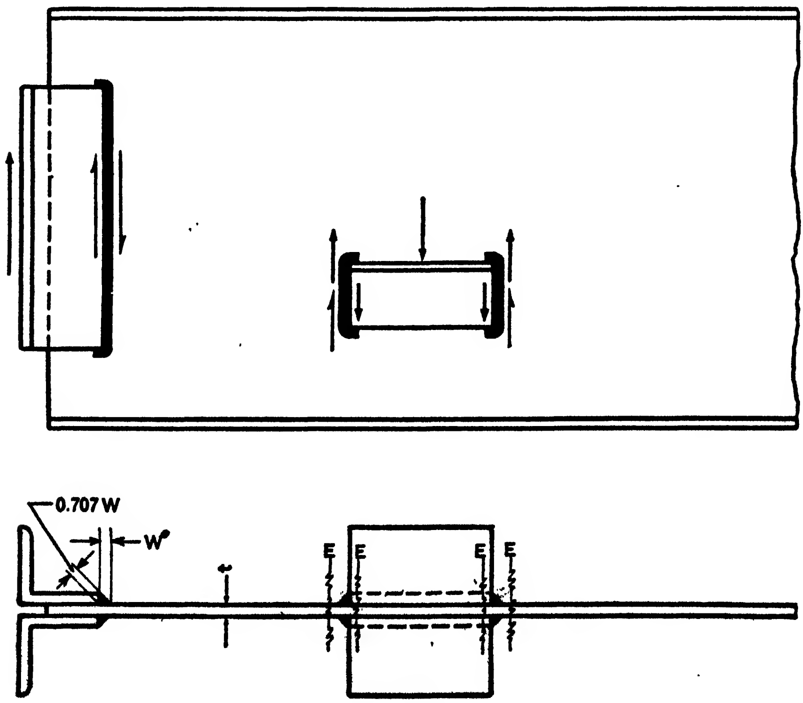


FIG. 5-11.

The beam shown in Fig. 5–11 will be used to illustrate determination of the maximum effective size of fillet weld. At the end connection the shearing stress per inch of weld is equal to that per inch height of web, or half of that amount if welds occur on both sides of the web.

$$\text{The value of two welds (per inch)} = 2W \times 0.707f_{sw}.$$

$$\text{The shear value of the web (per inch)} = tf_{sb}.$$

The terms f_{sw} and f_{sb} represent the allowable unit shearing stress in the weld and in the beam, respectively. When the maximum effective size W is used, $2W \times 0.707f_{sw} = tf_{sb}$. The maximum effective $W = (tf_{sb})/(1.414f_{sw})$.

If the weld were on one side only of the web, the maximum effective W would be twice the value given by the above equation. Substituting 13,600 for f_{sw} and 13,000 for f_{sb} in the preceding equation, the maximum effective W for steel buildings is limited to about $1.4t$ for welds on one side only of material of thickness t or to about $0.7t$ each for welds on both sides.

The connection at the interior of the beam is different in that there are two planes of shear in the web for each weld or pair of opposite welds. If the weld were loaded to produce failure in the web, there might be local yielding failure at any of the four sections marked E on the beam. For each pair of welds:

$$\text{The value of two welds (per inch)} = 2W \times 0.707f_{sw}.$$

$$\text{The shear value of the web (per inch)} = 2tf_{sb}.$$

When the maximum effective size W is used, $2W \times 0.707f_{sw} = 2tf_{sb}$. The maximum effective $W = (tf_{sb})/(0.707f_{sw})$, for welds on both sides of the web, or twice that amount for welds on one side only.

In either expression for the maximum effective weld size W, the term f_{sb} is the amount of shearing stress which the weld may cause in the web, near the edges of the weld, without overstressing the web in shear. In other words, it should be limited to the allowable shearing stress for the web minus the web shearing stress at the weld location resulting from loads other than that applied to the beam by the weld being considered.

Thus for the beam of Fig. 5–11, f_{sb} at the end connection could be taken as the full allowable shear stress for the web, since at that section no web shear exists except that caused by the load of the end weld itself. At a section marked E, however, f_{sb} would be the allowable shear for the web, reduced by the amount of web shearing stress at that section caused by other loads.

It is suggested that the weld size W for most purposes be not greater than $1.5t$, even though the equations for maximum effective W indicate a larger size.

The third consideration that may control the size of some fillet welds is the shape and thickness of the edge along which it is deposited. Since the base material melts during welding, it is difficult to produce a good fillet weld of nominal size equal to the thickness of the edge. To ensure full throat thickness for such a weld, it is necessary to build up with weld metal at rounded edges or where square edges have been melted by the welding heat. Unless it is absolutely necessary to use weld sizes equal to the thickness of the edge, the following maximum sizes are recommended:

1. For nominally square edges, $\frac{1}{16}$ in. less than the edge thickness.
2. For rounded edges, such as the toes of angles or of channel flanges, three-fourths of the nominal edge thickness.

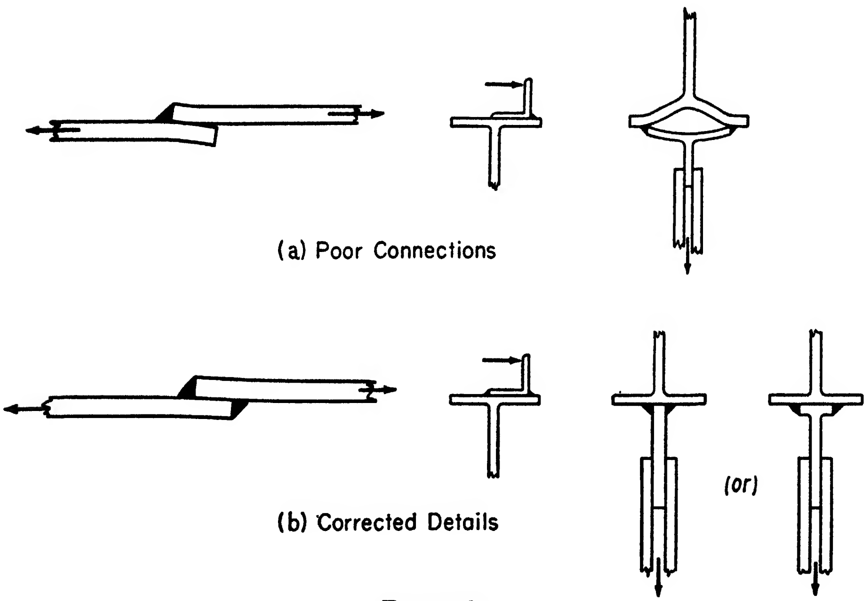


Fig. 5–12.

5–12. Effect of Bending. Fillet-welded joints should be designed so that high bending stresses do not result in the welds or the attached material. Figure 5–12(*a*) shows undesirable connections in which the fillets are subject to bending. The effect on the fillets is similar to that which would be caused by driving a wedge between the members of the connections so as to pry them apart. Welds so loaded will not develop their full rated strength. Corrected details are shown in Fig. 5–12(*b*).

5–13. Welded Connections for Unsymmetrical Axially Loaded Members. The design of connections for unsymmetrical members, such as channels or angles under axial load, is a common problem. If the axis of load coincides with the center of gravity of the member, the connecting group of welds for such a member is eccentrically loaded, unless it is balanced or so placed that its center of gravity is in line with that of the cross section of the member.

Tests* indicate that for static loads the ultimate strength of welded joints for angle members is not appreciably affected by balancing or unbalancing of the joint. As the stress in the member rises to above the elastic limit, deformations occur which relieve the slight eccentricity of load on the joint.

The tests do not, however, reveal the effect of balancing on the resistance to vibrating or often repeated loads. Fatigue failure can occur at stresses below the elastic limit. It is probable that the unequal distribution of stress on an unbalanced joint would cause the fatigue strength of the connection to be lower than that of a balanced joint. It is considered to be good practice to design the connection with the welds balanced. Ex. 5-5 illustrates the procedure for balancing welds.

EXAMPLE 5-4. What length and size of weld is required for the connection shown in Fig. 5-13? Use 13,600 psi for the allowable shearing stress in the weld.

The minimum allowable W is, by Table 5-1, $\frac{3}{16}$ in. The maximum effective W

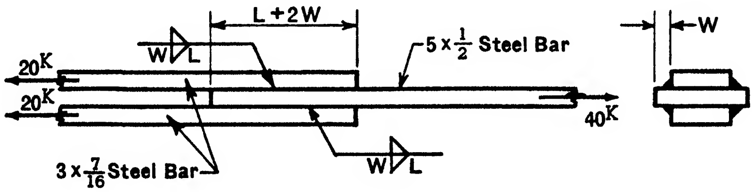


FIG. 5-13.

is $0.7 \times \frac{1}{2}$, or 0.35 in. Lengths will be determined for fillet sizes from $\frac{3}{16}$ in. to $\frac{3}{8}$ in. The load per weld is 10,000 lb.

For $W = \frac{3}{16}$, strength per inch = 1,800 lb
For $W = \frac{1}{4}$, strength per inch = 2,400 lb
For $W = \frac{5}{16}$, strength per inch = 3,000 lb
For $W = \frac{3}{8}$, strength per inch = $0.35 \times 0.707 \times 13{,}600 = 3{,}360$ lb

The net length required per weld:

$$L = \frac{10{,}000}{1{,}800} = 5.6, \text{ for } W = \tfrac{3}{16} \qquad \text{(Specify } 5\tfrac{3}{4} \text{ in.)}$$

$$L = \frac{10{,}000}{2{,}400} = 4.2, \text{ for } W = \tfrac{1}{4} \qquad \text{(Specify } 4\tfrac{1}{4} \text{ in.)}$$

$$L = \frac{10{,}000}{3{,}000} = 3.3, \text{ for } W = \tfrac{5}{16} \qquad \text{(Specify } 3\tfrac{1}{2} \text{ in.)}$$

$$L = \frac{10{,}000}{3{,}360} = 3.0, \text{ for } W = \tfrac{3}{8} \qquad \text{(Specify 3 in.)}$$

* G. J. Gibson and B. T. Wake, "An Investigation of Welded Connections for Angle Tension Members," *Welding Journal*, January 1942.

The required actual length per weld is the net length plus $2W$ to account for end tapers, so that the welder would provide over-all lengths of at least:

$6\frac{1}{8}$ in. for $\frac{3}{16}$-in. weld
$4\frac{3}{4}$ in. for $\frac{1}{4}$ -in. weld
$4\frac{1}{8}$ in. for $\frac{5}{16}$-in. weld
$3\frac{3}{4}$ in. for $\frac{3}{8}$ -in. weld

The length indicated on the detail drawings should be, using the AWS symbols, the net length. Thus, a length of $3\frac{1}{2}$ in. and a size of $\frac{5}{16}$ in. would be indicated. The welder will add to the indicated length for end tapers. If possible, the right end of each weld should be wrapped around the corner of the bar, as shown in Fig. 5–10. The $\frac{3}{8}$-in. weld would probably not be used, since it is difficult to make a good fillet weld so nearly the same size as the thickness of the edge along which it is deposited.

EXAMPLE 5–5. A 3 x 3 x $\frac{1}{4}$ steel angle is used as a tension member and is connected by welding to a $\frac{3}{8}$-in. gusset plate. Using the 1949 AISC specification, design the welded connection. Use a balanced arrangement of welds on the toe and back only.

The joint is illustrated by Fig. 5–14. The maximum load to be resisted is $P = 20{,}000 \times A = 20{,}000 \times 1.44 = 28{,}800$ lb. Call the portion of the load to be

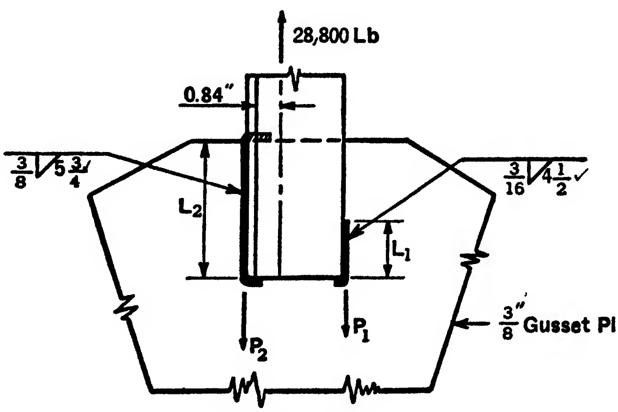


FIG. 5–14.

resisted by weld at the toe P_1, and that by the weld at the back P_2. The forces P_1 and P_2 are found by writing the equation of equilibrium, $\Sigma M = 0$, and solving. Thus,

$$P_1 = \frac{28{,}800 \times 0.84}{3.0} = 8{,}050 \text{ lb}$$

$$P_2 = \frac{28{,}800 \times 2.16}{3.0} = 20{,}750 \text{ lb}$$

$$\textit{Total } \overline{28{,}800 \text{ lb}}$$

Along the rounded edge of the angle, W cannot exceed the thickness of the angle and should preferably be not over three-fourths times the angle thickness. A $\frac{3}{16}$-in.

fillet will be used. Net $L_1 = 8{,}050/1{,}800 = 4.47$ in. Adding $2W$ for tapered ends, the gross $L_1 = 4.85$ in.

For the heel the maximum effective weld size is $1.4 \times \frac{3}{8}$ or 0.525 in. Welds up to $\frac{1}{2}$ in. could be used; the length of weld required depends on the size chosen. For a $\frac{3}{8}$-in. weld, net $L_2 = 20{,}750/3{,}600 = 5.75$ in. Adding $2W$ for end tapers, the gross $L_2 = 6.50$ in.

Note that the weld lengths shown on Fig. 5–14 are net lengths. The joint as designed above contains two different sizes of weld. If practical, all the welds should be of the same size, since the welder must stop work and change electrodes in order to produce a different size of weld. The example is given in this form, however, to show that the design procedure is as simple for a joint containing different weld sizes as for one in which all welds are of the same size.

EXAMPLE 5–6. Compute the length of fillet weld required in a balanced joint for the steel member and load shown in Fig. 5–15. Weld is to be placed on the back,

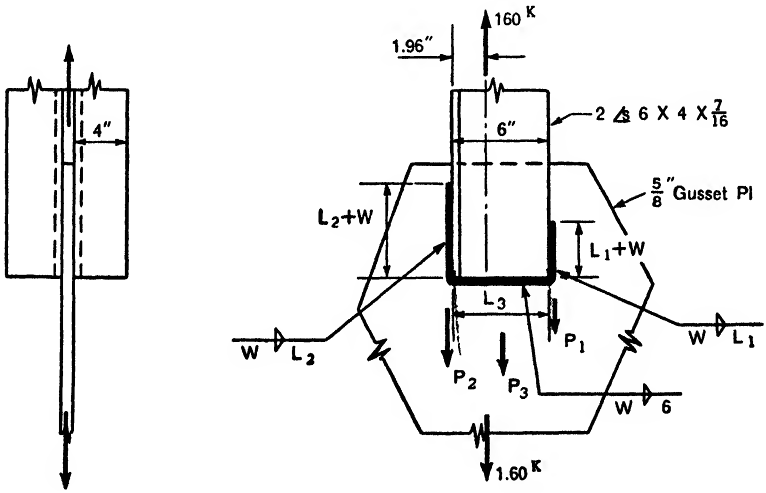


FIG. 5–15.

toe, and end of the member. Use the 1949 AISC allowable stresses for the weld and base material.

A number of solutions are possible, depending on the weld sizes used. This solution illustrates one possible combination.

The maximum effective W is $0.7 \times \frac{5}{8}$, or 0.44 in. The maximum practical W on the toe is $\frac{3}{4}$ of $\frac{7}{16}$, or approximately $\frac{5}{16}$ in. Using $\frac{5}{16}$-in. fillets on the toe and end,

$$P_1 + P_2 + P_3 = 160 \text{ kips},$$

$$P_3 = 2 \times 6 \times 3.0 = 36 \text{ kips}.$$

Taking moments about the back of the angles,

$$6P_1 + 3P_3 = 160 \times 1.96.$$

Then,

$$P_1 = 34.2 \text{ kips},$$

$$P_2 = 160 - 34.2 - 36 = 89.8 \text{ kips}.$$

The weld required is as follows:

Location	W (In.)	Value in Kips (per Inch)	Net Length in Inches (per Angle)	Gross Length in Inches (per Angle)
Toe.	$\frac{5}{16}$	3	5.7	6
Back.	$\frac{1}{2}$ (effective = 0.44)	4.2	10.7	$11\frac{1}{4}$
End.	$\frac{5}{16}$	3	6.0	6

In determining the gross lengths required, only 1 × W is added to the net length of each weld at the toe and at the back of the angles, since those welds are tapered at one end only. The welds at the end of the member have no end tapers, and the required gross length is the same as the net.

5–14. Eccentrically Loaded Shear Connections. A connection having its units acting in shear may be subject to rotating moment either with or without direct load. Usually the rotating moment is the result of a direct load applied eccentrically to the joint.

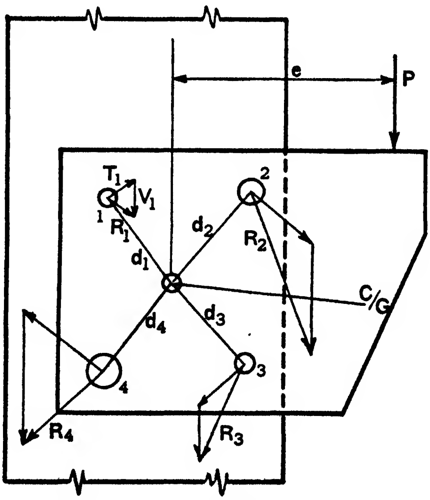


Fig. 5–16.

The load P applied to the connection of Fig. 5–16 causes a rotating moment Pe. This moment tends to rotate the connection clockwise about its center of gravity and causes a shear load in each connector unit (rivet, bolt, or unit length of weld), normal to the line drawn through that unit and the center of gravity of the connection. The amount of shear load caused in each unit depends on the amount of shearing and bearing deformation it undergoes as the result of rotation of the joint and on its stiffness, or the resistance it can provide to that deformation.

The deformation in each unit due to the rotation is proportional to the distance from the center of rotation. The resistance to deformation may be variously assumed as proportional to the cross-sectional area, the bearing area, or the ultimate strength of the unit. While none of these assumptions is exact, the last is probably the most accurate.

The terms to be used in the derivation of formulas are as follows:

S_1 = the ultimate strength of unit number one,
d_1 = the distance of unit one from the center of gravity of the connection,
T_1 = the load on unit one caused by the rotating moment Pe,
V_1 = the load on unit one caused by the direct load P without eccentricity,
R_1 = the total shearing load on the unit; the resultant of T_1 and V_1.

(For a unit other than number one, the same terms are used, but with the number of that unit replacing the subnumber one.)

If the load T is assumed proportional to the distance d and to the strength S, the following expression can be written:

$$T_2 = T_1\frac{d_2}{d_1} \times \frac{S_2}{S_1}; \qquad T_3 = T_1\frac{d_3}{d_1} \times \frac{S_3}{S_1}; \text{ etc.}$$

For equilibrium,

$$T_1d_1 + T_2d_2 + T_3d_3 + T_4d_4 = M,$$

or

$$\frac{T_1d_1^2S_1}{d_1S_1} + \frac{T_1d_2^2S_2}{d_1S_1} + \text{etc.} = M.$$

Collecting terms,

$$\frac{T_1}{d_1S_1}\Sigma(Sd^2) = M.$$

Solving for T_1,

$$T_1 = \frac{Md_1S_1}{\Sigma(Sd^2)}.$$

The term $\Sigma(Sd^2)$ may be considered as the polar moment of inertia of the strengths of the units about the center of gravity of their strengths. Since the load T for any unit depends on the *ratio* of its strength to that of another, that ratio or relative strength may be used instead of the actual strength S. Thus, the load on a connector unit due to rotating moment only is:

$$\frac{Md \times \text{relative strength } S \text{ of unit}}{J \text{ (of relative strengths of all units)}}.$$

If the joint is subject to direct load in addition to moment, the total load on any unit is the vector sum or resultant of the load due to moment and the load due to direct load. The load per unit due to direct load is assumed to be given by the following relationship:

$$V = \frac{P \times \text{relative strength } S \text{ of unit}}{\text{sum of relative strengths of all units}}.$$

The use of the equations developed is quite simple for the usual case in which the relative strength of all the units in the connection are equal. For that condition S_1, S_2, etc., may be considered as unity. Then, for a riveted connection, with rivets of equal size, the load vector due to moment is $(Pe)d/J$ where Pe is the moment about the centroid of the rivet group. The load vector parallel to the applied load is then P/n, where n is the number of rivets. Example 5-7 illustrates a simple case.

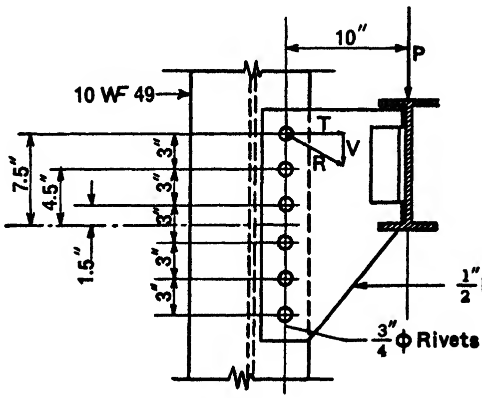


Fig. 5-17.

Example 5-7. Figure 5-17 shows a bracket connected to a column flange by rivets. The load P applied to the bracket is the end reaction of the beam. If P is 13 kips, does the connection satisfy the AISC specification?

All the rivets are of the same strength; therefore, the relative strength is unity. The controlling strength of the rivets is their shearing strength of 6.63 kips per rivet. Thus,

$$M = 13 \times 10 = 130 \text{ in.-kips},$$

$$J = 2(\overline{1.5}^2 + \overline{4.5}^2 + \overline{7.5}^2) = 157.5,$$

$$T = \frac{Md}{J} = \frac{130 \times 7.5}{157.5} = 6.19 \text{ kips (for either top or bottom rivet)},$$

$$V = \frac{13}{6} = 2.17 \text{ kips},$$

$$R = \sqrt{\overline{6.19}^2 + \overline{2.17}^2} = 6.56 \text{ kips.}$$

Since the computed load on the most highly stressed rivets is less than 6.63 kips, the connection satisfies the AISC requirements.

EXAMPLE 5-8. What is the maximum load P that can be applied to the welded connection shown in Fig. 5-18 without exceeding the AWS allowable unit shearing stress of 13,600 psi in the weld metal?

To facilitate graphical determination of the resultant stress on the weld, the illustration of the joint is drawn to scale. The weld is all of the same size and strength per inch; therefore, the relative strength S per inch is unity. Thus,

$$\bar{x} \text{ (from the left edge to center of gravity of welds)} = \frac{2 \times 4 \times 2}{8 + 4 + 4} = 1.0 \text{ in.,}$$

$$e = 6.0 \text{ in.,}$$

$$J = \left(\frac{1}{12} \times \overline{8}^3\right) + (8 \times \overline{1}^2) + (2 \times \frac{1}{3} \times \overline{1}^3) + (2 \times \frac{1}{3} \times \overline{3}^3) + (2 \times 4 \times \overline{4}^2)$$
$$= 197.4,$$

$$d = 5.0 \text{ in. (center of gravity to most highly stressed weld),}$$

$$T = \frac{Ped}{J} = \frac{P \times 6.0 \times 5.0}{197.4} = 0.152P \text{ per in. of weld,}$$

$$V = \frac{P}{16} = 0.063P \text{ per in. of weld,}$$

$R = 0.20P$ per in. of weld (obtained by graphically solving for the resultant of T and V.)

The maximum permissible load R per inch of $\frac{3}{8}$-in. fillet weld is 3,600 lb. Thus,

$$P_{max} = \frac{3{,}600}{0.20} = 18{,}000 \text{ lb.}$$

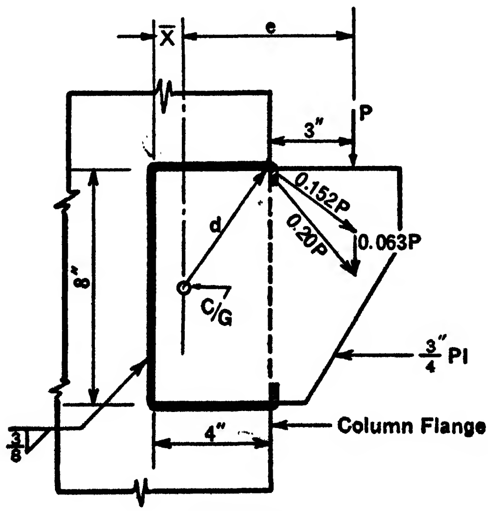


FIG. 5-18.

EXAMPLE 5-9. In Fig. 5-19 an aircraft connection is shown having bolts of two different sizes. What is the resultant shearing load on each of the bolts?

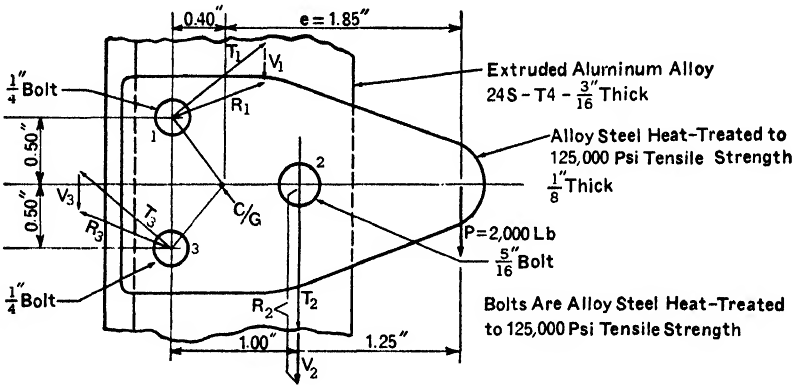


FIG. 5-19.

The publication *Strength of Aircraft Elements* (ANC-5) for 1942 gives the following ultimate strengths:

Shear on alloy steel, heat-treated to 125,000 psi tensile strength, = 75,000 psi.

Bearing on alloy steel, heat-treated to 125,000 psi tensile strength, = 175,000 psi.

Bearing on 24S-T aluminum alloy extrusions of less than 0.250 in. thickness = 83,000 psi.*

The solution of the problem is given in tabular form:

Bolt	Size	Shear Value (Lb)	Bearing Value on $\frac{3}{16}$-In.—24S-T	Relative Strength, S
1	¼	3,681†	3,890	1.0
2	$\frac{5}{16}$	5,751	4,860†	1.32
3	¼	3,681†	3,890	1.0

† Controlling value.

Bolt	S	x	Sx	x	y	Sx^2	Sy^2	d	$T = \frac{MSd}{J}$	$V = \frac{PS}{\Sigma S}$	R (Scaled)
1	1.00	0	0	0.40	0.50	0.160	0.250	0.64	1,830 lb	602 lb	1,530 lb
2	1.32	1	1.32	0.60	0.00	0.475	0.000	0.60	2,260	796	3,056
3	1.00	0	0	0.40	0.50	0.160	0.250	0.64	1,830	602	1,530
Σ	3.32		1.32	0.40		0.795	0.500				

Thus,

$$e = 2.25 - 0.40 = 1.85 \text{ in.},$$

$$M = 1.85 \times 2{,}000 = 3{,}700 \text{ in.-lb.},$$

$$J = 0.795 + 0.500 = 1.295$$

* Designation 24S-T changed to 2024-T4, October 1, 1954.

5–15. Design of Eccentrically Loaded Shear Connections. Equations can be developed for direct solution of the required length of weld, or number of rivets or bolts, in eccentrically loaded shear connections. In the usual case, however, solution by successive approximation is quicker

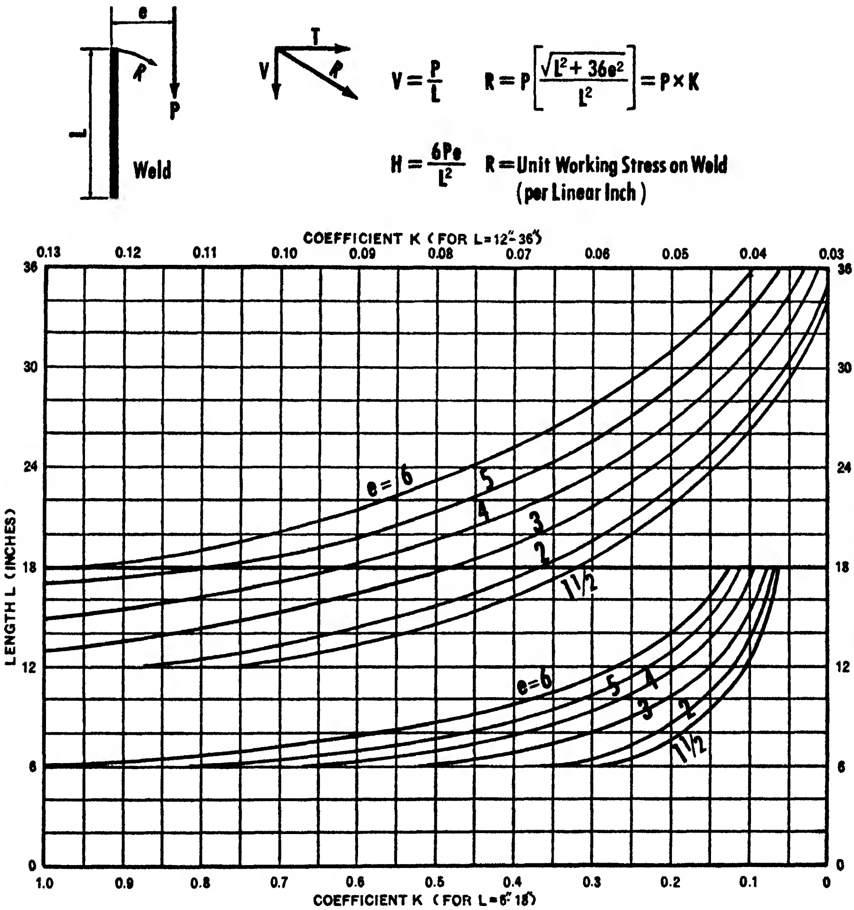


FIG. 5–20. Eccentric connections. [Source: H. Malcolm Priest, *The Practical Design of Welded Steel Structures* (New York: American Welding Society, 1943).]

and easier than using the equations. The weld pattern, or number of bolts or rivets, is estimated. Then an analysis is made on the estimated connection to determine whether the resultant load on any connector unit is beyond the amount permitted by the specification. If any unit is overstressed, the number of units is increased, or their arrangement is changed to provide greater resistance to the rotating moment. If the computed stress is far below the allowable, the number of units may be reduced. The connection is revised and recomputed until a satisfactory arrangement and number of units is determined. While this sounds like a long and cumbersome process, it is seldom so. Rarely are over three trials required; two are usually sufficient.

Direct solution for a single fillet weld having an eccentric load parallel to its axis is made possible by an equation and a set of curves given by H. M. Priest.* The equation and curves are reproduced in Fig. 5–20, but with the symbols changed to conform with those of this text. To use the curves in design, it is necessary to know only the eccentricity e, the load P, the size of fillet to be used, and the allowable load R per inch of weld. Solve for K (K required $= R/P$) and enter the graph with known values of K and e. The net length required is read directly from the scale at

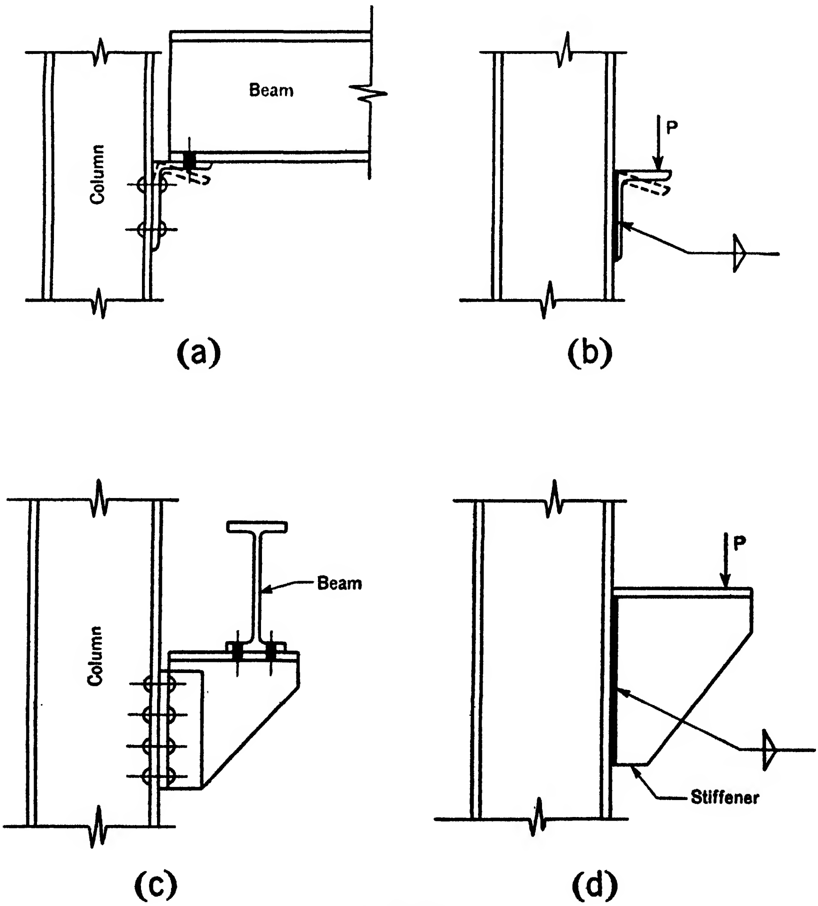


Fig. 5–21.

* H. Malcolm Priest, *The Practical Design of Welded Steel Structures*, (New York: American Welding Society, 1943).

either the right or left end. The upper set of curves is used when K is between 0.03 and 0.13, the lower set when K is from 0.07 to 1.0.

5–16. Seat Connections. A few of the more common types of beam seat or bracket are shown in Fig. 5–21. They are used to support beams from columns or from the webs of girders. The supported beam may be in the position shown in Fig. 5–21(*a*) or that shown in (*c*). While many minor variations are possible, beam seats may be classified as either "unstiffened" or "stiffened."

Unstiffened seats are shown in parts (*a*) and (*b*) of Fig. 5–21. In an unstiffened seat, resistance to deflection of the horizontal leg of the seat is provided only by the flexural strength of the leg itself. Consequently, unstiffened seats are suitable only for relatively small loads.

For heavier loads, stiffened seats such as shown in Fig. 5–21(*c*) or (*d*) are required. Deflection of the horizontal leg of the seat is prevented by the stiffener, sometimes called the diaphragm.

Seat connections for simple beams can fail in three main ways. The beam itself can fail through web crippling. The seat material may fail through bending of the horizontal leg, or through crippling or buckling of the stiffener. Finally, the connection joining the seat to the supporting column or girder may fail.

The design of a beam seat consists merely of proportioning the seat and its connection so as to prevent the above three types of failure and to meet certain practical requirements. The two steps in the design are:

1. Choose the seat dimensions (to avoid the first and second types of failure and to meet practical detail requirements).
2. Choose the connection detail, number of rivets, weld length, etc.

These two steps in the design procedure are explained more fully in the next six articles. Two articles cover the choosing of the seat dimensions; one article pertains to unstiffened seats and the other to stiffened seats. Four articles deal with the connections; one each with riveted and bolted connections and two with welded connections.

5–17. Dimensions of Unstiffened Seats. The first step is to select the width A, as in Fig. 5–22(*a*), for the horizontal leg of the seat. A "setback" or clearance c (usually ½ in.) is provided at the end of the beam. The length of contact between the beam and the seat is A–c. This length of contact must be at least as large as the bearing length b required to prevent crippling of the web of the beam. (See Art. 4–24.)

Dimension A, as computed to satisfy the above requirement, is often small. Other considerations then control the selection of the outstanding leg width. The contact length $(A-c)$ must obviously be large enough to permit the installation of field rivets, bolts, or welds joining the supported beam to the seat. Dimension A should also be large enough so that field erection will be easy; a slight mislocation of the beam must not result in dropping the beam. Finally, dimension A should, when possible, be that of a standard available angle.

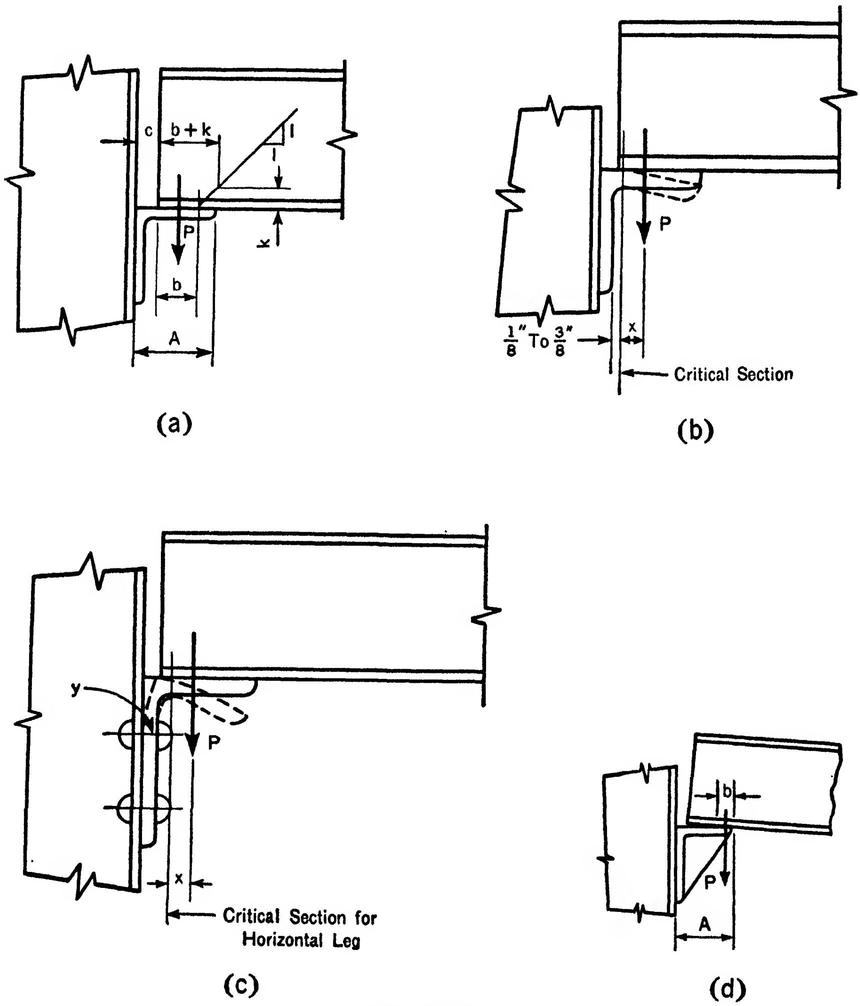


Fig. 5–22.

Having selected the width of the outstanding leg, the designer now should select the thickness of the angle and the length D (perpendicular to the plane of the paper in Fig. 5–22). To do this, the action of the seat under load must be considered. This action is shown by Fig. 5–22(b) for a welded seat and by Fig. 5–22(c) for a riveted or bolted seat. In each case the beam reaction causes load P to act on the seat. The seat deflects under the load P, causing the center of pressure between the beam and the seat to shift toward the end of the beam. For unstiffened seats, with the beam positioned as in Fig. 5–22, the resultant pressure P is assumed to be $b/2$ from the end of the beam, where b is the length required to prevent web crippling. (But, if the required distance b is less than dimension k, the resultant pressure P is assumed to act at a distance $(b + k)/4$ from the end of the beam.)

The thickness t and length D are chosen to avoid overstress by the bending shown in Figs. 5–22(b) and (c). Various assumptions are made as to the location of the critical section in bending. For the seat angle of Fig. 5–22(b), the bending moment increases as the distance from the right edge increases, becoming maximum next to the face of the vertical leg. The section modulus is constant from the right edge until the fillet is reached, beyond which point it increases sharply. For points to the right of the fillet, the bending stress in the angle is My/I; but for points in the fillet region, the stress on the lower fiber is affected by the slope of the fiber and exceeds My/I. If all three variables (moment, section modulus, and slope) are considered, it is evident that the bending stress is maximum somewhere in the region of the fillet. Distances of $\frac{1}{8}$ in. and $\frac{3}{8}$ in. from the face of the vertical leg have been suggested as standard assumptions for the location of this critical section. Since the critical section lies slightly to the left [Fig. 5–22(b)] of the end of the fillet, the use of the $\frac{3}{8}$-in. dimension would be suitable for angles having fillet radii greater than say $\frac{7}{16}$ in. For those of smaller radius, a lower dimension would be preferable. As most structural steel angles of sizes suitable for beam seats have larger fillet radii, the $\frac{3}{8}$-in. dimension is most commonly used. (Equal-leg angles of 4 x 4 size or over and unequal-leg angles of 5 x $3\frac{1}{2}$ size or over have fillet radii large enough to permit the use of the $\frac{3}{8}$-in. distance as a standard.)

To compensate for neglecting the slope of the bottom fibers at the assumed critical section, the nominal thickness of the angle is used in computing the section modulus of the outstanding leg. While the outstanding leg is assumed to act as a cantilever, partial restraint to such action is given by the welded or bolted attachment of a beam to the outstanding leg in the position shown in Fig. 5–22. Thus, the bending moment at the critical section is less than computed. For this reason, and to ensure flexibility, the computed bending stress is usually allowed to be higher than the bending stress for beams. For example, the *AISC Manual* (not the specification) recommends the allowable stress to be 24,000 psi for the seat.

For a riveted or bolted seat, the critical section is in the connected leg at the top line of rivets or bolts (point y on part (c) of Fig. 5–22). It would seem proper that the angle should be thick enough and wide enough to prevent failure at this point by combined direct load and bending. Yet, the *AISC Manual* shows the angle thickness as determined by the requirements for the horizontal leg. This approximate method of design is apparently justified by the restraint mentioned in the preceding paragraph.

Having computed the moment at the assumed critical section as $M = Px$, the dimensions D and t are now selected. The length D is controlled only by detail considerations. It should be at least as large as the flange width of the supported beam. In cases where field welding is to be used, the length of the seat angle D should be preferably about 2 in. more than the beam flange width so as to permit flat-position welding, and also of the length required for attachment to the column.

With this step, all dimensions are now known except the vertical-leg width. That dimension is determined when the connection of the bracket to the supporting member is designed.

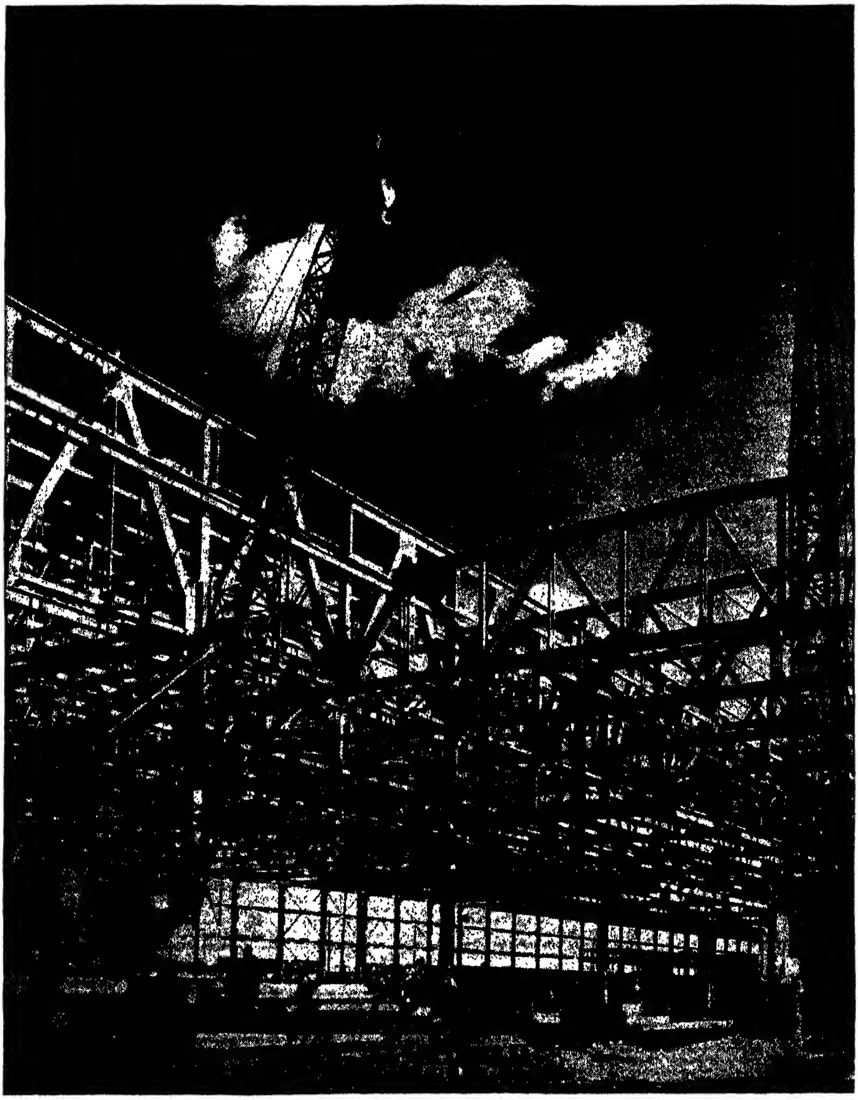
5–18. Dimensions of Stiffened Seats. As for the unstiffened seat, the first step in the design of a stiffened seat is to select dimension A, as shown in Fig. 5–22(d). Sufficient bearing length b must be provided, and detail requirements for clearance, field connections, and easy erection must be met.

In order to proceed, the location of the resultant load on the seat must be considered. As the end of the beam rotates, the center of pressure between the beam and the seat shifts toward the outer edge of the seat, as shown by Fig. 5–22(d). The resultant load P is assumed to be located at the center of the required bearing length b. Thus P is located at a distance $b/2$ from the edge of the seat.

The stiffener thickness is selected next. There is no rational method for designing the stiffener. The methods now in use are approximate only. One of the methods used assumes that the stiffener is a vertical compression member. The two requirements under that method are:

1. The stiffener should be considered as a compression member subject to an eccentric load. By equating the combined stress $P/A + Mc/I$ for the outer edge to the stress allowed for bearing of stiffeners, the required thickness can be computed. In no case, though, should the thickness used be less than the web thickness of the supported beam.
2. To prevent local buckling of the outer edge, the ratio of stiffener width to thickness should be limited according to the specification requirements for outstanding elements in compression.

In another method the entire seat is assumed to act as a horizontal cantilever beam. This action is not likely for brackets whose depth vertically is much larger than their horizontal length A. When the horizontal dimension is larger, however, it would be well to use this second method. The stress computed for the lower edge must be adjusted for the slope of that edge. (See Art. 4–35.) The edge of the stiffener can be stiffened



Bethlehem Steel Co.

CONSOLIDATED AIRCRAFT BOMBER PLANT, FT. WORTH, TEXAS.

when necessary by connecting angles or a plate to it in the position of a flange.

In order to use the second method above, the designer must either know or assume the vertical dimension of the bracket. Normally, though, this dimension can be left till last. It is selected when the connection of the seat to the supporting member is designed. The next four articles describe the design of varieties of such connections.

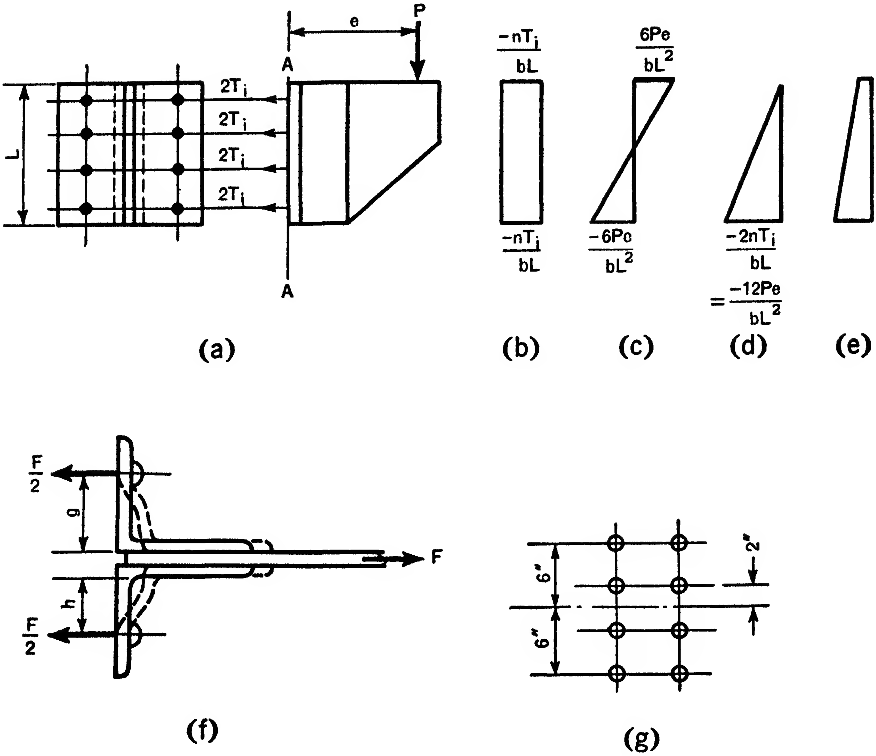


FIG. 5–23.

5–19. Bracket Connections, Riveted. The design method shown in this article is for use with connectors having a reliably high initial tension. Thus, it could be used for brackets connected by hot-driven steel rivets or adequately tightened high-tensile steel bolts.

Figure 5–23(*a*) shows such a bracket. The rivets each have an initial tension T_i. These tensions cause compression between the bracket and the supporting member. The initial compressive stress is nT_i/bL, in which n is the number of rivets, b is the width of the contact surface, and L is the length of the connection. The initial compressive stress is assumed to be uniformly distributed, as shown by Fig. 5–23(*b*).

When the load P is applied a moment Pe occurs at section A–A. This moment increases the intensity of compression between the contacting surfaces below their center of gravity and reduces it above. Until the moment becomes large enough to reduce the compression between the surfaces to zero and then to cause the surfaces to separate, no appreciable elongation of the rivets will occur. Thus, no appreciable change of tensile stress in the rivets will occur provided that compressive stress remains between the entire areas of the contact surfaces at line A–A.

The change in compressive stress caused by the moment Pe is Pey/I. At the upper edge y is $L/2$. The moment of inertia I is $bL^3/12$. Thus, the change of compressive stress at the top or bottom edges is $6Pe/bL^2$, as shown in Fig. 5–23(c). When the compression reduction due to moment does not exceed the initial compression, the combined compression is distributed as shown by the trapezoidal diagram of Fig. 5–23(e) or the triangle of (d).

Measured values of T_i indicate initial tensile stresses in hot power-driven rivets in the order of 24,000 psi.* Rivets having this initial tension are required to carry the full shear load permitted by the specification. It does not seem reasonable to require such rivets to resist the full shear load and at the same time an increase of tensile stress beyond their initial tension. To avoid this increase, it is merely necessary to limit the moment Pe, or to proportion the connection so that compression remains over the entire contact surface. The limiting condition is that shown by the triangular distribution of Fig. 5–23(c), for which

$$\frac{6Pe}{bL^2} = \frac{nT_i}{bL}, \quad \text{or} \quad 6Pe/L = nT_i.$$

This may be solved for the product nL.

$$nL \text{ required} = 6Pe/T_i$$

Various combinations of joint length L and number of rivets n can be solved from this equation. The combination to be selected will depend on detail requirements of spacing, etc., and on appearance of the completed bracket. In order to provide a factor of safety with respect to reaching the limiting triangular stress distribution, it is recommended that an initial tension of 20,000 psi be used in computing T_i (or, for high-tensile bolts, one-half of the specified initial tensile stress).

When the rivet pitch s is known in advance and when end distances of $s/2$ are to be used, $n = 2L/s$, so that

$$L \text{ required} = \sqrt{\frac{3Pes}{T_i}}$$

In addition to the length of the angles, their thickness must be determined. This dimension may have much effect on the design.

When a force F is applied to a pair of angles in the manner shown in Fig. 5–23(f), a bending moment is produced in the legs of the connection angles. The amount of the bending moment in each leg is approximately $Fh/4$, in which h is the gage on the angle minus the thickness of the angle. The force F is maximum at the upper edge of the angles of a bracket and may be considered equal to the change of compression due to the moment

* Wilbur M. Wilson and John V. Coombe, "Fatigue Tests of Connection Angles," *University of Illinois Engineering Experiment Station Bulletin*, No. 317 (October 1949).

Pe. Thus, for the upper edge, the force F per inch of connection angle is b times the extreme fiber stress due to moment, or $F = 6Pe/L^2$.

It is recommended that the bracket connection provide for:

1. Sufficient rivets for shear at the allowable design stress.
2. Sufficient length of connection and number of rivets so that the initial tension in the rivets will not be exceeded.
3. Angle thickness such that the bending stress in the legs of the angles will not exceed the allowable bending stress.

EXAMPLE 5–10. Design the connection of Fig. 5–23(a) for a load of 20 kips when e is 10 in. Use ⅞-in. rivets with 4-in. pitch and AISC allowable stresses. Assume that rivets have an initial tension of 20,000 psi.

The number of rivets required for shear is

$$\frac{20,000}{0.6 \times 15,000} = 2.22.$$

Four rivets would then provide amply for the shear.

The length of the connection angles may be solved directly.

$$L = \sqrt{\frac{3Pes}{T_i}} = \sqrt{\frac{3 \times 20,000 \times 10 \times 4}{0.6 \times 20,000}} = 14.14 \text{ in.}$$

The number of rivets to satisfy the moment requirement will then be $\dfrac{14.14}{4} = 3.54$ in each row. The actual length of the connection will then be that required for four rivets in each row, or 16 in., with a 2-in. end distance for the top and bottom rivets.

$$\text{The value of } F = \frac{6Pe}{L^2} = \frac{6 \times 20,000 \times 10}{16 \times 16} = 4,690 \text{ lb per in.}$$

The standard gage for a 4-in. leg is 2½ in. Assuming that the angles will be ¾ in. thick, $h = 1.75$ in. The bending moment per inch of height of connection angle will then be,

$$\frac{4,690 \times 1.75}{4} = 2,050 \text{ in.-lb.}$$

The required section modulus is 2,050/20,000 = 0.103 in.^{3} and the thickness of the angle is $t = \sqrt{6 \times 0.103} = 0.78$ in. The design requirements will be satisfied with 4 x 4 x ¾ angles 1 ft 4 in. long, with one row of four rivets on standard gage in each angle.

It should be noted that the use of 20,000 psi for initial tension is conservative when compared with the probable tension equal to yield-point stress (see Art. 5–3). It is also less than the 24,000 psi values that have been reported by the University of Illinois. Since the AISC recommends a value of 20,000 psi for tension in rivets, it appears consistent to use that value for a working stress.

EXAMPLE 5–11. The riveted connection of Ex. 5–10 will now be computed by a second method. This second method is quite popular; the authors, however, do not recommend its use because it is based on an impossible type of action for the rivets. In that method it is assumed that the area of the rivets, Fig. 5–23(g), acts as the section resisting bending moment. This assumption would imply that the rivets below the neutral axis are in compression and those above the neutral axis in tension because of the bending. The initial tension in the rivets is neglected and, obviously, the rivets cannot be loaded in compression. By this method of analysis, the moment of inertia of the rivet group of Fig. 5–23(g) would be

$$\begin{aligned} 4 \times 0.6 \times 6^2 &= 86.4 \\ 4 \times 0.6 \times 2^2 &= \underline{9.6} \\ I &= 96.0 \text{ in.}^4 \end{aligned}$$

The tension in the top rivet is assumed to be $Mc/I = Pec/I$, or

$$\frac{20{,}000 \times 10 \times 6}{96.0} = 12{,}500 \text{ psi, or } 7{,}500 \text{ lb per rivet.}$$

The shear would be

$$\frac{20{,}000}{8 \times 0.6} = 4{,}170 \text{ psi, or } 2{,}500 \text{ lb per rivet.}$$

The combined stresses are,

$$\text{Maximum shear} = \sqrt{6{,}250^2 + 4{,}170^2} = 7{,}510 \text{ psi},$$

$$\text{Maximum principal stress} = 6{,}250 + \sqrt{6{,}250^2 + 4{,}170^2} = 13{,}760 \text{ psi tension.}$$

Since both tension and shear stresses are below the allowable stresses, it would seem that fewer rivets could be used. However, the results of Ex. 5–10 indicate that if fewer rivets were used the actual tension would exceed 20,000 psi.

5–20. Bracket Connections, Bolted. Consider a seat of the type shown in Fig. 5–21(c), having negligible initial tension in the bolts. This condition can be produced by tightening the nuts just sufficiently to cause the connected materials to touch. If it is certain that this condition exists, the following method of analysis may be used. (There is a possibility that it might be safely applied also to joints having initial tension, since deformation of the bracket material causes the neutral axis to move toward the position shown in the following discussion. There is enough uncertainty, however, that the more conservative analysis of Art. 5–19 should be used unless the initial tension is negligible.)

Rotation of the bracket of Fig. 5–24 causes tension in the rivets or bolts above the neutral axis and compression between the bracket and the column on the contact surface below the neutral axis. The amount of unit stress at any point due to rotation is assumed to be expressed by the flexure formula, $f = My/I$. (The flexure formula is not strictly applicable because the contact surface warps as the bracket rotates.)

The moment of inertia used is that of the areas in compression and tension about the neutral axis. The neutral axis is at the center of gravity of the acting areas. The tensile areas are the gross areas of the bolts or rivets above the neutral axis. The height of the compression area is the distance from the lower edge of the bracket to the neutral axis. The width of the compression area is sometimes taken as the width of the bracket flange. Because of deformation of the flanges, however, the compressive unit stress at the edges of the flange is less than that indicated by the flexure formula. To include the effect of such deformation, an effective width b may be used

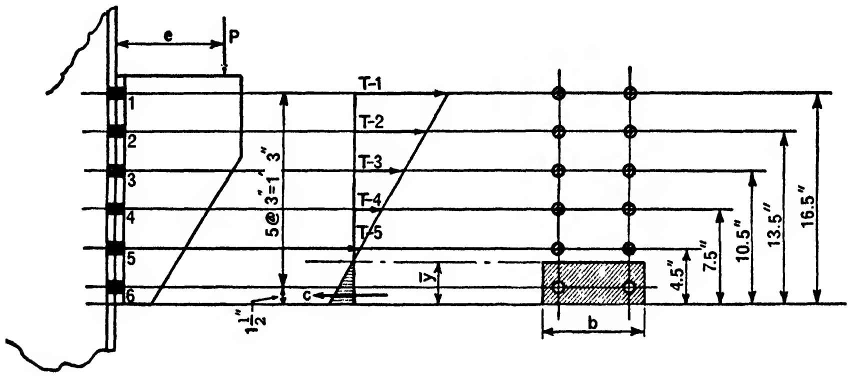


FIG. 5-24.

instead of the entire width of the bracket flange. An analysis by R. E. Green* indicates that the effective width b for connections in which the flange consists of a pair of angles is approximately twice the angle thickness plus five-eighths of the width of the outstanding legs. The analysis also shows that, because of bending of the flange, the width of contact may be less than the entire width of flange. The contact width is a function of the thickness and elastic properties of the material in both the flange and the structure to which it is connected.

It is probably sufficiently accurate for practical design purposes to use for the effective width b five-eighths of the entire width of contact before rotation.

To determine the distance $\bar{y}$ to the neutral axis, it is necessary to assume its approximate location and the number of rivets in tension. This procedure will be illustrated in Ex. 5-12.

After the tensile and shear loads have been determined for the top rivets or bolts, principal and maximum shear stresses should be computed, or the specification requirements for combined loading checked to determine whether the connection is satisfactory.

EXAMPLE 5-12. What is the maximum principal stress and the maximum shearing stress in the top bolts of Fig. 5-24, when P is 42 kips, e is 8 in., the flange width is

* Thesis by R. E. Green at Fenn College, Cleveland, Ohio, 1938.

8 in., and the bolts are all ¾-in. in diam? The nuts are tightened to produce no appreciable initial tension.

The area per bolt = 0.442 sq in., and b (effective width) = $5/8 \times 8.0 = 5.0$ in.

Assuming that the neutral axis lies between the first and second pairs of bolts from the bottom,

$$\bar{y} = \frac{\Sigma \text{ moments of acting areas (about bottom edge)}}{\Sigma \text{ acting areas}}.$$

Item	A	y	Ay
Compressive area..................	$5\bar{y}$	$\bar{y}/2$	$2.5\bar{y}^2$
10 bolts (above neutral axis).......	4.42	10.5	46.4

Then

$$(5\bar{y} + 4.42)\bar{y} = 2.5\bar{y}^2 + 46.4.$$

Solving the above equation, $\bar{y} = 3.52$ in. As the neutral axis lies between the first and second pairs of bolts from the bottom, the value just determined for $\bar{y}$ will be used to compute I, the moment of inertia of the acting areas about the neutral axis. The areas of the 13/16-in. bolt holes in the compressive area below the neutral axis were neglected in computing $\bar{y}$, since their effect on $\bar{y}$ is slight. They will be considered in computing I.

Item	A	d	Ad^2	I
Compressive area.....	17.60	1.76	54.6	18.2
Holes No. 6..........	−1.04	2.02	−4.2	0
Bolts No. 1..........	0.884	12.98	149.0	0
2..........	0.884	9.98	88.0	0
3..........	0.884	6.98	43.0	0
4..........	0.884	3.98	14.0	0
5..........	0.884	0.98	0.8	0
			345.2	18.2

Then

$$I = 345.2 + 18.2 = 363.4 \text{ in.}^4$$

For each top bolt (No. 1),

$$T = \frac{42 \times 8 \times 12.98 \times 0.442}{363.4} = 5.31 \text{ kips},$$

$$V = \frac{42}{12} = 3.50 \text{ kips}.$$

The threads of the bolt are located near the nut only, so that the maximum stress in the threaded section is not affected by the shearing load V. Because of stress concentrations at the root of the threads, the principal stress there may be higher than the average tensile stress in the threaded portion. The approximate stress-concentration factor is 2.5 for U.S. Standard threads.*

For the threaded portion,

$$f_t \text{ (average)} = \frac{5{,}310}{0.302} = 17{,}600 \text{ psi},$$

$$f_t \text{ (probable max)} = 2.5 \times 17{,}600 = 44{,}000 \text{ psi}.$$

For the unthreaded portion, at the shear plane,

$$f_t = \frac{5{,}310}{0.442} = 12{,}000 \text{ psi},$$

$$f_s = \frac{3{,}500}{0.442} = 7{,}920 \text{ psi},$$

$$f_t' \text{ (max principal stress)} = \tfrac{1}{2}f_t + \sqrt{(\tfrac{1}{2}f_t)^2 + f_s^2} = 15{,}940 \text{ psi},$$

$$f_s' \text{ (max shear stress)} = \sqrt{(\tfrac{1}{2}f_t)^2 + f_s^2} = 9{,}940 \text{ psi}.$$

To determine whether the connection is satisfactory it would be necessary to compare the computed stresses with those allowed by the applicable specification. Most specifications quote an allowable *average* tensile stress at the root of the threaded portion, so that for static loads, the effect of stress concentration need not be computed. For vibrating or frequently repeated loads, however, consideration of stress concentration may be necessary. (See Chap. 10.)

The present specifications do not give allowable values of the principal and maximum shearing stresses resulting from combined loading. In the absence of such information, it is considered safe to use the same allowable values as given for tension alone and shear alone. Thus, a bolt under combined loading cannot be loaded to its working capacity in either shear or tension without being overstressed.

EXAMPLE 5–13. Is the connection of Ex. 5–12 and Fig. 5–14 satisfactory under the AISC allowable stresses?

In Ex. 5–12 the following bolt loads were computed: $T = 5.31$ kips, and $V = 3.50$ kips. The resultant of these loads is

$$\sqrt{\overline{5.31}^2 + \overline{3.50}^2} = 6.36 \text{ kips}.$$

The AISC specification allows the following unit stresses for bolts:

Tension, 20,000 psi,

Shear, for turned bolts in reamed holes, 15,000 psi,

Shear, for unfinished bolts, 10,000 psi.

The shear values, being lower, are used as the controlling values. Multiplying by the shank area (0.442), the allowable shear loads are 6.63 kips for turned bolts in reamed holes or 4.42 kips for unfinished bolts. Thus, under the AISC specification, the connection is satisfactory with turned bolts in reamed holes, *provided the condition of no initial tension can be ensured.*

* F. B. Seely, *Resistance of Materials* (New York: John Wiley & Sons, 1947).

5–21. Welded Connections, Unstiffened Brackets. The welds connecting the seat shown by Fig. 5–25 are subject to vertical shear stresses and to tensile or compressive stresses due to the moment Pe. It is usually assumed that the stress due to moment is given by the flexure formula. If the flexure formula holds, the intensity of horizontal shear (and therefore vertical shear also) is $PQ/2I$, as shown in Fig. 5–25. However, certain

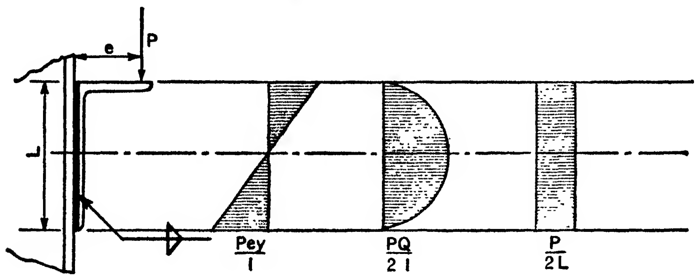


FIG. 5–25.

conditions assumed in the derivation of the flexure formula are not found here. The depth of the section in bending is L at the weld and increases sharply at the column. This change causes the stress at the upper and lower extremities to exceed the value given by the flexure formula.* Accurate computation of the stress in the weld is difficult, but the following approximation has been used with success:

Compute the stress T per inch of weld by the flexure formula. Assume the shear load P to be uniformly distributed over the entire length of weld, so that V equals $P/2L$. The vector resultant stress R at the ends of the welds is then equal to $\sqrt{T^2 + V^2}$.

To design such a connection, the procedure of Art. 5–15 and the curves of Fig. 5–20 may be used. It should be noted that the curves of Fig. 5–20 are for one weld of length L. To use the curves for determining the length L for each weld of a beam seat, assume each weld to resist one-half the total seat load. The value of L read from the curve will then be the net length of one weld.

It is good practice to continue the welds a short distance, say ½ in., around the upper end of the seat so as to relieve the concentration of stress that may occur at that point.

EXAMPLE 5–14. Design an unstiffened welded seat of the type shown by Fig. 5–22(*a*) to support a 10 W 21 beam having an end reaction of 15 kips. Use the AISC and AWS specifications.

The width of the seat, transverse to the beam, should preferably be about 2 in. more than the flange width of the beam so as to allow for flat-position field welding of the beam to the seat. A transverse width of 8 in. will be used.

* F. B. Seely, *Advanced Mechanics of Materials* (New York: John Wiley & Sons, 1949).

(*a*) *To determine*: The width of seat in a direction parallel to the beam.
The bearing length b required to prevent web crippling is

$$\frac{P}{24t_w} - k = \frac{15}{24 \times 0.24} - 0.69 = 1.91 \text{ in.}$$

Width A required $= 1.91 + 0.50 = 2.41$ in. min. The exact width A to be used will be determined after the thickness and vertical leg dimension are computed.

(*b*) *To determine*: The required thickness of angle (see Art. 5–17).

$$e = 0.50 + \frac{1.91}{2} = 1.45 \text{ in.}$$

Assuming the angle thickness to be ½ in., the distance from P to the critical point x of Fig. 5–22(*b*) is 0.58 in. Thus,

$$M_x = 0.58 \times 15 = 8.7 \text{ in.-kips.}$$

Using an allowable flexural stress of 24,000 psi,

$$S \text{ required} = \frac{8.7}{24} = 0.362 \text{ in.}^3 = \frac{1}{6} \times 8t^2,$$

$$t \text{ required} = \sqrt{\frac{6 \times 0.362}{8}} = 0.522 \text{ in.}$$

A 9/16-in. thick angle will be used.

(*c*) *To determine*: The weld size and length.

Assume that a 6 x 4 x 9/16 angle will be used, with the 4-in. leg horizontal. The length of weld to the column will then be 6 in. The load per inch of weld due to the vertical force will be $P/L = 15/6 = 2.5$ kips. The load per inch-length of weld at the ends of the weld due to moment will be

$$M/S = 6Pe/L^2 = \frac{6 \times 15 \times 1.5}{6 \times 6} = 3.75 \text{ kips.}$$

The resultant load per inch of weld height will be $\sqrt{(2.5)^2 + (3.75)^2} = 4.51$ kips (on two welds). The value of two ¼-in. welds is 4.8 kips per in. The detail will then be an angle 6 x 4 x 9/16 x 8, with the short leg horizontal and with two ¼-in. welds 6 in. long.

(*d*) *Alternate solution for weld size*: Refer to Fig. 5–20.

Enter the curve of Fig. 5–20 at a length of 6 in. to the intersection with $e = 1.5$. Reading at the bottom of the curve, $K = 0.3$.

Then, since $R = P \times K$, $R = 15 \times 0.3 = 4.5$ kips per in. for two welds.

5–22. Welded Connections, Stiffened Brackets. The welds for a seat with a single stiffener are usually of the general shape shown in Fig. 5–26. The exact proportions may vary from those shown, but a length of horizontal weld equal to $0.2L$ is generally satisfactory.

Again, the resultant stress at any point is $\sqrt{T^2 + V^2}$, and V is assumed to be uniformly distributed over the entire length of weld. A connection of any proportions can be designed by successive trials to determine an economical combination of L and weld size for which the allowable stress per inch of weld is not exceeded.

For the welds shown by Fig. 5-26, $\bar{y}$ (from top to center of gravity) = $L/2.4$, and I (about center of gravity) = $0.25L^3$.

At the top of the connection,

$$T = \frac{M\bar{y}}{I} = \frac{Pe}{0.6L^2},$$

$$V = \frac{P}{2.4L},$$

$$R = \frac{P\sqrt{16e^2 + L^2}}{2.4L^2}.$$

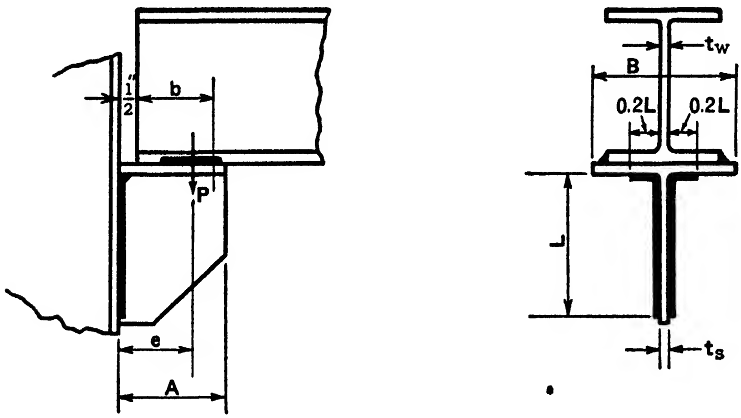


FIG. 5-26.

The stress in the weld is computed at the top of the weld rather than at the bottom. It is considered that the nature of the connection does not permit a compression failure.

The connection is designed by limiting R to the allowable shear per inch of one fillet weld. Design can be accomplished by trial, by solution of the above equation, or by the use of curves of the equation prepared by H. M. Priest.* The curves are reproduced in Fig. 5-27, but with some symbols changed to conform with those of this text. The lengths given by the equation or by the curves are net lengths, to which must be added the allowance for end tapers.

The allowable value of R may be controlled by the stiffener thickness t_s (see Art. 5-11). If an excessive length of weld seems necessary, it may be possible to reduce the length by using a thicker stiffener.

* Priest, *op. cit.*

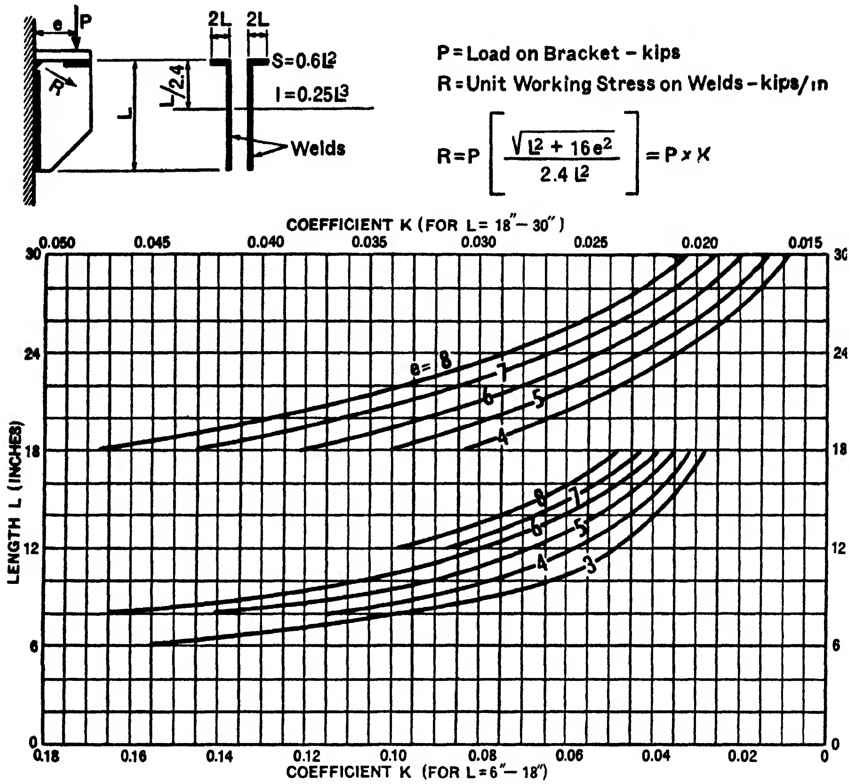


FIG. 5–27. Welds for stiffened brackets. [Source: H. Malcolm Priest, *The Practical Design of Welded Steel Structures* (New York: American Welding Society, 1943).]

EXAMPLE 5–15. Design completely a stiffened seat, as shown in Fig. 5–26. The beam supported is a 12 WF 27 having a reaction of 18 kips. Use a T-section (cut from a WF or American Standard beam section) for the seat. The allowable stresses are those of the AISC and AWS (building) specifications.

(*a*) *To determine:* The bearing length b and seat width A required to prevent crippling of the beam web.

$$b = \frac{P}{24t_w} - k = 2.31 \text{ in.}$$

$$A = 0.50 + b = 2.81 \text{ in. min.}$$

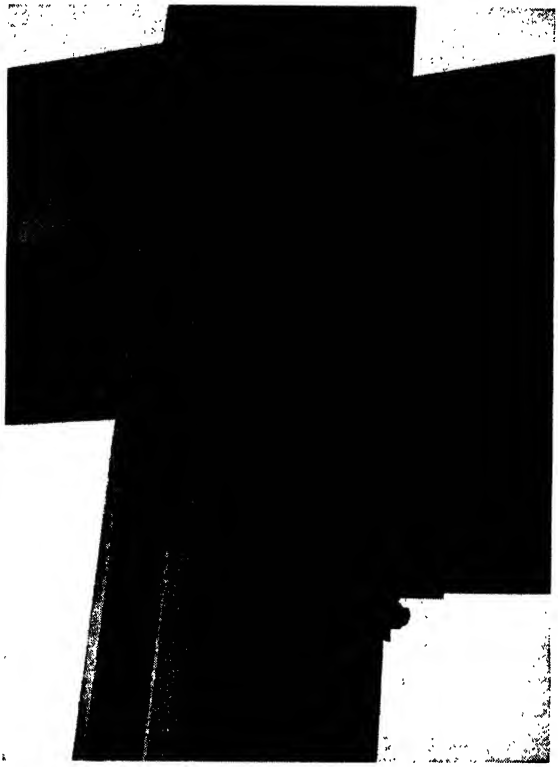
Use $A = 4$ in. (See Art. 5–18.)

(*b*) *To determine:* The required stiffener thickness t_s.

e (eccentricity of the load with respect to the welds) $= 4.00 - 2.31/2 = 2.84$ in.

Eccentricity of the load P with respect to the center of the stiffener width $= e - A/2 = 0.84$ in.

The bearing stress f_{br} (at the upper right corner of the stiffener) is now computed.



Bethlehem Steel Co.

A Bolted Beam to Column Connection.

$$\frac{P}{\text{Stiffener area}} + \frac{Mc}{I} = \frac{18}{4t_s} + \frac{18 \times 0.84 \times 2}{\frac{1}{12} \times t_s \times (4)^3} = \frac{10.17}{t_s}.$$

Since the seat and the stiffener are parts of the same rolled section, the AISC allowable stress of 30,000 psi for milled stiffeners will be used. (If the stiffener were to be joined to the seat by welding, the stress of 27,000 psi for fitted stiffeners would be used.) Limiting f_{br} to 30 kips per sq in.,

$$t_s \text{ required} = \frac{10.17}{30} = 0.34 \text{ in.}$$

Since the stiffener is a compression element, its ratio of outstanding width to thickness should be limited to not more than 16. One-sixteenth of A is only 0.25 in.; therefore, a stiffener having a thickness of 0.34 in. or more is satisfactory.

(*c*) *To determine:* The weld length and size.

For welds on both sides of a stiffener of t = 0.34 in., W (max effective) = 0.7 × 0.34 = 0.24 in. (per weld), and R allowable per inch = 0.24 × 0.707 × 13.6 = 2.30 kips per in.

At the top of the connection (see Art. 5–22),

$$R = \frac{P\sqrt{16e^2 + L^2}}{2.4L^2}$$

Substituting 2.3 kips per in. for R, 18 kips for P, and 2.84 in. for e gives

$$2.3 = \frac{18\sqrt{129 + L^2}}{2.4L^2}$$

$$0.094L^4 = 129 + L^2$$

$$0.094L^4 - L^2 = 129$$

Solution by substitution of trial values can be made as follows:

L	$0.094L^4$	$-L^2$	
6	122	$-36 =$	$86 < 129$
7	226	$-49 =$	$177 > 129$
6.5	168	$-42 =$	$126 \doteq 129$

Specify L as $6\frac{3}{4}$ in. net, using $\frac{1}{4}$-in. fillet welds. The stiffener height should allow $\frac{1}{4}$ in. extra for taper at the lower end of the welds.

The seat could be made from a piece of 8 WF 40, 10 WF 49, or many other sections; or it could be made of two plates, welded to form a tee-section.

5–23. Riveted or Bolted Web Connections. Perhaps the most frequently used but least understood connection is the beam web connection shown in Fig. 5–28(a). It consists usually of two angles, shop connected to the web of the supported beam and field connected to the supporting member. It is the intent to provide by this detail a simple-beam connection which permits rotation of the end of the beam. For end rotation to occur, it is necessary that the upper part of the connection angles deform in bending, as illustrated by (b) of Fig. 5–28. It can be shown that, for this deformation to be great enough to permit the end of the beam to rotate as a simply supported end, the angles will be stressed in bending beyond the yield-point stress. It is probable that, with the application of load to the beam, the angles undergo permanent deformation; that deformation permits rota-

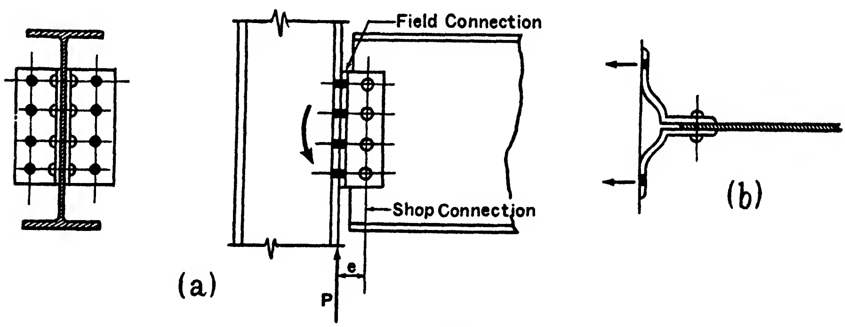


Fig. 5–28.

tion almost as great as needed for the simple-support condition. A small end moment is produced by the connection, although the amount of the end moment is not great enough to be considered as appreciably reducing the positive bending moment for which the beam section is selected. Although it has been satisfactory for beams having nearly static loads, such connections for beams having loads applied and removed a large number of times have frequently resulted in fatigue failure near the fillet at the upper end of the angles.

Since the intent is to make rotation as easy as possible, the proportions should be determined with that aim in mind. The AREA specification attempts to provide some flexibility by limiting the thickness through an empirical formula relating it to the other dimensions of the angle.

Since slight end moment is provided by the connection, the field rivets or bolts are frequently assumed to act in vertical shear only. It is assumed also that the small moment *Pe* on the shop rivets or bolts is resisted by friction between beam web and connection angles. The shop rivets are then assumed subject to vertical shear only. The design procedure follows:

1. Determine the number of shop rivets or bolts required. The value of one rivet is usually controlled by bearing on the web, but it may be controlled instead by double shear or by bearing on the angles.
2. Determine the number of field rivets or bolts required. The value of one rivet may be controlled by shear, by bearing on the connection angle, or by bearing on the supporting member.
3. Specify the number of rivets or bolts of each type to be used so that the detail and fabrication of the joint is practical. For example, if it is determined in steps (1) and (2) that four shop rivets and five field rivets are required, four shop rivets and eight field rivets would be used, as in Fig. 5–28(*a*).

5–24. Welded Web Connections. A welded web connection is shown in Fig. 5–29. The shop welds and the field welds of this connection are each designed to resist both vertical shear and moment.

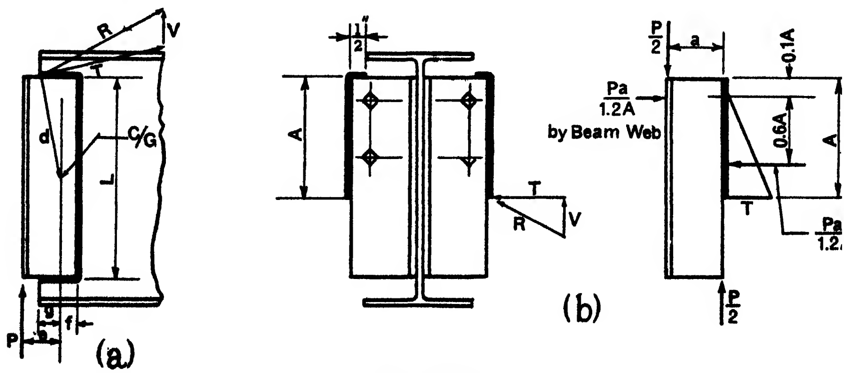


FIG. 5–29.

In analysis of the shop welds, the distance e from the back of the angle to the center of gravity of the shop welds is computed. This analysis assumes that there is no bending moment on the line of application of force P. Each weld resists the direct load $P/2$ and the moment $Pe/2$. The resultant stress R on the shop welds is the vector sum of the stresses V and T, due to vertical load and moment, respectively. The following equations show how V and T may be determined:

$$V = \frac{P}{2(L + 2f + 2g)},$$

$$T = \frac{Ped}{2J},$$

$$J \text{ (per weld)} = \frac{f^3 + g^3}{1.5} + \frac{(f + g)L^2}{2} + \frac{L^3}{12} + Lf^2.$$

The maximum R occurs at a corner nearest to the end of the beam. [See Fig. 5–29(a).] A suitable connection is one for which R does not exceed the allowable shear per inch of weld.

Direct determination of the required dimensions from the above equations is not practical, but design may be readily accomplished by successive approximation. Usually two or three trials are sufficient.

The field connections shown by Fig. 5–29(b) are each subject to a vertical load $P/2$ and to a rotating moment $Pa/2$. Rotation of the connection angle is assumed to be prevented by the pressure of the top end of the angle against the beam web and by horizontal shear in the lower part of the field weld. The top pressure is assumed as concentrated at $0.1A$ from the end of the angle. The horizontal shear in the welds is assumed to vary in intensity from zero at $0.1A$ from the top to a maximum T at the bottom. The arm of the resisting couple is then $0.6A$. For equilibrium, the pressure of the angle against the web and the horizontal force in the weld must each equal $Pa/1.2A$.

R for the field welds is computed as follows:

$$V = \frac{P}{2A},$$

T at the lower end is twice the average horizontal force per inch of weld, or

$$T = \frac{2Pa/1.2A}{0.9A} = \frac{Pa}{0.54A^2},$$

$$R = \frac{P}{A}\sqrt{\frac{1}{4} + \left(\frac{a}{0.54A}\right)^2}.$$

The required length A is the minimum length for which R does not exceed the allowable shear per inch of weld.

For a particular size of angle, curves can be developed giving values

L and *A* for various loads *P* and weld sizes. One such set of curves, developed by H. M. Priest,* is shown in Fig. 5-30. These curves are for 4 x 3 structural steel angles, connected as shown, and for an allowable unit shear stress of 13,600 psi in the welds.

End rotation of the beam causes the upper end of the angles to bend in a manner that might easily tear the field weld. Two precautions can be taken to prevent this action. A short return of the field welds (say, ½-in.) can be made along the upper edge of each angle. The return would rein-

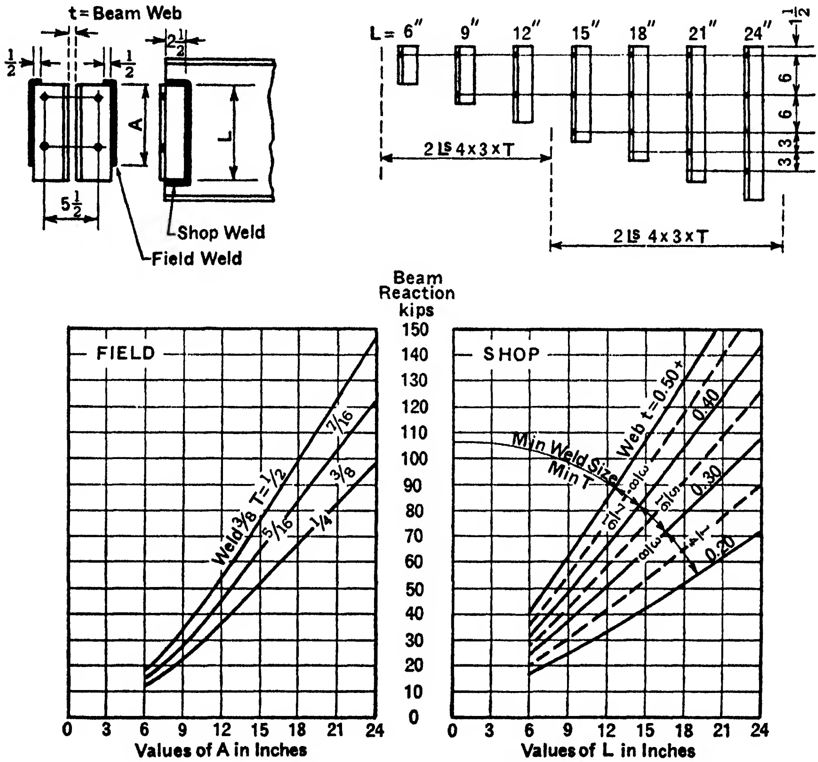


Fig. 5-30. [Source: Malcolm H. Priest, *The Practical Design of Welded Steel Structures* (New York: American Welding Society, 1943).]

force the weld at the point where the tearing action is most severe. The other method is to locate the field erection bolts near the top of the angles and to leave them tightly installed after field welding. While such bolts would prevent the tearing or bending of the field welds, it would be difficult to be certain that the bolts would not later be loosened or removed. Also, they would reduce the flexibility of the connection slightly. The weld returns are the more satisfactory solution.

* Priest, *op. cit.*

5-25. Standard Web Connections. To simplify design procedure and to reduce the large variety of sizes of web connection that might be designed for use on a single structure, engineers have established standard sizes of connections. For riveted building construction, the AISC manual *Steel Construction*, gives tables of standard web connections for varying nominal depths of beam section. The tables give also the maximum safe load for each connection. It is an acceptable and usually economical practice to use the standard connection, even though it may be much stronger than required for the expected loads. When very heavy reactions are encountered, it may be necessary to design a special connection for safety.

A set of standard sizes, suggested by H. M. Priest for welded web connections, is shown in Fig. 5-30.

Web connections are made by the following combinations of shop and field methods:

1. Shop riveted and field riveted
2. Shop riveted and field bolted
3. Shop bolted and field bolted
4. Shop welded and field welded
5. Shop welded and field bolted
6. Shop welded and field riveted

EXAMPLE 5-16. Design an all-welded web connection to join a building beam, 18 WF 50, having a reaction of 25 kips to the flange of a column.

The beam web thickness equals 0.358 in. The maximum effective weld size for welds on both sides of the web is 0.7×0.358, or 0.251 in. Use ¼-in. welds.

By Fig. 5-30 the required dimensions are $L = 6.0$ in. and $A = 10.0$ in.

The field weld controls the length of the connection. If a large number of these connections were to be used, an angle 4 x 3 x ⅜ x 0-10 would be used with shop and field welds each 10 in. in height. If only a few were required, greater economy of detailing and fabrication would probably result from using the 12-in. length standard connection of Fig. 5-30, with L of 12 in. for the shop welds and A of 10 in. for the field welds.

EXAMPLE 5-17. Design a riveted web connection for the beam of Ex. 5-16. Use ¾-in. rivets. Assume the beam to be connected to a ¼-in. thick web of a transverse girder.

The strengths of one shop rivet are:

Double shear, 13.25 kips,

Enclosed bearing on 0.358-in. web, 10.8 kips, *Controls*

Bearing on two ⅜-in. angles, 18.0 kips,

$$\text{Number required, } \frac{25}{10.8} = 3.$$

The strengths of one field rivet are:

Single shear, 6.63 kips,

Bearing on 0.250-in. girder web, 6.00 kips,

Number required, $\frac{25}{6} = 5$.

A practical detail would be one having six field rivets and three shop rivets. The angle used must be thick enough to provide resistance to shear and buckling. (The angles act like extensions of the beam web.) To check for shear in the angles is simple, but to compute whether they will buckle is difficult. A standard connection angle 4 x 3½ x ⅜ would be suitable.

Unless a large number of very light connections like that of this example were needed, the standard connection B4, shown in *Steel Construction* (AISC), would be used.

5-26. Flexibility of Welded Connections for Simple Beams. If a beam is designed as a simple beam, its end connections must be arranged to

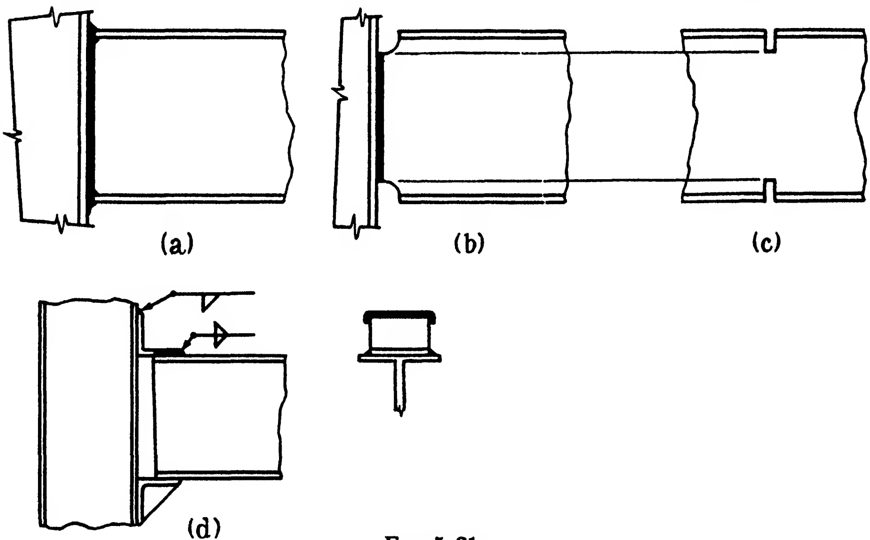
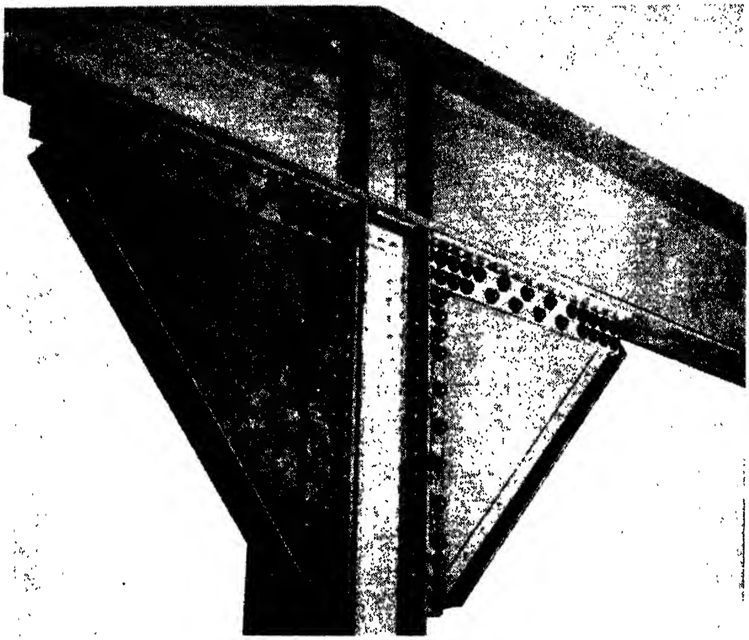


Fig. 5-31.

permit easy rotation of the end of the beam. If they are not, then either the connection, the beam, or supporting structure may be overstressed.

Occasionally the urge to simplify welded beam connections has led to rigid designs, without providing for the effects of end moment. For example, the simplest kind of beam connection, shown by Fig. 5-31(*a*), is a complete welding of the beam section to the support. The connection is obviously rigid, permitting only that rotation which results from bending of the column; it is proper if provision is made for end moment.

That shown by Fig. 5-31(*b*) is only a partial development of the beam section by the welds. Yet, it cannot permit free end rotation of the beam.



Bethlehem Steel Co.

BEAM TO COLUMN CONNECTION WITH MOMENT CONNECTION ON COLUMN.

If rotation of the ends is prevented, an end moment develops which may be as great as two-thirds of the simple-beam moment for which the section is selected. Since almost the entire section is needed to resist that amount of bending moment, the reduced section at the welds is likely to be overstressed. The effect is similar to that which would occur if the flanges and web were cut at the point of maximum bending moment, as shown by Fig. 5–31(*c*). This connection has been used for building work—often without ill effects. Why? Three explanations may be given. Since building loads are largely static, a yielding of overstressed connection material may occur, thus permitting greater rotation. In some cases, a large portion of the load may be present before welding, so that the rotation required after welding is reduced. Ultimate collapse does not occur, but the factor of safety for some parts is reduced. Since a uniform factor of safety is desirable, it is considered best to design flexible connections, or to consider the effect of their rigidity in the design of both the connection and the members it connects.

The connection of Fig. 5–31(*d*) is merely a beam-seat connection with a top angle added. The top angle is often used, either to prevent overturning of the beam or to provide some resistance to end moments due to wind loads. Yet it is usually considered as a simple-beam connection. Obviously, it cannot be a simple-beam connection and provide end-moment resistance. As the beam end rotates under vertical loading, the top angle is bent away from the column. For the usual amounts of end rotation, the bending stresses in the angle exceed the yield point. The angle permanently deforms, permitting end rotation of the beam but causing a slight end moment to remain in the beam. The angle could aid in resisting end moments due to wind, but only if those moments did not exceed the amount needed to cause yield-point stress in the angle. The weld connecting the angle to the column should be located near the upper edge only. If it were extended all the way down the sides of the angle, the flexibility of the connection would be destroyed. The weld, as shown by Fig. 5–31(*d*), however, is subject to bending like that shown by Fig. 5–12(*a*). Field bolts or a top weld having ½-in. returns along the vertical edges would be better. That this connection is used with success can probably be explained by the three reasons given in the preceding paragraph.

While it has some faults, the connection of Fig. 5–31(*d*) may be considered satisfactory for static loads. That of Fig. 5–31(*a*) should be used only if the resulting end moment is considered in the design. That of Fig. 5–31(*b*) should be avoided.

A simplification of the top angle is a flat plate, connected as shown by Fig. 5–33(*c*). If this is used, however, the beam end cannot rotate freely. While such a connection is good for continuous beams, it is not a simple beam connection.

5–27. Beam Connections to Resist Moment. A rigid connection is needed if the beam is intended to have end moment. Such is the case where resistance to wind or other lateral loads is to be provided by the

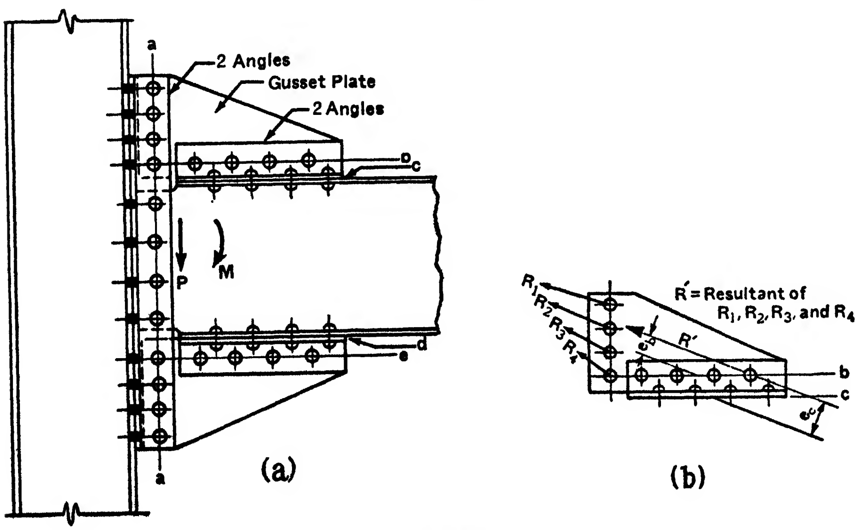


FIG. 5–32.

beam connection or where continuity is required between the beam and the adjoining structure. Moment resistance is generally provided by one of the two types of connection which will now be described.

A connection of the first type is illustrated in Fig. 5–32. In this connection, resistance to moment and shear is provided through action like that of the stiffened beam seat. The field riveted or bolted connection may be designed as suggested in Arts. 5–19 and 5–20. The rivets of row *a* are designed to transfer the moment *M* and vertical shear *P* from the gusset plates and beam web to the end-connection angles. The rivets of lines *b* or *c* are stressed by the resultant *R′* of the loads transferred to

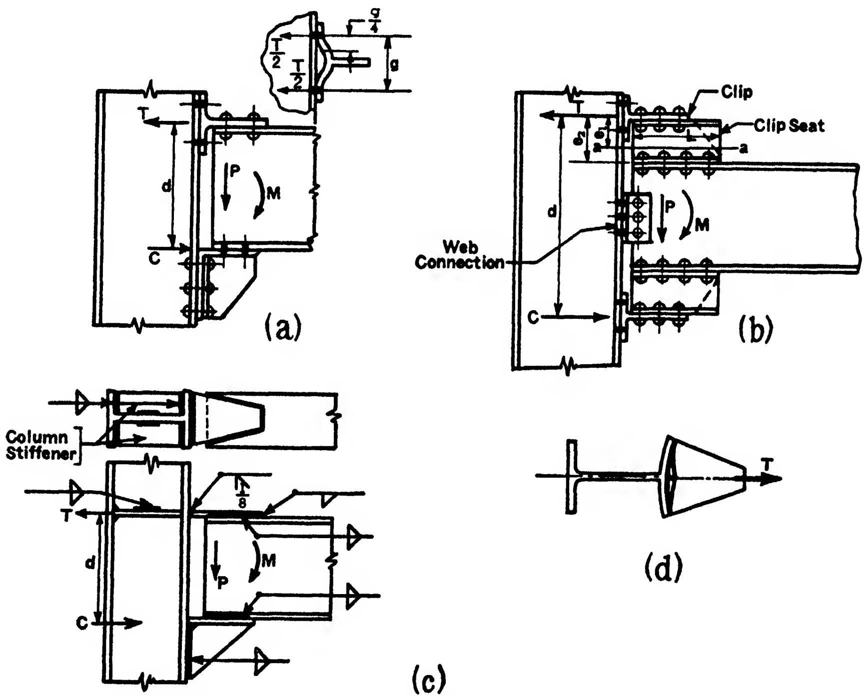


FIG. 5–33.

the upper gusset plate by the rivets of row *a*. As shown in Fig. 5–32(*b*), this resultant may be eccentric to the groups *b* or *c*, so that those groups must be designed to resist the moment $R'e_b$ or $R'e_c$, as well as the vertical and horizontal components of the resultant *R′*. Rows *d* and *e* are designed in a similar manner. A connection of this type involves no new information but is just a combination of several connections of types previously presented. For this reason no example of this type of connection will be given.

Several illustrations of the second type of connection to resist moment are given by Fig. 5–33. In each of these examples the end moment is resisted by a couple consisting of a tensile force T in one flange clip or bracket and a compressive force C in the other ($T = C = M/d$). Each of the examples will be discussed briefly.

The top clip of Fig. 5–33(a) is subject to the tensile load T. In designing the top connection, consideration must be given to the tensile strength of the field rivets or bolts connecting it to the column and to the shear or bearing value of the shop rivets or bolts connecting it to the top flange. The stem or web of the tee used must have sufficient net cross-sectional area to resist tensile failure. The flanges of the tee must be of such proportions that they will not fail in bending. The bending moment on each flange may be assumed to be equal to $T/2 \times g/4$.

The lower seat of Fig. 5–33(a) is designed to resist the vertical reaction P and the compressive force C. The shop connection is designed in the same manner as the stiffened beam seats of previous articles. The moment on the seat, however, is the algebraic sum of the moments caused by the eccentricities of P and C. The moment caused by simultaneous application of P and C may be less than that caused by the application of P or C alone. Therefore, if the separate application of either C or P can occur, the connection should be designed to resist either the separate or the simultaneous application of vertical shear and end moment. (The C force shown is the *reaction* by the column. The force C applied to the seat by the beam is in the opposite direction.)

The rivets connecting the beam to the seat are designed to transfer the load C from the seat to the beam flange.

Because the flanges of both the column and the upper tee-clip of Fig. 5–33(a) bend under the load T, it is doubtful if more than four field bolts or rivets can be effectively used for the top clip. If more were used, the outermost would receive little load; the innermost would be overloaded. Consequently, the application of this connection detail is limited to cases in which T is low enough to permit the use of a reasonable and practical size of field rivet or bolt. For cases in which T is larger, a connection of the type described next might be used. (In some cases, stiffeners might be added to the column flange to permit the effective use of more than four field rivets or bolts.)

Figure 5–33(b) shows a connection in which the amount of T or C is reduced by increasing the lever arm d. The vertical shear P for this detail is transferred to the column by a web connection. The design of the top clip is like that of the top clip of Fig. 5–33(a). The web of the bottom clip must be of sufficient thickness and width to prevent column failure of the portion between the flange and the first row of rivets or bolts.

Clip seats are selected of the depth necessary to keep the forces T and C low enough to permit practical clip details. The length and thickness of the web of the clip seat must be sufficient to prevent failure along line a–a of Fig. 5–33(b). The web at that section is subject to a shearing force equal to T and to a bending moment equal to Te_1. (In computing the unit shearing stress, it should be remembered that the area of the section resisting shear is a rectangle. The maximum unit shear stress is therefore $1.5 \times T/tL$, in which t is the web thickness of the clip seat.)

The rivets or bolts connecting the clip seat to the beam flange are subject to shear and to a moment Te_2. Their number and arrangement may be determined in the same manner as the rivets or bolts of a beam seat.

Many variations of the connection of Fig. 5–33(b) may occur. The clip seats may be cut diagonally (dotted lines) to improve appearance. If the moment M is not too great, only one clip seat may be used, one clip being attached to the clip seat, the other directly to the beam flange.

Figure 5–33(c) shows one of many possible types of welded beam connection to resist moment. The force T is transferred by fillet welds from the beam flange to the top plate and by a bevel butt weld from the top plate to the column. The thickness and maximum width of the top plate is determined by the area of plate and the area of butt weld required to resist the force T. For easier attachment in the field, the top plate may be tapered as shown. If the required butt-weld length and the plate width are about 1 in. less than the beam-flange width, the field welding may be in the flat position without tapering the plate.

A root opening of about $\frac{1}{8}$ in. is usually specified for the butt weld. To avoid difficulties in providing exactly that amount of root opening, it is advisable to fabricate and ship the top plate separate from the beam and to weld it to both the beam and the column during erection.

The bottom part of the detail of Fig. 5–33(c) is a stiffened beam seat. The weld connecting the beam to the seat is designed to transmit the force C from the seat to the beam flange. The weld connecting the seat to the column is assumed to be subject to the shear P, the force C, and the moments caused by the eccentricities of both P and C. The design of the weld may sometimes be controlled by P alone rather than by a simultaneous application of P and C. Note that the curves of Fig. 5–27 cannot be used for the combined action of P and C. A trial seat must be selected, and an analysis must be made to determine whether it is satisfactory, or an adjustment in the design may be necessary.

The effect of the force T on the column flange must be considered. If the top plate extends over an appreciable width of the column flange, there is a tendency for the flange to bend, as shown in Fig. 5–33(d). Such bending causes the butt weld near the center to be more highly stressed than that near the ends. If the width and thickness of top plate were chosen assuming a uniform distribution of tensile stress, the weld near the center would be overstressed. A progressive failure starting from the center and working toward each end could result.

Overstressing of the butt weld at the center can be avoided by using extra weld length beyond the computed requirement, or better, by providing stiffeners to prevent the column flanges from bending. A pair of plates is included for this purpose in the column of Fig. 5-33(*c*).

5-28. Connections of Tension and Compression Members. The load for which end connections of axially loaded members are designed is usually well defined by the applicable specification. The amount required varies from the design load of the member to the full capacity of the member. Connections for members having reversal of stress are designed for the larger load plus a specified percentage of the smaller load.

5-29. Splicing of Compression Members. The design of splices for compression members is usually well controlled by the specification. There are two distinct types of compression splice; those having the ends of the spliced members cut by ordinary methods, and those having the ends milled to provide plane surfaces.

In splices of members not milled, the ends are kept slightly apart, and splice material (bars, plates, angles, etc.) is connected to each element

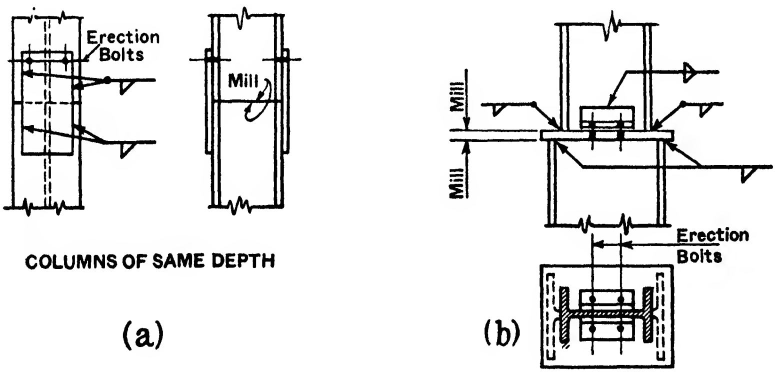


FIG. 5-34.

(flange, web, etc.) of the members. The splice material is designed to transmit from one member to the other the load of the element to which it is connected. The complete splice is designed to transmit the design load of the member, or more if required by the specification.

In splices of compression members having milled ends, the ends are placed firmly in contact with each other and are held together by splice material connected to each element of the section. At least a part of the load is assumed to be transferred from one member to the other by the bearing pressure between the milled surfaces. For example, the AREA and aluminum specifications permit 50 per cent of the load to be assumed transmitted by bearing and require that the splice materials be strong enough to transmit at least the remaining 50 per cent. The AISC and AWS specifications permit the assumption for tier-building columns that all the load is transmitted by bearing of the milled ends against each other and require that splice material be placed on all sides, so as to hold the abutting elements firmly in line with each other. For other types of compression member than tier-building columns, the AISC and AWS specifications require that the splice materials be proportioned to transfer 50 per cent of the computed load. The AASHO specifications make no allowance for load transmitted through the milled ends.

Two typical splice details for building columns with milled ends are shown in Fig. 5–34, part (*a*) of which shows a splice that may be used when the sections joined are of the same nominal depth. A similar splice could be made with riveted construction. Filler plates may be used for small differences in depth of sections.

Figure 5–34(*b*) shows a splice detail which can be used for sections of equal depth or for two different depth sections. In this splice a plate is shop welded to one member, and erection clip angles are welded to the other. In the field, the sections are held together by temporary erection bolts while the upper section is field-welded to the plate. An advantage of this splice over that of Fig. 5–34(*a*) is that no holes are needed in the main members. The holes for the erection bolts are punched in the small, easy-to-handle parts before assembly in the shop.

The welds of both splices shown have three purposes. They hold the abutting parts in correct alignment. They transfer shear, if it is present, from one section to the other. If the column is in bending at the splice, the welds transmit the tensile stresses from one section to the other. (If shear in the plane of the web were expected in the splice of Fig. 5–34(*a*), splice plates would be placed against the web also.)

If the column-section size changes, as in Fig. 5–34(*b*), the plate is placed in bending. It must be selected thick enough to resist this bending. If a very thick plate seems necessary, it can be reduced by the use of stiffeners welded to the web of the larger section and to the plate opposite the flange of the smaller section.

For all compression splices, if either bending moment or transverse shear is present in the member at the splice location, the splice must be capable of resisting them. Where two members of different section are spliced, it is recommended that they be located with their centers of gravity in the same line. If this is not possible, the moment caused by the eccentricity should be considered in the splice design.

EXAMPLE 5–18. Design a riveted splice for the steel column shown in Fig. 5–35(*a*). Use ⅞-in. rivets. The axial load is 115 kips, the shear 10 kips, and the bending moment 45 ft-kips at the splice. Use the AREA specification.

The unit stess f in the member at any point is $P/A \pm My/I$. The computed values of f are shown in part (*b*) of Fig. 5–35.

Splice material for the right flange will be selected first. The total flange load P_m is the product of the flange area and the average unit stress in the flange. Thus,

$$P_m = 10 \times 0.64 \times 11{,}990 = 76{,}700 \text{ lb.}$$

The amount of load to be transferred by the flange splice material is one-half of P_m, or 38,350 lb compression.

$$F_a \text{ (allowable compressive stress)} = 15{,}000 - \frac{1}{4}\left(\frac{L}{r}\right)^2$$

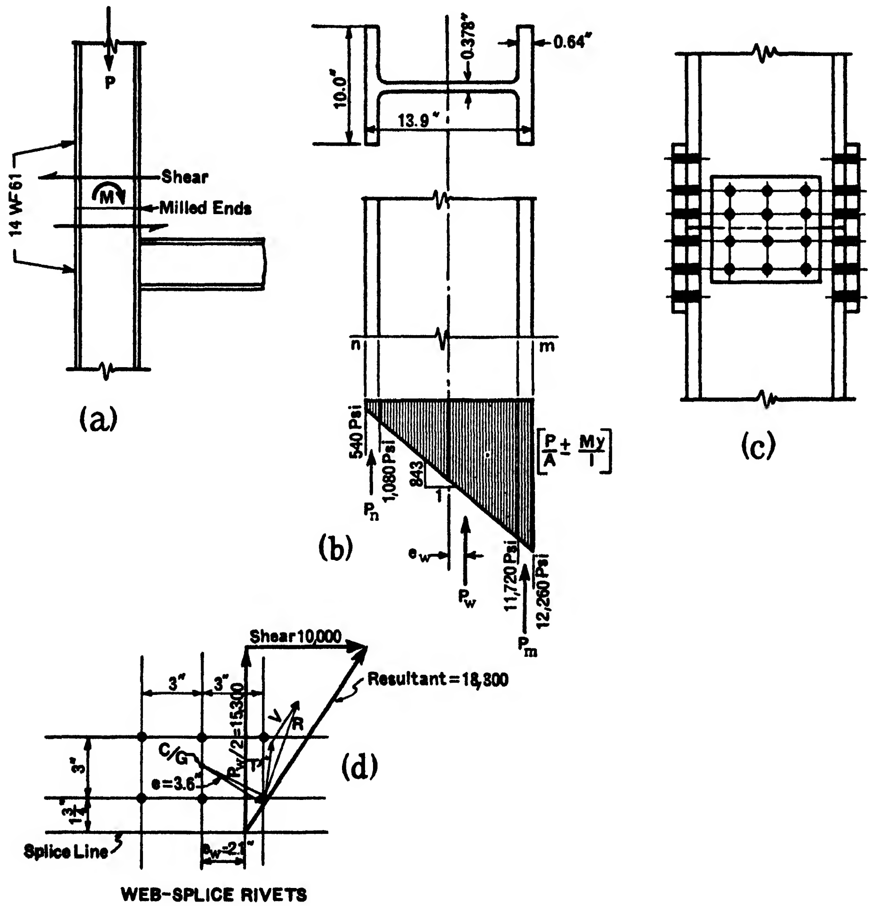


FIG. 5-35.

The minimum material thickness permitted by the specification is ⅜-in. Assuming that a ⅜-in. splice plate is to be used and the pitch between rivets is 4 in.,

$$\frac{L}{r} \text{ (for the splice plate)} = \frac{4}{0.375 \times 0.289} = 37,$$

and $F_a = 14{,}660$ psi,

$$A \text{ required} = \frac{38{,}350}{14{,}660} = 2.62 \text{ sq in.}$$

A plate 7 x ⅜ provides this area, but to avoid weakening the column laterally, it is advisable to use a plate of about the same width as the column flange. Therefore, a 10 x ⅜ plate will be used.

The values of one ⅞-in. rivet are: 8,120 lb in single shear, or 8,860 lb in bearing on ⅜-in. material.

N (number of rivets) $= 38{,}350/8{,}120 = 4.7$. Use six rivets at each end of the splice plate, as shown in Fig. 5–35(c).

Unless great economy is made possible by doing otherwise, both flanges will be spliced in the above manner.

The web-splice material must transfer half of the load P_w. Thus,

$$\frac{P_w}{2} = 12.62 \times 0.378 \times \frac{1{,}080 + 11{,}720}{4} = 15{,}300 \text{ lb.}$$

Since the stress distribution is nearly triangular, the web compressive load P_w is assumed to act at one-third the web depth from the right edge. The eccentricity e_w for the web splice is then 2.1 in.

Two web splice plates, each 10 x ⅜ will be used. (If $(P_w/2A) + (P_w e_w c/2I)$ for the web splice is computed, it will be seen that these plates are not overstressed.)

The rivets connecting the web plates to the web must transfer the eccentric load $P_w/2$ and the transverse shear in the column. A graphical addition of these forces is shown by Fig. 5–35(d). The resultant load on the group of rivets is 18,300 lb, having an eccentricity of 3.60 in. from the center of the rivet group. Then,

$$J = (4 \times 3^2) + (6 \times 1.5^2) = 49.5.$$

For the lower right rivet, the most highly stressed,

$$T = \frac{18{,}300 \times 3.6 \times 3.35}{49.5} = 4{,}460 \text{ lb,}$$

$$V = \frac{18{,}300}{6} = 3{,}050 \text{ lb,}$$

$$R = 7{,}500 \text{ lb.}$$

Since R does not exceed the allowable bearing value of a ⅞-in. rivet on the 0.378-in. web thickness, the splice as designed is satisfactory.

5–30. Splices of Tension Members. In a correctly designed tension splice, each element (flange, web, leg, etc.) of the member is connected to splice material. The total load for which the splice is computed is defined by the specification. Each element of the member is spliced for its portion of the total splice load. If the total load is eccentric to the center of gravity of the member, the effect of the moment thus produced is considered when computing the load for which the element is spliced.

The design of a tension splice is similar to that of a compression splice, except that (1) buckling of the splice material need not be considered, and (2) net areas of both the splice material and the member must be considered.

Often for built-up sections, and occasionally for rolled single sections, the splices of the various elements are staggered. A more reliable splice is probably obtained by distributing the splices of the several elements along the member. Stress concentrations may be reduced by this process.

An example of a staggered splice is shown in Fig. 5–36. Attention is called to the fact that the splice material at any point is approximately

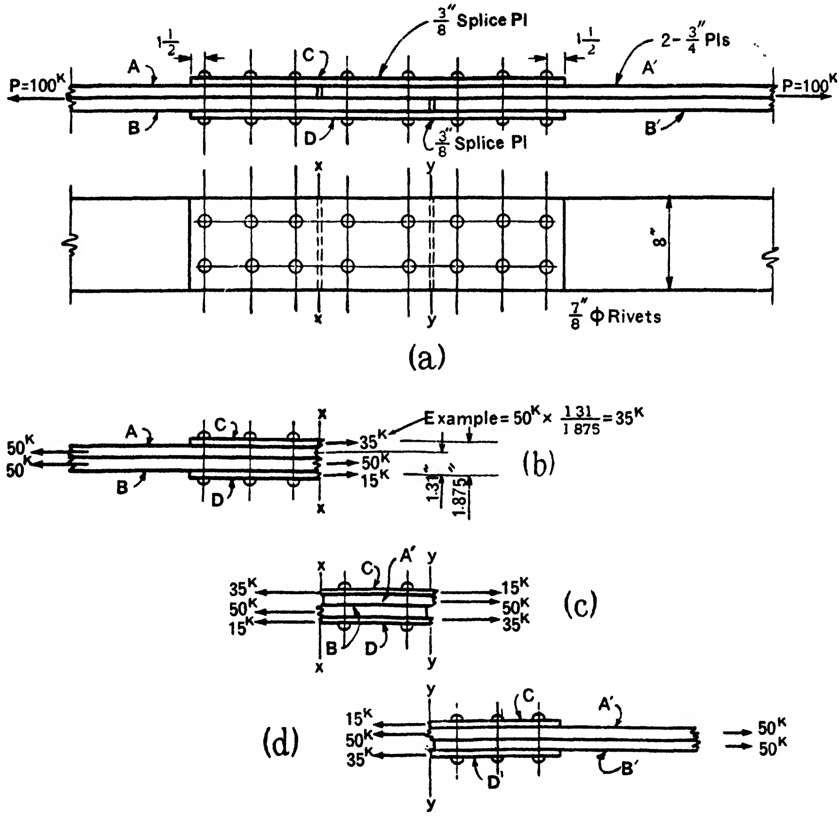


Fig. 5–36.

that required to develop only one of the plates of the original member. To design such a splice, the following procedure is recommended:

1. Assume a size of splice plate. A pair of plates having a total net area equal to, or slightly greater than, that of one of the plates of the member will probably prove sufficient.

2. At sections such as x–x and y–y of the splice in Fig. 5–36 compute the amount

of load in each element of the member and in each splice plate. It may be assumed that the load in the plate which ends at the section is divided between the splice plates inversely as their distance from the ended plate.

3. Compute the number of rivets required to cause the load changes in the member plates and splice plates for each length of splice.

4. Check the net areas to be certain that neither the original material nor the splice material is overstressed.

EXAMPLE 5–19. Determine whether the splice shown in Fig. 5–36 conforms to the requirements of the aluminum specifications. The member is to carry 100 kips tension. The member is of aluminum alloy 2014-T6 (formerly called 24S-T6). The rivets are ⅞-in., cold-driven, and made of aluminum alloy 2117-T3 (formerly called A17S-T3).

Steps (1) and (2) of the procedure suggested in Art. 5–30 are performed on Fig. 5–36. The load in each element is shown at the ends of the splice and at sections x–x and y–y.

The values of one ⅞-in. rivet of 2117-T3 are computed next:

$$\text{Effective rivet diameter} = \text{hole diameter} = 0.891 \text{ in.},$$

$$\text{Ratio } D/t = 0.891/1.50 = 0.595 \;(\therefore \text{ full shear allowed}),$$

$$\text{Shear value} = \frac{10(0.891)^2\pi}{4} = 6.24 \text{ kips},$$

$$\text{Ratio: edge dist}/D = 1.5/0.891 = 1.68,$$

$$\text{Allowable bearing stress} = 30 \text{ kips per sq in.},$$

$$\text{Bearing value on } \tfrac{3}{8}\text{-in. material} = 30 \times 0.891 \times 0.375 = 10.0 \text{ kips},$$

$$\text{Bearing value on } \tfrac{3}{4}\text{-in. material} = 20.0 \text{ kips}.$$

The load in plate A is transferred to the six rivets shown in Fig. 5–36(b). The load per rivet is as follows:

$$\text{Bearing per rivet in plate } A = \frac{50}{6} = 8.33 \text{ kips},$$

$$\text{Bearing per rivet in plate } C = \frac{35}{6} = 5.83 \text{ kips},$$

$$\text{Shear per rivet between plates } A \text{ and } C = \frac{35}{6} = 5.83 \text{ kips}.$$

These values are within the allowable, and so the riveting between section x–x and the left end is satisfactory. Similarly, that between the right end and section y–y is satisfactory.

The four rivets shown in Fig. 5–36(c) are subject to loads as follows:

$$\text{Bearing per rivet in plate } A' = \frac{50}{4} = 12.5 \text{ kips},$$

$$\text{Bearing per rivet in plate } B = 12.5 \text{ kips},$$

$$\text{Bearing per rivet in plate } C \text{ or plate } D = \frac{20}{4} = 5.0 \text{ kips},$$

$$\text{Shear per rivet between } A' \text{ and } B = \frac{30}{4} = 7.5 \text{ kips,}$$

$$\text{Shear per rivet between } A' \text{ and } C, \text{ or between } B \text{ and } D = \frac{20}{4} = 5.0 \text{ kips.}$$

The rivets are overstressed in shear between plates A' and B.

The net areas and maximum unit tensile stresses in the plates are as follows: For plates A, A', B, or B',

$$A \text{ (net)} = 0.75(8 - \overline{2 \times 0.891}) = 4.66 \text{ sq in.,}$$

$$f = \frac{50{,}000}{4.66} = 10{,}700 \text{ psi.}$$

For plates C or D,

$$A \text{ (net)} = 2.33 \text{ sq in.,}$$

$$f = \frac{35{,}000}{2.33} = 15{,}000 \text{ psi.}$$

The allowable unit tensile stress is 22,000 psi; therefore, the net areas provided are sufficient.

To make the splice satisfactory it would be necessary only to increase the number of rivets between sections x–x and y–y to 6, thus reducing the shear between plates A' and B to 5.0 kips per rivet.

5–31. Riveted or Bolted Fillers. Filler plates are often needed to fill spaces between parts which are riveted or bolted to each other. A connection requiring filler plates is shown in Fig. 5–37.

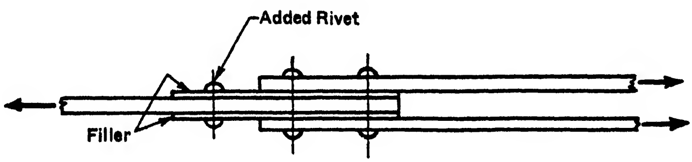


FIG. 5–37.

The allowable stresses for rivets are established for connections transferring load from one layer of material to the adjacent layer. If additional layers, such as fillers, intervene, there is more than the usual tendency for slipping of the plates and for bending of the rivets. Under such conditions, the rivet strength may be reduced.

Slippage and rivet bending can be reduced by connecting the filler to one of the parts with additional rivets, beyond those required if fillers are absent. The number of added rivets required is often controlled by the specification, but if it is not, the following procedure is recommended:

1. Compute the load in the filler, assuming that the load of the member to which

it is connected is divided uniformly over the combined cross-sectional area of the member and the filler.

2. Provide enough extra rivets to transfer this computed load to the filler. The added rivets are usually installed through the extended filler and one of the members, as shown in Fig. 5-37. The added rivets may, however, go through the filler and both members, if it is more convenient or economical.

If the effect of friction between the materials is neglected, this procedure is a logical and a safe one to use. Tests have shown, however, that the maximum number of added rivets, through the filler and one member, that is effective is about one-third of the number required for the same connection without fillers. Added rivets beyond that number cause no increase in the static strength of the joint.

If the load is vibratory or often repeated, so that the friction may be destroyed, it would be well to use the method previously outlined in this article, even though the number of added rivets thus obtained exceeds one-third the number required for the joint without fillers.

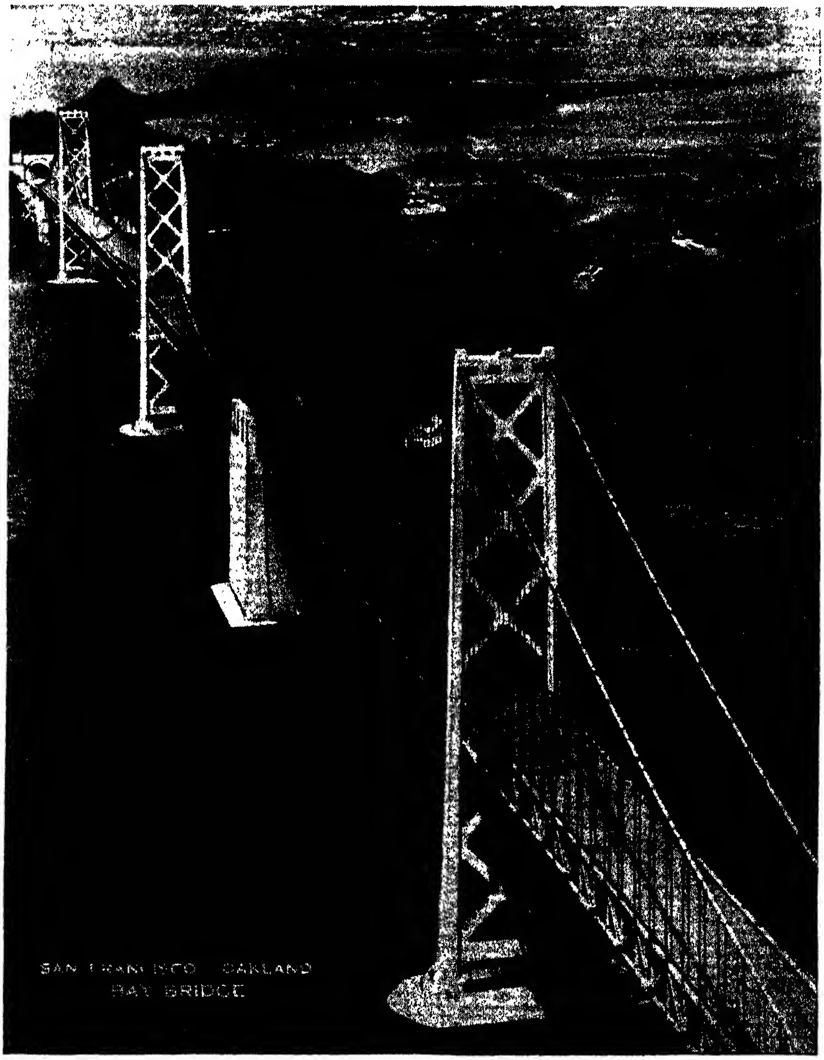
5-32. Gusset Plates. For all but the simplest types of gusset plate there is no direct method of design. The outline of a gusset plate is usually controlled by edge-distance requirements and by the number of rivets or the length of weld required to connect each of the adjoining members. The thickness is usually established by comparison with existing similar structures. In the majority of cases, the gusset plate thus selected has ample strength.

It is a good practice, however, to estimate the gusset-plate stresses whenever the detail is of an unusual type—either in the amount of the loads or in the shape of the plate. It is well, also, to check occasionally the more usual type of plate in order to make certain that the details being used are economical, yet definitely safe.

An exact analysis of gusset stresses is not practical with our present knowledge of their action. An estimate of their stresses may be made at any section of the plate by considering the plate to be cut at that section. The plate on one side of the cut is then treated as a free body; and the internal shear, direct load, and moment required to maintain equilibrium are computed. The stress normal to the cross section is assumed to be $f_n = (P/A) \pm (Mc/I)$. The stress parallel to the edge of the plate is $f_n \sec^2 \theta$, in which θ is the angle between the edge and f_n. If these assumptions are accepted, it is logical also to consider the maximum unit shear to be 1.5 times the average shear.

It is recognized, of course, that the gusset plate does not act in the same manner as the theoretical beam for which the flexure formula is developed. (Because of its low ratio of length to depth, it is probable that the shear deformation is not negligible.)

After the stresses have been estimated, they must be compared with the



American Bridge Co.

SAN FRANCISCO-OAKLAND BAY BRIDGE, SAN FRANCISCO, CALIF.
TWIN SUSPENSION SPANS, CANTILEVER STRUCTURE, THROUGH AND DECK TRUSS SPANS.

allowable values. For tension and shear it is reasonable to use the allowable values given by the specification for bending and for shear in webs. The allowable compressive stress is a function of the amount of restraint to buckling of the edge of the plate provided by the connecting members and by the stiffness of the plate itself. If the compressive stress in the plate is near the full allowable value, not reduced for L/r, it would be well to limit the ratio of laterally unsupported plate width to plate thickness, in the manner prescribed by the specification for compression flanges. If the plate is not stressed so highly in compression, the ratio of unsupported width to thickness could logically be increased. It should be remembered, however, that the critical buckling stress varies approximately inversely as the *square* of that ratio.

In checking a gusset plate it may be necessary to investigate several sections. No rule can be given telling just where those sections are located. Enough sections—horizontal, vertical, and diagonal—should be investigated to assure the designer that the gusset plate is satisfactory.

EXAMPLE 5–20. Estimate the stresses on the gusset plate of Fig. 5–38. The plate will be checked at two sections, labeled *a–a* and *b–b* on Fig. 5–38(*a*).

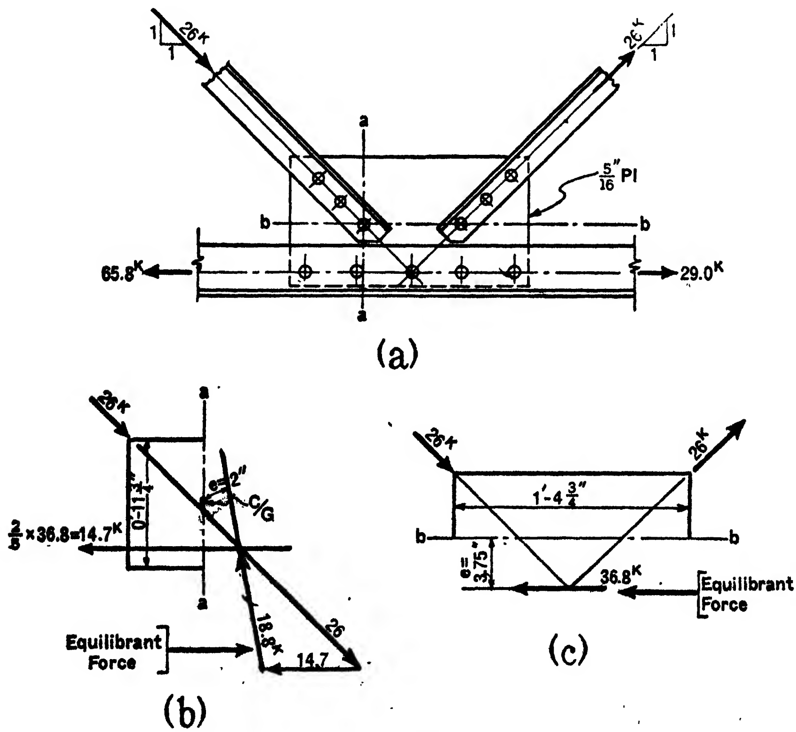


FIG. 5–38.

The forces acting on the portion to the left of section *a–a* are shown in Fig. 5–38(*b*). The resultant of the internal stresses, the equilibrant force, is determined graphically. The eccentricity of the equilibrant, measured to the center of gravity of the section, is 2.0 in. The equilibrant may be resolved into a horizontal component of 3.7 kips to the left and a vertical component of 18.4 kips upward.

The properties at the section are as follows:

$$A = 11.75 \times \frac{5}{16} = 3.68 \text{ in.}^2,$$

$$S = \frac{I}{c} = \frac{1}{6} \times \frac{5}{16} \times \overline{11.75}^2 = 7.2 \text{ in.}^3$$

The approximate compressive stress at the upper edge of the plate is

$$\frac{P}{A} + \frac{Mc}{I} = \frac{3.7}{3.68} + \frac{18.8 \times 2}{7.2} = 6.22 \text{ kips per sq in.}$$

The approximate tensile stress at the lower edge is

$$\frac{P}{A} - \frac{Mc}{I} = 4.22 \text{ kips per sq in.}$$

The approximate unit shear at the center is

$$1.5\,\frac{V}{A} = \frac{1.5 \times 18.4}{3.68} = 7.50 \text{ kips per sq in.}$$

If the tensile stress at the lower edge were closer to the allowable tensile stress, it would be advisable to repeat the computation, using the properties of the net section of the plate. It is apparent by inspection in this case that the holes will not cause the tensile stress to rise to as high as 20 kips per sq in.

The forces acting on the portion of plate above section *b–b* are shown in Fig. 5–38(*c*). For this section,

$$A = 16.75 \times \frac{5}{16} = 5.23 \text{ in.}^2,$$

$$S = \frac{1}{6} \times \frac{5}{16} \times \overline{16.75}^2 = 14.6 \text{ in.}^3,$$

$$P = 0,$$

$$M = 36.8 \times 3.75 = 138 \text{ in.-kips},$$

$$f = \frac{138}{14.6} = 9.45 \text{ kips per sq in.},$$

$$v = \frac{1.5 \times 36.8}{5.23} = 10.5 \text{ kips per sq in.}$$

The tensile and shear stresses on both sections are well below those permitted by the specification. A conservative value of the allowable compressive stress is approximated as follows: *w* (laterally unbraced width of plate in compression on section *a–a*) = 6.5 in. Then,

$$\frac{w}{t} = \frac{6.5}{5/16} \quad 21.$$

Since a compression failure of the plate would be a local buckling of the upper edge, an allowable compressive stress of 17,000 psi would be logical (under the AISC specification) if w/t were not over 16.

The approximate allowable compressive stress F_a for $w/t = 21$ is, therefore,

$$17 \times \left(\frac{16}{21}\right)^2 = 9.9 \text{ kips per sq in.}$$

The allowable value just estimated is believed to be conservative, since it is based on a specification rule for flanges in which the stress is uniform, while in this particular case the stress varies from maximum at the free (upper) edge of the unbraced width to nearly zero at the fixed edge (at the toe of the bottom angles).

PROBLEMS

5-1. Prepare a table giving the edge distance required to develop the full bearing or shearing value (whichever controls) of an AN430-DD6 rivet (24S-T aluminum alloy, $\frac{3}{16}$-in. diam round head) in 24S-T Alclad sheet of the following thicknesses: 0.032 in., 0.040 in., 0.051 in., 0.064 in., 0.072 in., 0.081 in. The ultimate strengths given by ANC-5 are as follows: shear in 24S-T Alclad, 34,000 psi; bearing in 24S-T Alclad, 82,000 psi; shear in 24S-T rivets, 35,000 psi. (*Note*: Designation 24S-T was changed in 1954 to 2024-T.)

5-2. Two axially loaded 3 x $\frac{3}{8}$ bars of structural steel are overlapped and connected to each other in a single-lap joint by two $\frac{3}{4}$-in. diam rivets. What is the maximum axial tensile load allowed by the AISC specification?

5-3. Compute the allowable load P and state which type of failure would probably occur in the detail of Fig. 5-39. Use the aluminum specifications. (Note shear reduction due to use of rivets in thin material.)

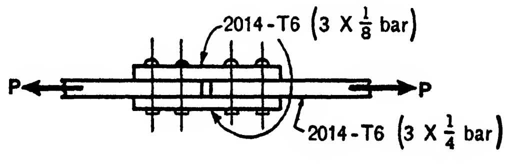


FIG. 5-39.

5-4. How many $\frac{3}{4}$-in. rivets are required for a joint of the type shown in Fig. 5-2(b) when the inside member is a $2\frac{1}{4}$-in. thick bar, the exterior members are each $1\frac{1}{8}$-in. thick bars, and the tensile load in the inside member is 145 kips? Use the AISC specification.

5-5. Design a connection to develop the full tensile value of two 8 [11.5, placed $\frac{1}{2}$ in. back-to-back and riveted with two rows of $\frac{3}{4}$-in. rivets to a $\frac{1}{2}$-in. gusset plate. Use the AISC specification.

5-6. A 4 x 4 x $\frac{3}{8}$ angle is riveted to a $\frac{3}{8}$-in. gusset plate, using $\frac{7}{8}$-in. rivets in one leg only. Design and detail the connection completely, showing number of rivets, gage, spacing, and edge distance. Design for the full allowable tensile load. Use the AREA specification.

5-7. A tension member consists of two 2014-T6 aluminum alloy angles, 3 x 3 x ¼, connected to opposite sides of a ⅜-in. 2014-T6 gusset plate, as shown in Fig. 5-5. Design a riveted connection to develop the strength of the member. Use cold-driven ⅝-in. 2117-T3 aluminum alloy rivets in 0.641-in. reamed holes. Do not use lug angles. Use the aluminum specifications.

5-8. Redesign the connection for Prob. 5-7, assuming that the length by which the angles overlap the gusset plate is limited to 11 in. Use lug angles as required. Assume a minimum allowable spacing of three times the nominal diameter.

5-9. For the two 10½ x 5/16 Pls shown in Fig. 5-40, find the capacity of the group of ¾-in. rivets. What is the minimum stagger S required in order to leave a net

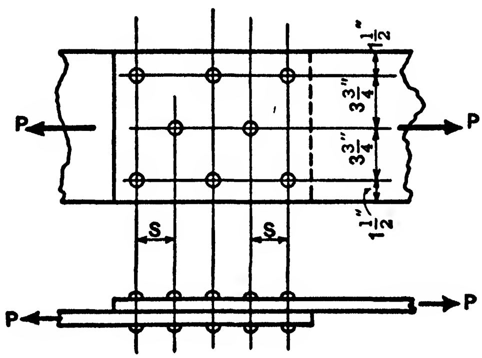


FIG. 5-40.

area in the plates which is great enough to resist a tensile load equal to the capacity of the rivet group? Use the AISC specification.

5-10. What are the maximum allowable tensile and shear loads for the butt-

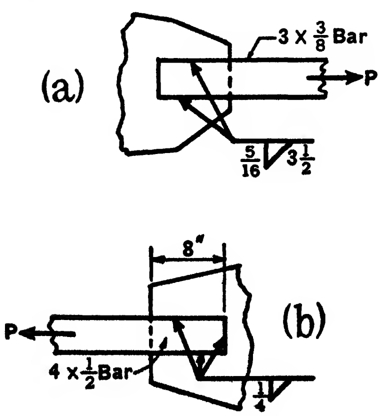


FIG. 5-41.

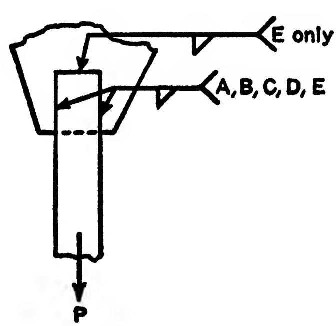


FIG. 5-42.

welded connection of Fig. 5-7, when L is 10 in. and t is ¾ in. Use the AISC and AWS allowable stresses.

5-11. Compute the strength of the connections shown by Fig. 5-41. Use the allowable unit stresses of the AISC and AWS specifications.

5-12. Design welded connections to develop the full strength of the member of Fig. 5-42, as follows:

(a) 3½ x ⅜ bar, side welds only, ⅜-in. fillet.
(b) Same as (a), but with $\frac{5}{16}$-in. fillet.
(c) Same as (a), but with ¼-in. fillet.
(d) Compare (a), (b), and (c) for economy. Note that the amount of weld metal used is a measure of material cost, current cost, and, to a lesser degree, labor cost.
(e) 4 x ½ bar, ¼-in. fillet on sides and end.

Use the AISC and AWS allowable stresses.

5-13. Using longitudinal $\frac{5}{16}$-in. fillet welds, design a double-lap joint to connect a 6 x ⅜ bar to two 4 x ⅜ bars. The unit tensile stress in the 6 x ⅜ bar is 20,000 psi. Use the AWS allowable stresses.

5-14. A 5 x 3½ x ⅜ steel angle is connected by its long leg to a $\frac{5}{16}$-in. gusset plate. Develop the full tensile value of the angle, as computed using the AISC specification, by balanced welded connections, as follows:

(a) Using ¼-in. fillet weld on the toe and the back, none on the end.
(b) Using ¼-in. fillet weld on the toe and ⅜-in. on the back.

5-15. Design balanced welded connections for the member of Prob. 5-14, as follows:

(a) Using ¼-in. fillet weld on the toe, back, and end.
(b) Using ¼-in. fillet weld on the toe and $\frac{5}{16}$-in. on the back and the end.

5-16. What is the maximum tensile load that can be developed in the angle of Prob. 5-14 without exceeding the AISC allowable stresses, when the connection is made by ¾-in. rivets in punched holes? Design the connection, using two rows staggered.

5-17. Design a balanced welded connection of two 3 x 3 x ⅜ steel angles to a ¼-in. gusset plate, arranged as shown in Fig. 5-5. The axial load in the pair of angles is 60 kips. Use the AWS allowable stresses.

5-18. A member is connected to a gusset plate by three rivets, as shown by Fig. 5-43. Compute the load in each rivet when P is 18 kips.

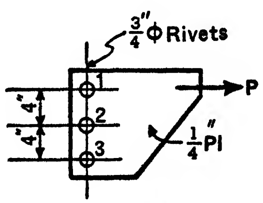


FIG. 5-43.

5-19. Recompute the load on each rivet of Prob. 5-18 when rivet No. 1 is ¾-in. and rivets No. 2 and No. 3 are ⅝-in. diam. Use the AISC specification.

5-20. Using the AISC specification, what is the allowable load P for the connection of Prob. 5-18?

5-21. Find the resultant load on each rivet of Fig. 5-44. Draw the joint to scale and show the resultant direction and amount on the drawing.

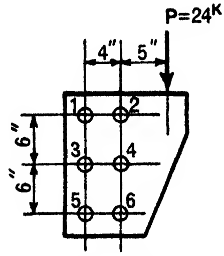


FIG. 5-44.

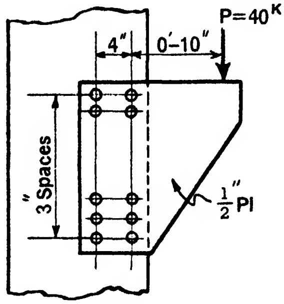


FIG. 5-45.

5-22. For the joint shown by Fig. 5-45, determine how many ¾-in. rivets are required. Use the AISC specification.

5-23. Same as Prob. 5-22, but with P equal to 50 kips, using ⅞-in. rivets. Use the AREA specification.

5-24. Redesign the connection of Prob. 5-22 for 2014-T6 material with 1-in. diam 6061-T43 hot-driven rivets. Use the aluminum specifications.

5-25. Determine the load in each bolt of the connection shown by Fig. 5-46. Bolts 1 and 2 are ¼-in. diam; bolts 3 and 4 are ⅜-in. diam. Use ANC-5 to determine the ultimate stresses in bearing and shear. The far member is ¼-in. thick normalized alloy steel bar, and the near member is ¼-in. thick 2024-T4 plate. The bolt material is alloy steel, heat-treated to a tensile strength of 125,000 psi. (See Ex. 5-9 for the ultimate stress values given by ANC-5.)

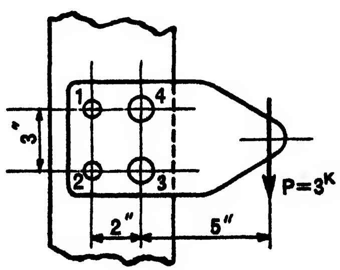


FIG. 5-46.

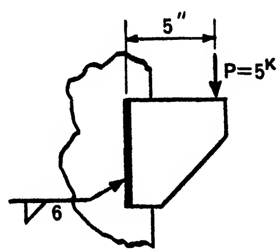


FIG. 5-47.

5-26. For the joint of Fig. 5-47, what is the shear per inch of weld at the most highly stressed point? (In computing the polar moment of inertia, ignore the size of weld. Then $J = (\frac{1}{12})L^3$ for a single weld and $T = (M \cdot L/2)/J$. The unit of T will be pounds per inch of weld.)

5-27. What size of fillet weld is required for the connection of Fig. 5-47 when P is 5 kips applied horizontally at the top edge of the plate?

5-28. Design a connection like that of Fig. 5-17 but of the depth required for a load P of 28 kips. (The 4-in. and 3-in. dimensions of Fig. 5-17 remain unchanged.) Give the depth to the nearest whole inch above the required minimum.

5-29. Using the curves of Fig. 5-20, determine the net length of fillet weld required for each of the following conditions. In each case, the load is parallel to the axis of the weld.

	P	e	*Weld Size*
(a)	30 kips	6 in.	$\frac{3}{8}$ in.
(b)	20	4	$\frac{5}{16}$
(c)	18	5	$\frac{5}{16}$
(d)	25	$2\frac{1}{2}$	$\frac{1}{4}$

5-30. (a) Ignoring the effect of returns at the lower edge of the gusset, and allowing a resultant stress of 2,400 lb per inch of weld, what is the maximum allowable load P for the connection of Fig. 5-48?

(b) Recompute, assuming the side welds only to be $\frac{3}{8}$-in. fillets.

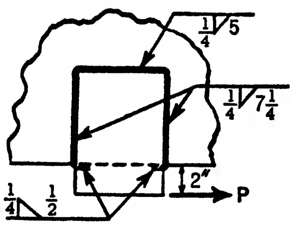


FIG. 5-48.

5-31. Using the AWS specification, determine the length L required for the joint of Fig. 5-49. Assume end returns to cover effect of end taper.

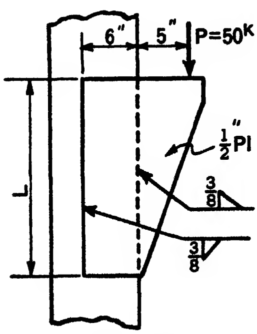


FIG. 5-49.

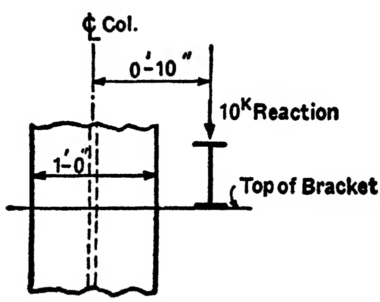


FIG. 5-50.

5–32. The beam of Fig. 5–50 is to be supported by a bracket welded to the near side of the column. Design the bracket and the welded connection of the bracket to the column. Use the AWS and AISC specifications.

5–33. Design completely an unstiffened riveted seat of the type shown by Fig. 5-21(a) for supporting a 14 WF 30 beam having a reaction of 10 kips. Use ¾-in. rivets and an 8-in. length of seat angle. (A complete design should include the size of the angle, the number of rivets, and the spacing of the rivets.) Use the AISC allowable stresses.

5–34. Same as for Prob. 5–33 but for a 10 WF 21 beam having a span of 15 ft and laterally unsupported. Assume the beam to be loaded with uniform loading and stressed to the full amount allowed by the AISC specification.

5–35. Design a welded unstiffened seat of the type shown by Fig. 5–21(b) for the beam of Prob. 5–33. Use a length of angle 2 in. more than the width of the beam flange.

5–36. Design welded unstiffened seats for the following beams:

(a) 8 B 13, reaction 6 kips.
(b) 10 I 25.4, reaction 15 kips.
(c) 12 B 22, reaction 11 kips.
Use may be made of the curves of Fig. 5–20.

5–37. If the rivets in the two rows of Fig. 5–21(c) are ⅞-in. diam, spaced vertically 4 in. center-to-center with end distances of 2 in., and e is 8 in., what is the maximum allowable load P? Use the AISC allowable stresses.

5–38. Using the AISC allowable stresses, design a riveted connection of the type shown by Fig. 5-21(c) for a load P of 50 kips applied 10 in. from the face of the column. Use two rows of ¾-in. rivets spaced at 3-in. centers, and end distances of 1¼ in.

5–39. Design welded stiffened seats of the type shown by Fig. 5–21(d) for the following conditions. The setback is ½ in. in each case. Use the AISC allowable stresses. Use sections of rolled beams for the seats wherever possible; otherwise build up seats with plates. Use the curves of Fig. 5–27.

(a) 12 WF 40, reaction 21 kips.
(b) 16 WF 36, reaction 18 kips.
(c) 24 WF 84, reaction 29 kips.
(d) 15 I 50, reaction 25 kips.

5–40. Redesign the connection of Prob. 5–38 as a welded stiffened seat. Detail the connection completely, giving material sizes and lengths. For uniformity of class results, use only ⅜-in. fillet welds.

5–41. Rework Prob. 5–37, assuming that the rivets are replaced by ¾-in. turned bolts in reamed holes, tightened only enough to bring the parts in contact but not enough to cause appreciable initial tension. Use 4-in. angles and an end distance of 1¼ in.

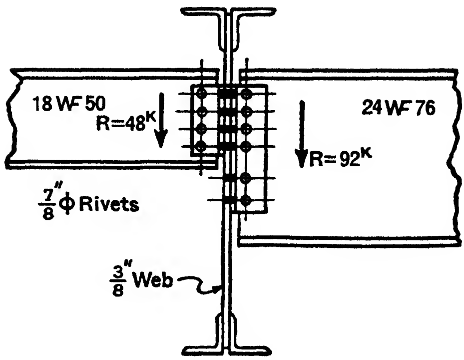


Fig. 5–51.

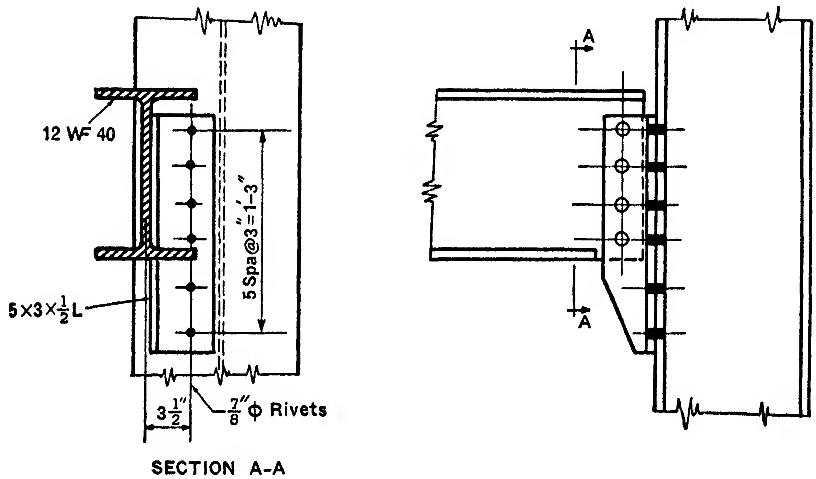


Fig. 5–52.

5–42. A 16 WF 36 beam is connected to the flange of a 10 WF 33 column by a web connection. The connection angles are 4 x 3½ x ⅜. Four ⅞-in. shop rivets connect the angles to the beam web, and 8 field rivets connect the angles to the column. Using the AISC allowable stresses, what is the maximum allowable load on the connection?

5–43. Is the connection shown by Fig. 5–51 satisfactory, according to the AISC specification? Note that the top eight field rivets each receive one-eighth of the reaction of one beam, plus one-twelfth of the reaction of the other.

5–44. Design web angle connections for a condition similar to that of Fig. 5–51, but consisting of a 16 WF 36 with R of 32 kips framing opposite a 21 WF 62 with R of 50 kips to the web of a 24 WF 76. Use ¾-in. rivets and the AISC specification.

5–45. What is the maximum load allowable on the eccentric web connection shown in Fig. 5–52? Use the AREA allowable stresses.

5–46. Design a web connection to connect a 14 WF 30 to the web of another beam of the same size (at right angles to the first beam). Use the AISC allowable stresses, ⅜-in. angles, and ¾-in. rivets. The beam reaction is 20 kips.

5–47. Two 12 I 31.8 beams, having reactions of 24 kips each, are connected opposite each other to the web of a transverse beam. The transverse beam has a $\frac{5}{16}$-in. web thickness. Design riveted web connections. Use ¾-in. rivets and the AREA allowable unit stresses (shear in rivets, 13,500 psi; bearing, 27,000 psi).

5–48. Design the single-angle web connection indicated by Fig. 5–53. Use ¾-in. rivets and the AISC specification.

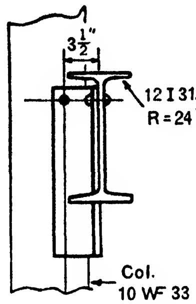


Fig. 5–53.

5–49. Redesign the web connection of Prob. 5–46 using fillet welds and 4 x 3 x ⅜ angles. Use the AISC specification.

5–50. Using the AISC specification, design welded web connections for the following beams:

(a) 12 WF 27, reaction of 20 kips.
(b) 14 WF 30, reaction of 30 kips.
(c) 16 WF 36, reaction of 40 kips.

5–51. Redesign the connection of Prob. 5–48 as a welded connection.

5–52. Design a web connection to connect an 18 WF 50, having a 28-kip reaction to the flange of a 10 WF 33 column. The connection is to be shop welded and field riveted. Use 4 x 3 x ⅜ angles and ¾-in. rivets. Use the allowable stresses of the AISC and AWS building specifications.

5–53. Design completely a connection of the type shown in Fig. 5–32 for the following conditions: beam size 18 WF 50; rivet size ⅞-in.; 3½-in. rivet spacing; beam reaction 57 kips; beam end moment 75 ft-kips. Use the AISC allowable unit stresses.

5–54. A 16 WF 36 beam has a reaction of 18 kips and an end moment of 25 ft-kips. Design completely a riveted connection of the type shown in Fig. 5–33(*a*), using 4 bolts in the top flange clip. Use the AISC specifications. Assume *M* and *P* act together, or separately.

5–55. Design completely a riveted connection for a 16 WF 36 having a reaction of 18 kips and an end moment of 70 ft-kips. Use a detail of the type shown in Fig.

5-33(*b*), but with four rivets in the field connection of each clip. Use the AISC specifications.

5-56. Redesign the connection of Prob. 5-54 as an all-welded connection of the type shown in Fig. 5-33(*c*). Use a stiffened tee-seat below. Use the AISC and AWS specifications.

5-57. Design a welded connection for the beams shown in Fig. 5-54. The two 14-in. beams are to be made continuous across the supporting beam. The end moment of the 14-in. beams is 30 ft-kips, and the reaction of each beam is 12 kips. Use the allowable stresses of the AWS and AISC specifications.

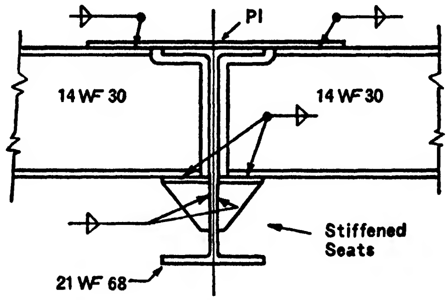


FIG. 5-54.

5-58. Design a riveted splice for two parts of a 10 WF 49 building column. The column is loaded axially with 150 kips. Assume the ends to be milled for full bearing. Use the AISC specification.

5-59. Redesign the splice of Prob. 5-58, assuming that the ends are not milled.

5-60. Redesign the splice of Prob. 5-58, using the AREA specification.

5-61. Design a full-milled riveted splice to connect a 12 WF 72 building column to a 12 WF 161 column. The axial load to be transferred is 270 kips. Use the AISC specification. Note that filler plates are required.

5-62. Design and detail a riveted tension splice for members each consisting of

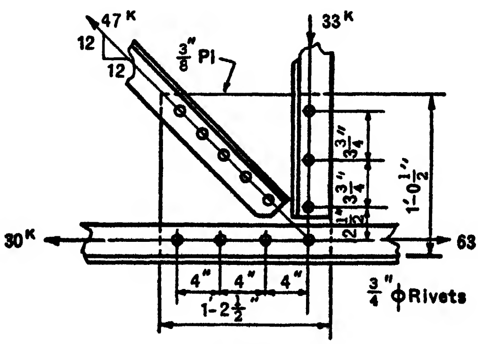


FIG. 5-55.

two angles 4 x 3 x $\frac{3}{8}$ (long legs spaced $\frac{3}{8}$ in. back-to-back). The axial load is 70 kips. Use $\frac{3}{4}$-in. rivets. Use splice material to connect each leg, one plate connected to the outstanding legs and one between the parallel legs. Assume that at the ultimate load of the splice all rivet shear planes are loaded equally. Use the AISC allowable stresses.

5–63. Redesign the splice of Prob. 5–62 using $\frac{1}{4}$-in. welds instead of rivets.

5–64. Investigate the approximate stresses on several sections of the gusset plate of Fig. 5-55.

CHAPTER 6

PLATE GIRDERS

6–1. The Plate Girder and Its Purpose. The plate girder is essentially an I-beam built up from structural shapes by riveting or welding. Rolled beams are available up to 36 in. in depth, weighing 300 lb per ft and providing a section modulus of 1,105 in.[3] When greater load-carrying capacity is required, plate girders may be employed. Frequently it is more economical to fabricate a plate girder than to use a rolled beam if greater space than 36 in. is available. For example, a plate girder having the load-carrying capacity of the largest 36-in. rolled section may be designed with approximately 70 per cent of the weight of the beam, if a depth of 6 ft is used. The unit price will, of course, be greater for the plate girder construction, so that economy is not in direct proportion to weight. Other considerations resulting in the choice of a plate girder section may be stiffness and available lengths of rolled beams. The 6 ft deep plate girder will probably have about 40 per cent as much deflection as the 36-in. beam for the same loading. Plate girders with spans well over 100 ft and with depths more than 10 ft are common. The Pittsburgh and Lake Erie Railroad erected a plate girder in 1936 having a span of 160 ft 2½ in. and depth over 11 ft. The Passaic and Hackensack River crossings of the New Jersey Turnpike are 375-ft plate girders.

6–2. Plate Girder Types. Plate girders are used for many purposes and are in many instances named by their use or by the manner of their framing

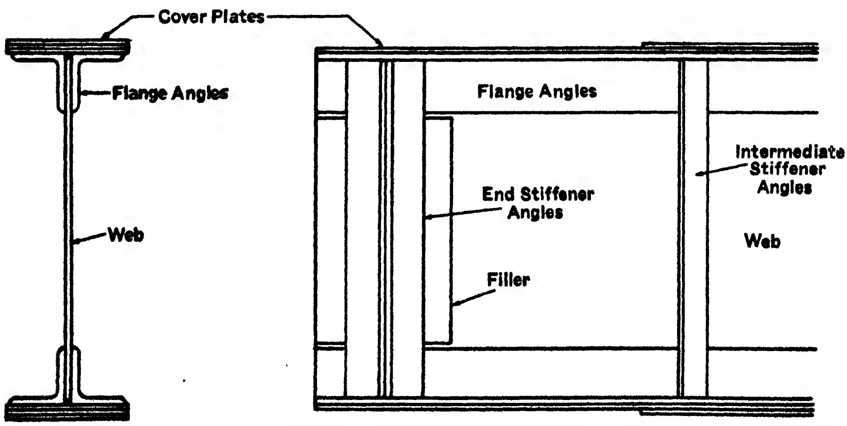


FIG. 6–1. Typical plate girder.

into a structure. Among these names are railway plate girder, highway plate girder, crane girder, deck girder, and through girder.

Figure 6–1 shows the main parts of an ordinary plate girder of the riveted type which include web, flange angles, cover plates, end stiffener angles, intermediate stiffener angles, and fillers. Figure 6–2 shows the cross section of a box girder in which two web segments have common cover plates.

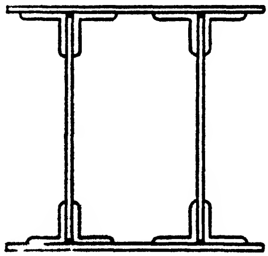


FIG. 6–2. Box girder.

Figure 6–3 shows a girder cross section in which side plates are used. Side plates are used in some special cases where it is not possible to provide

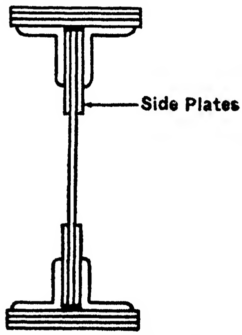


FIG. 6–3. Girder with side plates.

sufficient flange area otherwise. In general, side plates are uneconomical, increase the difficulties of splicing the web, and increase the number of fillers needed under stiffener angles.

Figure 6–4 shows three typical cross sections of welded plate girders. Two use flanges cut from rolled I-beams or WF beams with a plate welded between them to form a web; the other is entirely assembled from plates.

6–3. Stress Distribution on Cross Sections. The plate girder, whether riveted or welded, is analyzed as a beam. Complete design includes calculation of the attachment of the several parts to ensure that they will act as a unit.

The diagram of Fig. 6–5(*a*) shows the distribution of shear on a rectangular section. The value of shear intensity (either horizontal or vertical)

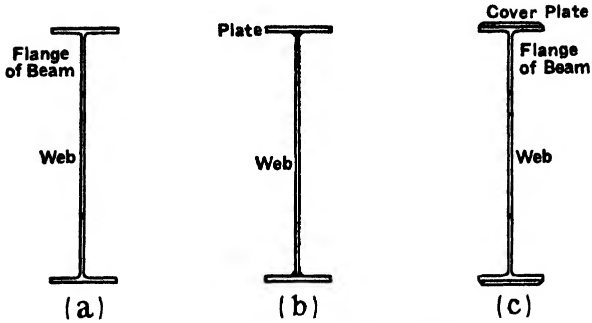


FIG. 6–4. Types of welded girders.

was found in the strength of materials to be $v = VQ/It$, in which V is the shearing force on the section, Q is the statical moment, about the center of gravity of the section, of those areas of the cross section farther removed from the neutral axis than the point in question, I is the moment of inertia

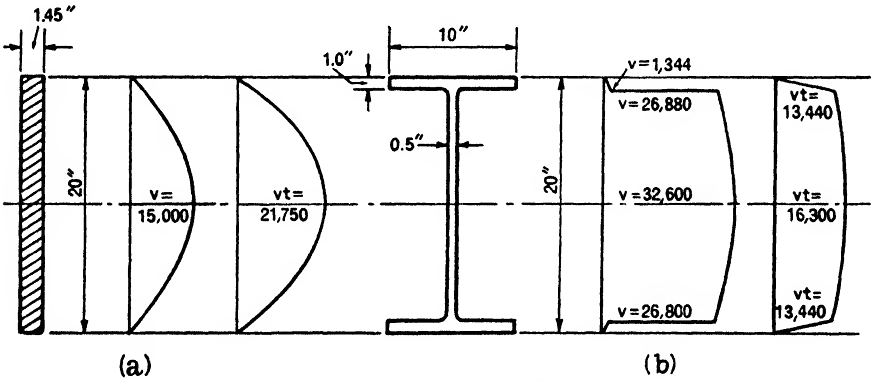


FIG. 6–5. Shear distribution.

of the acting cross section, and t is the thickness of the section. In the case of the rectangular section, it will be seen that I, t, and V are constants for any given loading condition and cross section. The value of Q is zero for extreme fibers and is maximum for fibers at the neutral axis. The distribution of shear on the cross section will be parabolic in shape, having a maximum intensity of 3/2 of the average shear intensity.

Since the unit shear is $v = VQ/It$, the shear force per inch of beam at any plane is $vt = t \cdot VQ/It = VQ/I$. Hence, the shear force per inch taken at any plane in a rectangular section is proportional to Q.

Figure 6–5(b) illustrates the nature of shear-stress distribution on a girder or I-beam section. In the case of an I-section, the value of t is not constant. In the flange area the value of t is the width of the flange; for the web the value of t is the thickness of the web plate. It will be seen that the value of Q increases rapidly from zero at the extreme fibers to a relatively high value at the bottom of the top flange and then increases slightly as the neutral axis is approached.

The following illustrative example shows the relation between shear distribution values on two cross sections having the same cross-sectional areas.

Assume that the sections shown in Fig. 6–5 are subjected to a shear force of 290,000 lb. In the rectangular section of (*a*),

$$I = \frac{1.45 \times 20^3}{12} = 967 \text{ in.}^4$$

The value of Q for a plane at distance y from the neutral axis is

$$(10 - y)1.45\frac{(10 + y)}{2} = \frac{145 - 1.45y^2}{2}.$$

The value of the shear intensity at the neutral axis ($y = 0$) is

$$v = \frac{VQ}{It} = \frac{290{,}000 \times 72.5}{967 \times 1.45} = 15{,}000 \text{ psi.}$$

The average intensity of shear is

$$\frac{V}{td} = \frac{290{,}000}{1.45 \times 20} = 10{,}000 \text{ psi.}$$

Thus, the maximum shear intensity is seen to be $1.5V/td$.

In the case of the I-section of Fig. 6–5(*b*),

$$I = (20 \times 9.5^2) + \frac{20 \times 1^3}{12} + \frac{0.5 \times 18^3}{12} = 2{,}050 \text{ in.}^4.$$

For the extreme fibers the value of $Q = 0$, hence $v = 0$. At the bottom edge of the top flange

$$Q = 10 \times 9.5 = 95,$$

and

$$v = \frac{290{,}000 \times 95}{2{,}050 \times 10} = 1{,}344 \text{ psi.}$$

At the top of the web,

$$v = \frac{290{,}000 \times 95}{2{,}050 \times 0.5} = 26{,}880 \text{ psi;}$$

at the center of the beam,

$$Q = (10 \times 9.5) + (4.5 \times 4.5) = 115.2,$$

and

$$v = \frac{290{,}000 \times 115.2}{2{,}050 \times 0.5} = 32{,}600 \text{ psi.}$$

The part of the shear load that will be taken by the flanges is approximately

$vt = 20 \times 1{,}344/2 = 13{,}440$ lb. The remainder of the shear load must be taken by the web, or the average shear intensity in the web is

$$\frac{290{,}000 - 13{,}440}{0.5 \times 18} = 30{,}730 \text{ psi.}$$

The computed shear intensity of 32,600 psi is approximately 8 per cent more than the average shear intensity in the web. If all the shear were assumed to be carried by the web, an average intensity would be $290{,}000/(0.5 \times 20) = 29{,}000$ psi, which is quite close to the computed maximum intensity. Specifications usually indicate that the average shear, computed with the web carrying all the shear, shall not exceed some specified allowable.

The relation between the distribution of bending moment stress on the two sections is also interesting. Assume that a bending moment of 2,000,000 in.-lb is to be taken by either of the sections. In the case of the rectangular section, the extreme fiber stress will be

$$f = \frac{Mc}{I} = \frac{2{,}000{,}000 \times 10}{967} = 20{,}680 \text{ psi.}$$

Since the thickness t is constant, the product tf will vary in the same manner as f.

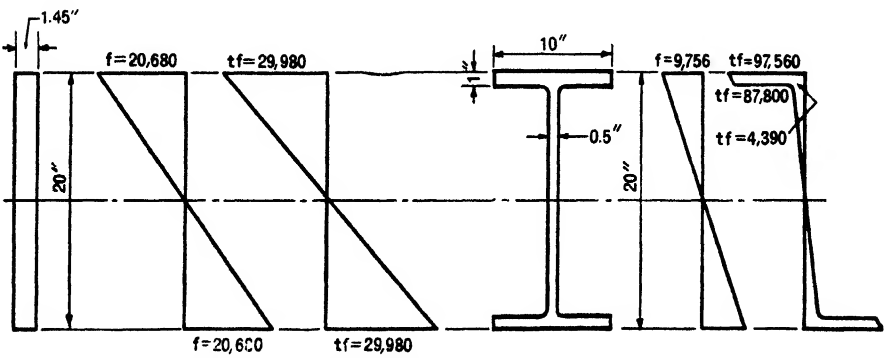


FIG. 6–6. Distribution of stress due to bending moments.

In the I-beam section of Fig. 6–6, the extreme fiber stress will be

$$f = \frac{Mc}{I} = \frac{2{,}000{,}000 \times 10}{2{,}050} = 9{,}756 \text{ psi.}$$

At the bottom edge of the top flange, the fiber stress will be

$$f = \frac{My}{I} = \frac{2{,}000{,}000 \times 9}{2{,}050} = 8{,}780 \text{ psi.}$$

The force carried by the fibers per inch of height is then tf, or 97,560 at the extreme fiber and 87,800 at the bottom of the flange. At the top of the web this value is $tf = 4{,}390$ lb per in.

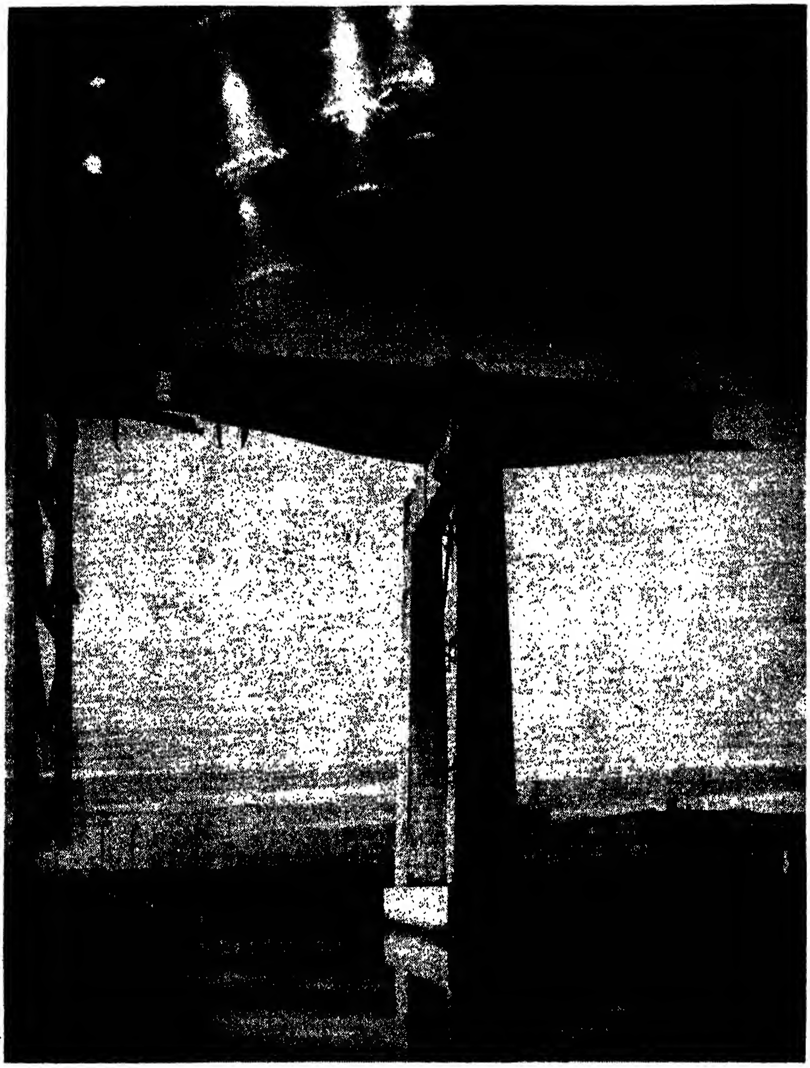
From the illustrated examples, it will be seen that the web carries the majority of the shear force while the flanges carry a majority of the bending loads. It is because of these facts that an approximate method of plate girder design has been used for many years. This method, known as the *flange area method,* is somewhat more rapid in most cases than the moment-of-inertia method, but it does not give satisfactory results for girders of unusual proportions or for shallow-depth girders.

The value of the flange area method is that it serves as a tool for establishing a trial section. The trial section may then be analyzed and adjustments made in the design. The method is approximate because of the assumptions that are made.

The problem in the design of plate girders usually starts with values for live-load shears and bending moments, and estimated dead loads. General dimensions of the girder must be selected first. The economical depths of plate girders are usually from $\frac{1}{10}$ to $\frac{1}{12}$ of the span lengths, but girders are frequently constructed with depths between $\frac{1}{8}$ and $\frac{1}{14}$ of the span lengths.

After selection of a girder depth, a web design may be made that provides sufficient cross section to resist the shear and that meets minimum thickness requirements. The next step in the development of a trial section is the selection of material for the flanges. Girder flanges in riveted design are usually composed of plates and angles. The best economy is accomplished when a considerable portion of the flange is in the cover plates and when several cover plates are provided. In this way each cover plate may be used for only that part of the length of the girder in which it is required. The portion of the flange area usually in the cover plates is from 40 per cent to 60 per cent of the total flange. In general, material should be selected that is not too thick to be punched in the fabricating shop but that is thick enough so that it will not buckle locally when it is used in field work construction.

The flange-area method assumes that stress distribution over the flange is uniform and that the resultant force is at the center of gravity of each flange. Inspection of Fig. 6–6 shows that the moment of resistance of a beam is provided principally by the flanges. The web contributes a small amount to the moment of resistance. The bending stress in the web is seen to vary from zero at the neutral axis to maximum at the flange. If it is further assumed that the neutral axis is at the center of the section, an expression for the equivalent value of the web may be established. Let the flange stress be f, the distance between resultant flange forces d, and the thickness of the web t. The force taken by the web on one side of the neutral axis is then $ftd/4$, and the moment of resistance of the web is $(ftd/4) \times (2d/3) = ftd^2/6$. The area that could be applied at the center



Bethlehem Steel Co.

Erection of Passaic River Bridge, Newark, N. J.

of the flange to provide a moment of resistance about equivalent to that of the web is then $td/6$, or one-sixth of the web area.

On the tension side of a girder, rivet holes may reduce the effective flange area. Additional flange area must be provided to compensate for the area taken out by rivet holes. If 7/8-in. diam rivets are spaced at 4 in. through the web, only three-fourths of the web remains effective for tension. Hence on the tension side this method uses 3/4 of $td/6 = td/8$ as the equivalent area of the web.

When the depth of the girder has been selected, the location of the flange centroid and a trial effective depth may be assumed. The bending moment may be divided by the effective depth and by the allowable flange stress to determine a trial flange area. The web area equivalent is then subtracted from the total required flange area, and the remaining area is used to select the elements of the flange. The center of gravity of the trial section may then be calculated and the assumed effective depth checked.

The flange-area method usually results in flanges that are too small. This is because the stress is not uniformly distributed across the flange as is assumed. Shallow-depth heavy girders are affected more by this assumption than are deep light sections.

The present design practice does not depend on the approximate flange-area method for final analysis but requires that the stresses be computed from the moment of inertia of the section. For this reason, the several steps of the flange-area method may be simplified for the purpose of determining a trial section. The authors have found that a satisfactory trial flange section may usually be obtained by one assumption of equivalent effective depth. When about one-half of the flange area is to be in the cover plates, this equivalent effective depth is equal to the depth of the girder from back-to-back of flange angles. No reduction in flange area is then made for the effect of the web. When a smaller percentage of the flange is to be in the cover plates, the equivalent effective depth is less than back-to-back of flange angles.

The student will recognize that neither of the methods suggested is accurate from a stress analysis consideration and he will undoubtedly use the method that is most direct. Experience in selection of trial sections will usually give more satisfactory results. The work that follows uses the simplified method.

6-4. Design Procedure. The design of a plate girder should follow a well-organized procedure. It is, of course, necessary to start with a trial section. The amount of load to be carried by the girder includes its own weight. Hence, it is necessary to make a preliminary estimate of the weight of the girder. This can be accomplished by some approximate method, such as the simplified flange-area method, before carrying out the analysis of the girder.

The following order of procedure is usually used:

1. Compute the known loads, making an allowance for the weight of the girder.
2. Compute the maximum bending moment and maximum shear values.
3. Determine the over-all depth of the girder or the distance back-to-back of flange angles.
4. Determine the web thickness and depth.
5. By an approximate flange-area method determine the required flange area and select a trial section. Check dead load assumptions.
6. Compute the moments of inertia of the cross sections of the girder, first with all cover plates included, then with one pair removed, two pairs removed, etc. Holes in the tension side of the section must be assumed, and corrections must be made in the computations if the assumption is later found to be inaccurate.
7. Compute unit stresses in the extreme fibers at the point of maximum moment for the full section and make whatever adjustment may be necessary.
8. Compute the bending moment for sufficient points along the span to determine the lengths of cover plates.
9. Design the end stiffener angles to transfer the reaction to the girder bearings and other stiffeners at any points of concentrated loads.
10. Design intermediate stiffeners and locate panel points for lateral bracing.
11. Check the allowable compressive stresses against the computed stresses with consideration of actual unsupported lengths.
12. Compute the spacing of rivets to attach flanges to web, cover plates to flange angles, and stiffeners to web.
13. Design end details, lateral bracing, connections, bearings, etc.

6–5. Plate Girder Specifications. Specifications for the design of plate girders usually include clauses specifying allowable unit stresses, depth-to-span ratios, the make-up of flanges, strength requirements for splices, spacing and size of stiffeners, web thickness, and bracing of compressive flanges. The AREA and AISC specifications include requirements governing the methods for calculation of unit stresses in plate girder flanges. Since the requirements of these two specifications are so radically different, some comparison between the results obtained from their use should be made by the student.

The AISC building specification states:

> Riveted and welded 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 . . . exceeds 15 per cent of the gross flange area, the excess shall be deducted. If such members contain other holes, as for bolts, pins, countersunk rivets, or plug or slot welds, 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.

The AREA bridge specifications cover the same subject in a similar manner.

Plate girders, I-beams, and other members subject to bending that produces tension on one face, shall be proportioned by the moment-of-inertia method. The neutral axis shall be taken along the center of gravity of the gross section. The tensile stress shall be computed from the moment of inertia of the entire net section and the compressive stress from the moment of inertia of the entire gross section.

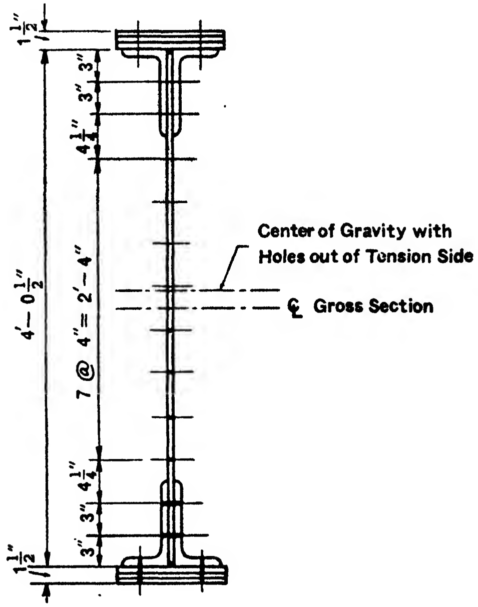


FIG. 6-7.

Figure 6–7 represents the cross section of a plate girder which will be analyzed by both specifications, and the results of which will be compared with the more classical method in which holes are deducted from only the tensile area.

PROPERTIES OF CROSS SECTION

	A	y	Ay^2	I_0
6—10 x ½ Pls	30.00	25.00	18,750	....
4—8 x 4 × ¾ Ls	33.76	21.30	15,317	220
1—48 x ½ web Pl	24.00	0	0	4,608
	87.76		34,067	4,828
			4,828	
			I = 38,895 in.⁴	

$$\text{Section modulus} = \frac{38{,}895}{25.75} = 1{,}510 \text{ in.}^3$$

HOLES OUT OF TENSION SIDE OF GIRDER

	A	y	Ay	Ay^2
2—2.25 x 1.0	− 4.50	24.62	−110.79	−2,728
1—2.00 x 1.0	− 2.00	21.25	− 42.50	− 903
1—2.00 x 1.0	− 2.00	18.25	− 36.50	− 666
1—0.50 x 1.0	− 0.50	14.00	− 7.00	− 98
1—0.50 x 1.0	− 0.50	10.00	− 5.00	− 50
1—0.50 x 1.0	− 0.50	6.00	− 3.00	− 18
1—0.50 x 1.0	− 0.50	2.00	− 1.00	− 2
	−10.50		−205.79	−4,465

PROPERTIES OF SECTION WITH HOLES OUT OF TENSION FLANGE

Gross area =	87.76
Holes	−10.50
Net area	77.26

$$\text{Shift of neutral axis} = \frac{205.79}{77.26} = 2.66 \text{ in.}$$

I for gross section	= 38,895
I for holes out	= −4,465
Effect of shift of neutral axis = 2.66 × 205.8	= −547
Net I	= 33,883 in.^{4}

$$\text{Section modulus, top flange} = \frac{33{,}883}{23.09} = 1{,}467 \text{ in.}^3$$

$$\text{Section modulus, bottom flange} = \frac{33{,}883}{28.41} = 1{,}193 \text{ in.}^3$$

PROPERTIES OF SECTION WITH HOLES OUT OF BOTH FLANGES

Gross I	= 38,895
Holes	−8,930
Net I	= 29,965 in.^{4}

$$\text{Section modulus} = \frac{29{,}965}{25.75} = 1{,}164 \text{ in.}^3$$

When holes are deducted from the tension area of the girder but not from the compression area, the section modulus for compression is found to be 1,467 in.^{3} and that for tension 1,193 in.^{3} Computed according to the AREA specification, the section modulus for the entire gross section is 1,510 in.^{3} and for the entire net section 1,164 in.^{3} It will be seen that stresses computed according to the AREA specification will be within about 3 per cent of the values obtained by deducting holes from only the tension flange, when the corresponding shift of the neutral axis is considered.

In considering the validity of the various approaches to stress computation for plate girders, there are two principal bases of comparison. If it is desired only to determine the unit stress for some particular loading, which produces maximum stress at values approximating design values, consideration should be given to the probability that all rivet holes in the compression flange may not be completely filled. As a consequence, for symmetrical sections, the neutral axis would remain at the center of gravity of the entire section; the maximum tension would be equal to the maximum compression and would occur on a section through the holes in each flange. Since stability of the compression flange is determined by the average compression rather than the maximum, the value between rivet holes would be close to the value desired. Thus, it would seem, the AREA specification would provide a method of obtaining unit stresses at design loads which should be correct except for the effect of stress raisers. A plate girder usually receives its maximum stresses only once during the passage of a train, and the loading is less than the design loading for a rather large percentage of cases. One hundred trains a day for approximately 56 years will load the girder about two million times. If the effect of stress raisers in the vicinity of rivet holes is to produce a maximum stress equal to the fatigue limit at two million cycles, failure could be expected at that time. (See Chap. 10.)

For conditions of overloading (load above design load), or if the rivets in the compression flange are tight, it is likely that the rivets transfer compression across the holes. In this case it would seem more nearly correct to consider that holes reduce the effective area of the tension flange only and that the neutral axis shifts toward the compression flange because of the reduction in the tension flange area. Tests* have shown rather conclusively that tension stresses in the flange are no less than those computed by deducting rivet holes in the tension flange and that there is a shift of the neutral axis toward the compression flange.

Consideration of a proper factor of safety at design load requires a study of fatigue characteristics. If it is apparent that fatigue failure is not probable, the factor of safety would need to provide for the occasional overload, reduction of sections due to corrosion, and any imperfections.

The AISC specification requires that only the rivet hole areas in excess of 15 per cent of the gross flange area need be deducted and that such deduction be made from both flanges. Since the specification does not specify which rivet holes are to be considered as including the excess for deduction, it will be assumed that it is divided among the rivet holes in proportion to their contribution to the total reduction. The gross area of a flange of the girder of Fig. 6–7 is 31.88 sq in.; 15 per cent of 31.88 is 4.78 sq in.; the rivet holes reduce the flange area by 7.50 sq in.; hence 2.72 sq in. is the excess reduction to be subtracted. This area is distributed over both flanges.

* S. B. Lilly and S. T. Carpenter, "Effective Moment of Inertia of a Riveted Plate Girder," *Proc. ASCE*, October 1939, and E. Mirabelli and C. D. Williams, "Discussions," *Proc. ASCE*, January 1940.

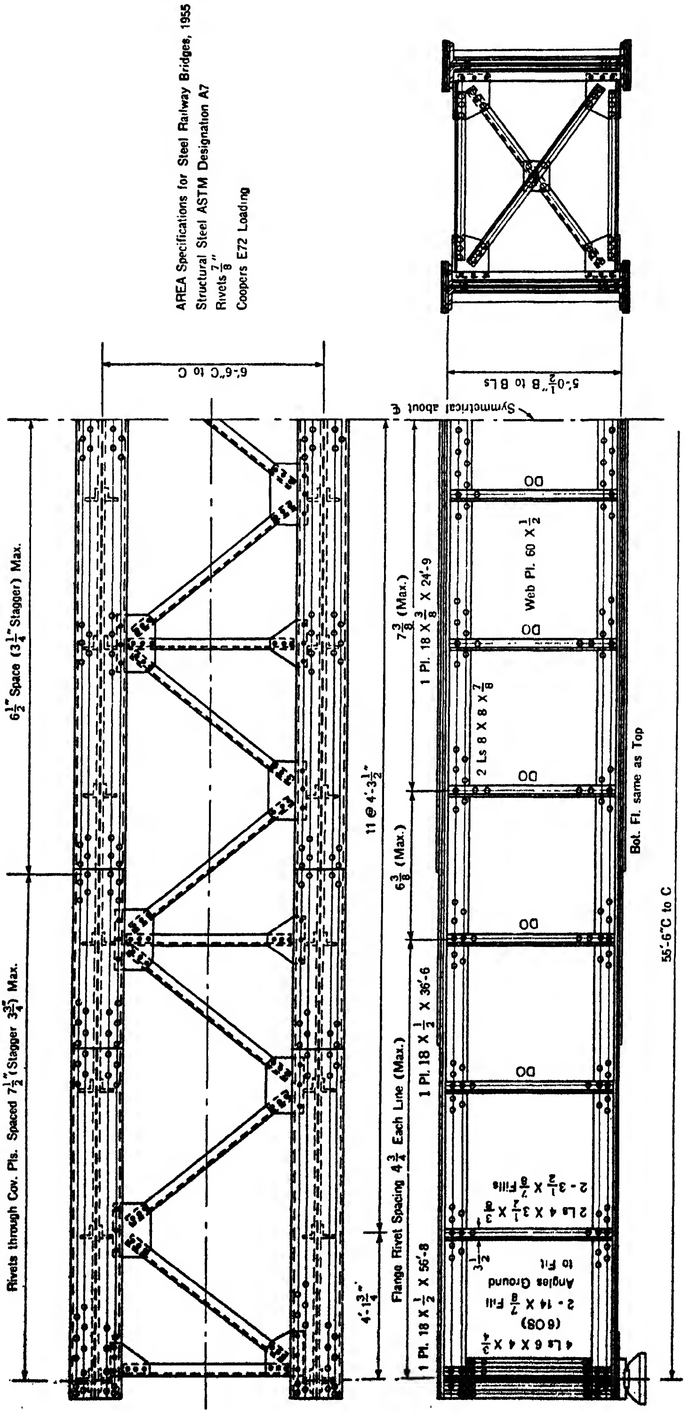


FIG. 6-8. Railway deck plate girder design.

$$\frac{4.5}{7.5} \times 2.72 = 1.63 \text{ sq in. at } 24.62 \text{ in. from the centerline,}$$

$$\frac{1.5}{7.5} \times 2.72 = 0.54 \text{ sq in. at } 21.25 \text{ in. from the centerline,}$$

$$\frac{1.5}{7.5} \times 2.72 = 0.54 \text{ sq in. at } 18.25 \text{ in. from the centerline.}$$

Since the reduction is to be made from both flanges, the following reduction in moment of inertia will be made from that computed for the gross section:

Gross moment of inertia		38,895 in.^{4}
$-2 \times 1.63 \times 24.62 \times 24.62 =$	$-1,976$	
$-2 \times 0.54 \times 21.25 \times 21.25 =$	$-\ \ 487$	
$-2 \times 0.54 \times 18.25 \times 18.25 =$	$-\ \ 359$	$-2,822$
		36,073 in.^{4}

According to the AISC specification, a section modulus of 36,073/25.75 = 1,401 in.^{3} would be used for both tension and compression. Since this specification is expected to be used for building loads not subject to fatigue-type repetitions, average stress through rivet holes will likely be the criterion for tension failure. If a computed unit stress of 20,000 psi is found from the section modulus of 1,401 in.^{3}, the unit stress based on a section modulus of 1,193 in.^{3} will be (1,401/1,193) $\times$ 20,000 = 23,500 psi. Rather than a factor of safety of 33,000/20,000 = 1.65 with respect to yield point, the probable factor of safety is 33,000/23,500 = 1.41. This factor of safety may be perfectly satisfactory for the type of loading contemplated, but it should be recognized in the design.

6–6. Design of a Plate Girder. In the discussion that follows, a railway plate girder designed in accordance with the AREA specifications is used as an example. The design of plate girders for other types of loading is not particularly different, except in certain detail requirements of the specifications. Figure 6–8 illustrates a deck plate girder bridge with a span of 55 ft 6 in. between centers of end bearings.

The girders of deck spans should be well spread to give stability to the structure and to minimize the effect of inequalities in loading, caused by such influences as rolling of the locomotive. It will be noted that the AREA specifications set a minimum of 6 ft 6 in. between the centers of the girders and a distance of not less than one-fifteenth of the span. In the design that follows, a spacing of 6 ft 6 in. is used. The preferred depth of girders is at least one-twelfth of the span, which in this case is 55.5 in. Assuming that sufficient depth is available, a depth of 5 ft is selected. If a web plate 60 in. wide is used, the flange angles should be 60.5 in. back-to-back to provide for irregularities in the edges of the plate and for ease in fabrication.

6-7. Trial Section of Girder. Cooper's E-72 loading is the most common railway design loading at the present time and will be used in this example. The value of the maximum bending moment for a span of 55 ft 6 in. may be determined from any table of Cooper's loadings. The dead load will need to be estimated. Impact is computed in accordance with the AREA specifications (see Section 20 of AREA specifications in Appendix).

The calculations that follow are shown to an exactness obtained by the use of a calculating machine. While this is the practice of modern design offices, no accuracy is sacrificed by use of the slide rule.

Live-load moment (from tables of Cooper's loadings) = 2,032,000 ft-lb

$$\text{Impact moment, } 60 - \frac{55.5^2}{500} = 53.9\%$$

$$\text{Rolling, 10\% of axle loads} = \frac{20 \times 5.0}{6.5} = \frac{15.4}{69.3} = 1{,}408{,}176$$

Dead-load moment,

Track dead load	200 lb per ft of track
8 in. x 10 in. ties 10 ft long	330
	530 lb per ft of track

530 ÷ 2 = 265 lb per ft of girder
600 lb per ft, estimated weight of girder
865 lb per ft, estimated dead load on girder

$$\frac{865 \times 55.5 \times 55.5}{8} = 333{,}052$$

Design bending moment = 3,773,228 ft-lb

An effective-depth (or flange-area) method may be used for obtaining the trial section. Assuming that the centers of the tensile and compressive forces are approximately at the backs of the angles, the net area required for the tension flange may be found by dividing the bending moment by the effective depth and the allowable stress in tension (18,000 psi).

$$\text{Net area required for tension flange} = \frac{3{,}773{,}228 \times 12}{60.5 \times 18{,}000} = 41.6 \text{ sq in.}$$

Assumed area of holes deducted, 20 per cent of net area = 8.3
Approximate gross flange area required = 49.9 sq in.

The top flanges of a deck girder are subjected to corrosive influences because of the contact with the ties. For this reason, it is customary to provide somewhat more top flange area than the stress requirements might require. The allowable compressive stress is also less than the allowable tension stress and is determined by the unsupported length of the top flanges. No deductions for rivet holes are made in the area of the top flange because it is assumed that the rivets fill the holes. Hence, if the top flange is made like the bottom flange, there will likely be sufficient area to satisfy the stress requirements and to provide some extra section to safeguard against corrosion. Supports for the top flange can be provided as close together as desired, since there will be no interference with operation. It will be noted that the specifications require that the compression flange be at least as large as the tension flange. It is also desirable to have approximately 50 per cent of the flange area in cover plates to permit variation in section to meet the reduced bending requirement toward the ends of the girder; 50 per cent of 49.9 sq in. = 25.0 sq in.

Two 8 x 8 x ⅞ angles will provide 26.46 sq in. The width of the angles and web will then be somewhat over 16 in. and cover plates 18 in. wide may be used. The trial section will then have flanges as follows:

2 − 8 × 8 × ⅞ Ls	= 26.46 sq in.
18 × 1⅜ cov. Pls	= 24.75
Total flange area	= 51.21 sq in.

The required gross area of the web will be determined from the maximum end shear, or the amount of the girder reaction:

Live-load reaction (from tables of Cooper's loadings)	= 166,600 lb
Impact reaction, 69.3 per cent of live load	= 115,450
Dead-load reaction, 865 × 27.75	= 24,000
End reaction	= 306,050 lb

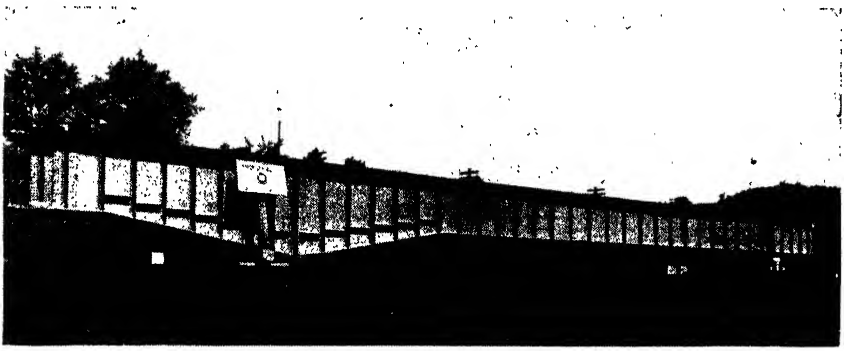
The area of the web required for stress is 306,050/11,000 = 27.8 sq in. A plate 60 x ½ will provide 30 sq in. The thickness of the web should also have a proper ratio to the unsupported height to ensure that it will act as a rigid web and not buckle. The ratio of the clear distance between flanges to the thickness of the web is 48.5/0.5 = 97, which is less than the ratio of 170 permitted by the specifications but greater than the ratio of 60 above which the web must be stiffened.

The approximate weight of the trial section is 3.4(51.2 + 51.2 + 30) = 450 lb per ft. Providing an allowance of 30 per cent for details and lateral bracing, the anticipated weight is 450 + 135 = 585 lb per ft. The estimated weight of 600 lb per ft is close enough for the purpose of calculating dead-load effects.

6–8. Moment of Inertia of Girder. The critical section of a girder usually occurs at a stiffener because of the rivet holes necessary for its attachment. The cover plates will be attached with two gage lines in each flange angle, but these rivets may be staggered so that only those on one line through each angle need be deducted. The staggered pitch, which must be exceeded to make the net section through two rivet holes critical in the cover plates, may be computed by making $s^2/4g = 1.0$, or $s^2/12 = 1.0$, and s = 3.46 in. When the attachment of cover plates to the flange angles is computed (Art. 6–10), the rivet spacing may be compared with the above minimum requirement.

The calculation of properties of the girder cross section is in accordance with the AREA specification, which uses the center of the section as the neutral axis.

It will be necessary to determine the lengths of cover plates, which will be of equal length for both top and bottom flanges; hence the computations list each pair of cover plates as an item. Having the gross and net moments



American Bridge Co.

GIRDER FOR PENOBSCOT RIVER BRIDGE, BANGOR, ME.

of inertia for the full section with all cover plates, deduction is made from the total figures to provide similar properties for the section with one pair of cover plates removed, then with two pairs of cover plates removed.

Section moduli are computed from the values of moment of inertia. It will be assumed that the compression flange will be braced, so that its compressive value will develop at least as much bending moment as the tension flange. Moments of resistance are then computed from the section moduli for the tension flange. These values will be used to determine the lengths of cover plates.

A preliminary layout of the probable location of rivets on the cross section must be made in order to calculate section properties. Figure 6–9 is a cross section of the girder showing the holes that are to be deducted in the calculation. While the standard gage for an 8-in. leg is 3 in., a dimension of 3.5 in. is used because of splice details to be developed later. Four rows of rivets will be used throughout the cover plates of each flange. They will be staggered, however, so that a complete deduction for these holes will not be made. The staggered pitch of rivets attaching the cover plates at the center of the span is governed by Section 47 of the AREA specifications: $4 + 4t - (3g/4) = 4 + 1.5 - 2.25 = 3.25$ in. maximum. The net section of the cover plates will be computed in accordance with

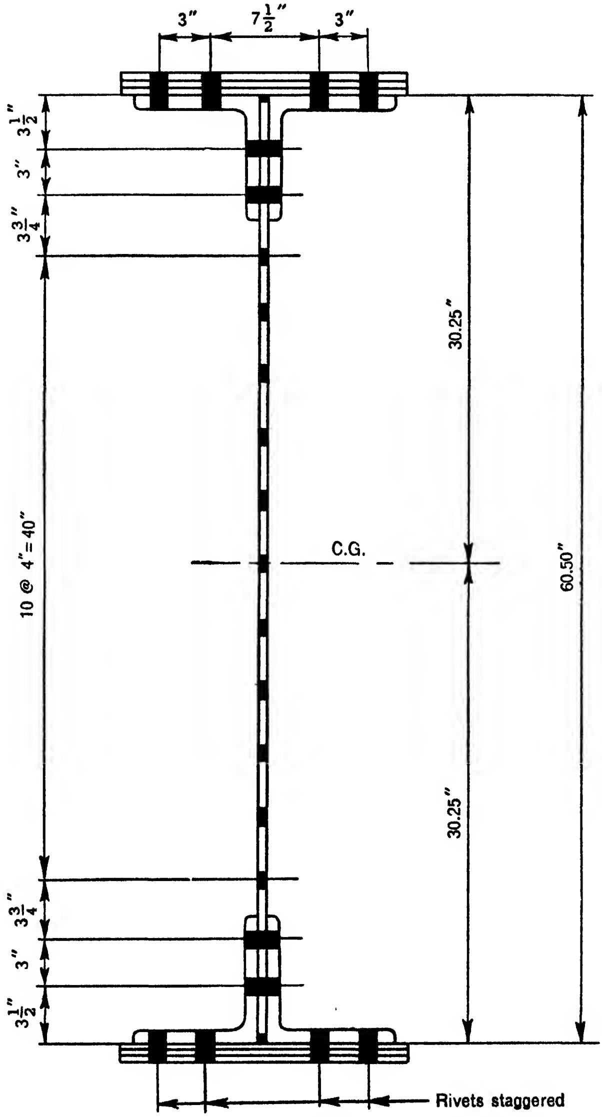


FIG. 6–9.

Section 44 of the AREA specifications. Four holes are deducted from the cover plates and the quantity, $2s^2/4g = (2 \times 3.25^2)/(4 \times 3) = 1.76$ in., is added. The number of holes to deduct from the horizontal elements of the flange will then be $4 - 1.76 = 2.24$. The stiffeners should be attached to the flange angles and web with two rivets; hence, two holes are deducted from the vertical legs of these elements.

FULL SECTION

Section	*A*	*y*	$Ay^2 + I_0$
2—18 x 3/8 Pls.	13.50	31.44	13,344
2—18 x 1/2 Pls.	18.00	31.00	17,298
2—18 x 1/2 Pls.	18.00	30.50	16,744
4—8 x 8 x 7/8 ∟s	52.92	27.93	41,600
1—60 x 1/2 web Pl.	30.00	....	9,000
Gross area	132.42 in.^{2}		97,986 in.^{4}
Holes			
4—2.25 x 1.00	−9.00	30.50	−8,372
2—2.25 x 0.24	−1.08	30.50	−1,005
2—2.25 x 1.00	−4.50	26.75	−3,220
2—2.25 x 1.00	−4.50	23.75	−2,538
2—0.50 x 1.00	−1.00	20.00	− 400
2—0.50 x 1.00	−1.00	16.00	− 256
2—0.50 x 1.00	−1.00	12.00	− 144
2—0.50 x 1.00	−1.00	8.00	− 64
2—0.50 x 1.00	−1.00	4.00	− 16
1—0.50 x 1.00	−0.50	0	0
Net area	107.84 in.^{2}		81,971 in.^{4}

$$S_c = \frac{97{,}986}{31.63} = 3{,}098 \text{ in.}^3$$

$$S_t = \frac{81{,}971}{31.63} = 2{,}592 \text{ in.}^3$$

TWO COVER PLATES TOP AND BOTTOM

Section	*A*	*y*	$Ay^2 + I_0$
Full gross section	132.42	—	97,986
2—18 x 3/8 Pls.	−13.50	31.44	−13,344
Gross—2 cov. Pls.	118.92 in.^{2}		84,642
Full net section	107.84	—	81,971
2—18 x 3/8 cov. Pls.	−11.82	31.44	−11,684
Net—2 cov. Pls.	96.02 in.^{2}		70,287 in.^{4}

$$S_c = \frac{84{,}642}{31.25} = 2{,}708 \text{ in.}^3$$

$$S_t = \frac{70{,}287}{31.25} = 2{,}249 \text{ in.}^3$$

ONE COVER PLATE TOP AND BOTTOM

Section	*A*	*y*	$Ay^2 + I_0$
Gross—2 cov. Pls.	118.92	—	84,642
2—18 x 1/2 cov. Pls.	−18.00	31.00	−17,298
Gross—1 cov. Pl.	100.92		67,344 in.^{4}
Net—2 cov. Pls.	96.02	—	70,287
2—18 x 1/2 cov. Pls.	−15.76	31.00	−15,145
Net—1 cov. Pl.	80.26 in.^{2}		55,142 in.^{4}

$$S_c = \frac{67{,}344}{30.75} = 2{,}190 \text{ in.}^3$$

$$S_t = \frac{55{,}142}{30.75} = 1{,}793 \text{ in.}^3$$

MOMENTS OF RESISTANCE—TENSION

Full section	$2{,}592 \times 18{,}000/12 = 3{,}888{,}000$ ft-lb
2 cov. Pls.	$2{,}249 \times 18{,}000/12 = 3{,}373{,}000$ ft-lb
1 cov. Pl.	$1{,}793 \times 18{,}000/12 = 2{,}689{,}000$ ft-lb

The trial section provides sufficient moment of resistance for the bending moment of 3,773,228 ft-lb.

6–9. Length of Cover Plates. The AREA specifications require that there be at least two rows of rivets attaching each cover plate beyond the theoretical end of the cover plate and that at least one cover plate extend the full length of each flange. The problem is to determine the points at which cover plates are no longer needed. Probably the most popular method is to plot the diagram of maximum bending moments and to superimpose the moment of resistance diagram upon it. In Fig. 6–10 the upper shaded line represents the moment of resistance of the girder, with the values plotted to scale for sections having one cover plate top and bottom, two cover plates top and bottom, and three cover plates. The curved line represents a diagram of maximum bending moment plotted to the same scale as the moment of resistance and including live-load, impact, and dead-load moments. The maximum bending moment diagram may be developed by computing the values for a few points, or an approximate method may be

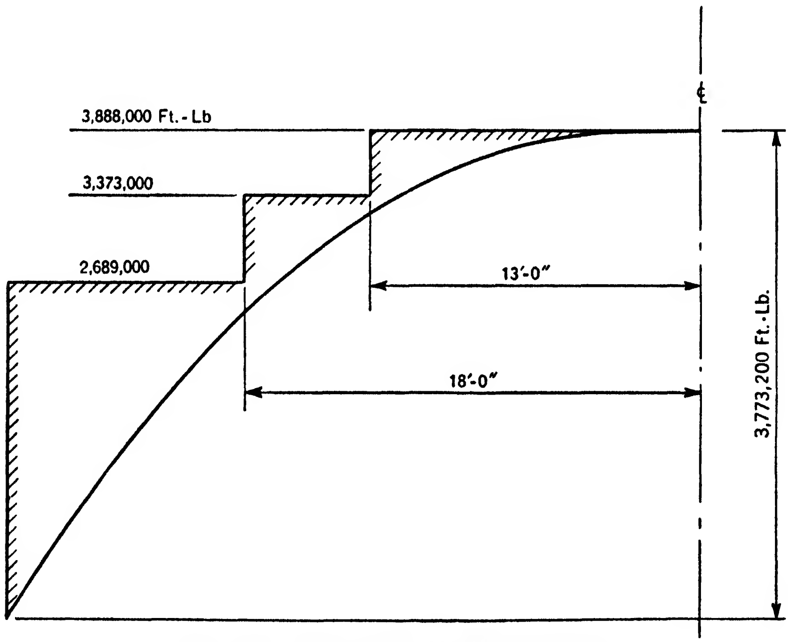


FIG. 6–10. Moment of resistance diagram.

used. The suggested approximation for Cooper's E-loadings* is developed by assuming that there is a uniform moment intensity over the middle one-tenth of the span and that the curve is parabolic between the ends and the $0.45L$ point from each end. The approximation has been found to be very close for most ratios of dead and live load.

* This method was used by the New York Central Bridge Department as early as 1928.

The intersections of lines representing moments of resistance with the diagram of maximum moment provide the points at which the larger section is no longer needed. The actual ends of cover plates will then be somewhat closer to the ends of the girder to provide for the partial development of section required by the specifications.

The lengths of cover plates may be computed analytically when the maximum bending moment is expressed by some simple algebraic law. For example, when the loading is entirely uniform and the bending moment is parabolic, the length of any cover plate will be

$$X = L\sqrt{\frac{M_a - M_b}{M_a}},$$

in which L is the span, M_a is the moment of resistance required at the center, and M_b is the moment of resistance at the end of the cover plate. An extra length of cover plate would be allowed beyond this theoretical length.

In the case of the railway plate girder being designed, the curve of the maximum bending moment is assumed to be constant over the central one-tenth of the span length and then to vary along a parabola. Hence, the theoretical length of any cover plate may be expressed as

$$X = 0.1L + 0.9L\sqrt{\frac{M_a - M_b}{M_a}}.$$

The theoretical lengths of the cover plates will then be as follows:

$$5.55 + 49.95\sqrt{\frac{3{,}773 - 3{,}373}{3{,}773}} = 21.93 \text{ ft},$$

$$5.55 + 49.95\sqrt{\frac{3{,}773 - 2{,}689}{3{,}773}} = 32.32 \text{ ft}.$$

6–10. Riveted Attachment of Flanges. The rivets through the cover plates must have sufficient value to develop the shear transfer from flange

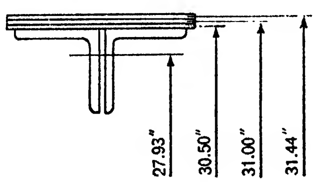


Fig. 6–11.

angles to cover plates; the flange angles must be attached to the web with sufficient rivets to transfer the shear increment from the web to the entire flange. The shear between any two elements per linear inch is represented by the shear expression, $q = VQ/I$, in which V is the vertical shearing load at the section, Q is the statical moment about the neutral axis of that part of the girder to which shear is being transferred, and I is the moment of inertia of the girder at the point being considered.

The following computations for the various sections of the girder are based on the gross sections. Figure 6–11 shows the flange section considered.

Figure 6–12 is a diagram of maximum shear for dead load, live load, and impact. At the end of the girder the shear is $V = 306{,}050$ lb; the value

Section	A	y	Ay	$\Sigma Ay = Q$	I	$\frac{Q}{I}$
2—8 x 8 x ⅞ ⦣s	26.46	27.93	740			
1—18 x ½ cov. Pl.	9.00	30.50	275	1,015	67,344	0.0151
1—18 x ½ cov. Pl.	9.00	31.00	279	1,294	84,642	0.0153
1—18 x ⅜ cov. Pl.	6.75	31.44	212	1,506	97,986	0.0154

of I with one cover plate top and bottom $= 67{,}344$, and $Q = 1{,}015$ for the cover plate and angles. The transfer of shear from web to flange is $q = VQ/I = 0.0151 \times 306{,}050 = 4{,}621$ lb per in.

The rivets attaching the flange to the web must also transfer the local tie loads from the flange to the web. The specifications require that the wheel load be considered as distributed over 3 ft of length. Hence, the maximum wheel load of 36,000 lb will provide 1,000 lb per in. of flange; impact of 80 per cent (see AREA Section 62) = 800 lb per in.; the track dead load of 265 lb per ft = 22 lb per in.; the total local vertical load per inch = 1,822 lb. Combining the vertical and horizontal loads,

$$w = \sqrt{(1{,}822)^2 + (4{,}621)^2} = 4{,}968 \text{ lb per in.},$$

which is the resultant load to be transferred through the rivets.

The value of a ⅞-in. rivet in single shear at 13,500 psi = 8,120 lb, in double shear = 16,240 lb, and in bearing on a ½-in. plate at 27,000 psi = 11,810 lb. The rivet spacing for attachment of the flange to the web is then controlled by the bearing value of 11,810 lb per rivet and at the end of the girder will be 11,810/4,968 = 2.38 in. maximum. If rivets are provided in two rows, each row will have a maximum spacing of 4.76 in.

The value of I/Q is approximately equal to the depth of the girder. Because of this fact, an "effective-depth" method of spacing rivets gives approximately correct values for rivet spacing. The effective depth of the girder is equal to the distance between the centers of gravity of the top and bottom flanges. For this section the effective depth is found to be 58.37 in. The reasoning used is that the increment of bending moment per linear inch is equal to the shear load and that the flange takes all the stress due to bending. This results in an increment of flange force per

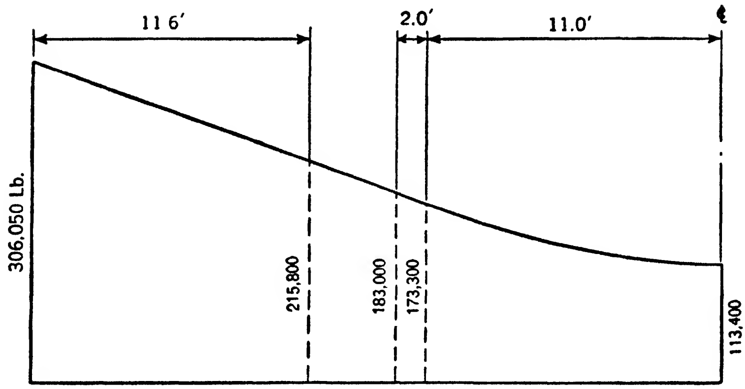


FIG. 6–12. Diagram of maximum shear.

linear inch equal to V/d, in which d is the effective depth. In this case the load per inch of flange would be $313{,}720/58.37 = 5{,}243$ lb.

The local tie load must be combined with the 5,243-lb load, or

$$w = \sqrt{(1{,}822)^2 + (5{,}243)^2} = 5{,}551 \text{ lb per in.}$$

The maximum spacing would then be $11{,}810/5{,}551 = 2.13$ in. for one row, or 4.26 in. for two rows. It will be seen that the effective-depth method gives conservative spacing as compared with the actual analysis in this particular case. The time required for computing the effective depth is practically the same as for computing the value of Q; therefore the approximate method has little in its favor.

The single cover plate at the end of the girder has a Q value of 275. The transfer of shear from cover plate to angles is then,

$$\frac{275 \times 306{,}050}{67{,}344} = 1{,}242 \text{ lb per in.}$$

The maximum spacing of rivets from cover plate to angles, as determined by the single shear value of the rivets, is $8{,}120/1{,}242 = 6.54$ in. Since there will be four lines of rivets through the cover plates, a spacing of $4 \times 6.54 = 26.16$ in. would provide sufficient rivet value. However, a maximum rivet spacing required to prevent separation of the elements is provided by the specifications, in which the staggered pitch shall not exceed $4 + 4t - (3g/4)$, when t is the thickness of the plate and g is the gage distance between rivet lines. The maximum spacing permitted by this specification, when $t = 0.5$ and $g = 3$ in., will be $4 + 2 - 2.25 = 3.75$, or 7.50 in. for each row.

At the theoretical end of the second pair of cover plates (11.6 ft from the end bearing), the shear is 215,800 lb; the value of I with two cover plates top and bottom $= 84{,}642$, and $Q = 1{,}294$. The horizontal shear between the web and the flange $= 215{,}800 \times 0.0153 = 3{,}302$ lb per in. Combining the vertical and horizontal loads,

$$w = \sqrt{(3{,}302)^2 + (1{,}822)^2} = 3{,}771 \text{ lb per in.}$$

The maximum spacing of rivets from flange to web at this point is then 11,810/3,771 = 3.13 in., or 6.26 in. on two rows. The shear between cover plates and angles is, then, (215,800 × 554)/84,642 = 1,412 lb per in. and the maximum rivet spacing is 8,120/1,412 = 5.75 in., or 23.0 in. on four rows. The spacing for cover plate attachment is again determined by the empirical value of 7.50 in. for each row.

At a point 11.0 ft from the center of the girder, the horizontal shear is 173,300 × 0.0154 = 2,669 lb per in.; the vertical load on the flange is 1,822 lb per in.; the resultant shear per inch is

$$w = \sqrt{(2{,}669)^2 + (1{,}822)^2} = 3{,}235 \text{ lb per in.}$$

and the computed rivet spacing for the rivets through angles and web is 7.30 in. on two lines. The shear between cover plates and flange angles is

$$\frac{173{,}300 \times 766}{97{,}986} = 1{,}355 \text{ lb per in.,}$$

the maximum spacing is 8,120/1,355 = 5.99 in., or 23.96 in. on each of four rows.

The rivet spacing may be summarized as follows: For the first 11.6 ft at the end of the girder, the spacing of flange rivets to web will be $4\frac{3}{4}$ in., maximum; from 11.6 to 16.8 ft the maximum spacing will be $6\frac{1}{4}$ in. on two lines; for the central 22.0 ft the maximum spacing will be $7\frac{1}{4}$ in. on two lines. The cover plates may be attached by four lines of rivets having spacing of 6.5 in. on each line where the outside plate is $\frac{3}{8}$ and 7.5 in. for the remainder of the length, and with the rivets of the inside rows staggered with those of the outside rows.

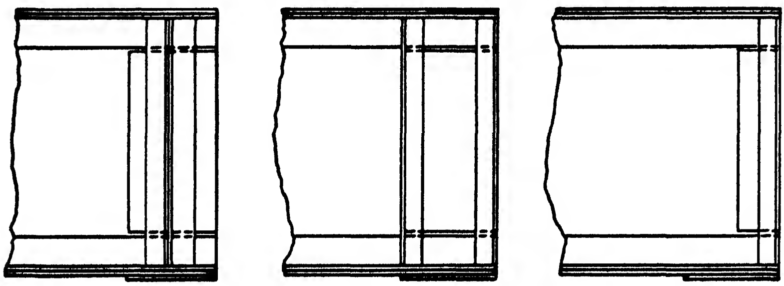
The original assumption for rivet holes to be deducted may be checked at this point. The spacing for rivets through the cover plates is no less than was assumed in computing the net section. The maximum number of rivets through the vertical planes occurs at the stiffeners.

6–11. Riveted Stiffeners. Two types of stiffeners will be required on most plate girder designs. Stiffeners over bearings or at points of application of concentrated loads are required to transfer the concentration from the flange to the stiffeners in bearing, hence through attachment to the web. Intermediate stiffeners serve primarily to prevent buckling of the web. For the railway plate girder under consideration, the end stiffeners are required to transfer a total end reaction of 306,050 lb from the web. Bearing stiffeners are not permitted to be crimped, and fillers equal in thickness to the flange angles must be used. The specification requires that the fillers be extended beyond the connected members (stiffeners) and must be attached with sufficient rivets through the extension to distribute the total stress over the combined section of fillers and member (web). The thickness of the fillers will be 1.75 in. and of the web 0.5 in.; hence, 78 per cent of the load must be transferred from the web to the fillers with rivets. The number of rivets required in the filler extension will then be

$$\frac{0.78 \times 306{,}050}{11{,}810} = 20.2,$$

or two rows of 11 rivets each. The transfer of load from the combined filler and web section to the stiffeners will be through double shear and will require 306,050/16,240 = 18.9 rivets, or two rows of 10 rivets each. It may be a little conservative, but it is considered good practice not to count the rivets through the flange and stiffeners as providing attachment of the stiffeners. As shown in Fig. 6–9, 4-in. spacing will provide 11 rivets in each line through the web, not counting those in the flanges.

The stiffeners are required to transfer the end reaction through bearing to the flange. The bearing required = 306,050/27,000 = 11.3 sq in.



(a) (b) (c)

FIG. 6–13.

It is practically impossible to obtain good bearing in that area where the contact would be on the fillets of the flange angles; hence, the specifications do not permit inclusion of this area. Assuming four angles 6 x 4 x $\frac{5}{8}$, with the 6-in. legs outstanding, the effective area in bearing will be $(4 \times 5.86) - (4 \times 4 \times 0.5) = 15.44$ sq in., allowing for a fillet with 0.5-in. radius. The $\frac{5}{8}$-in. angles meet the requirements of the specifications, including those for control of local buckling, but most designers would prefer to use $\frac{3}{4}$-in. angles for end stiffeners on a girder of this size because of the uncertainty of securing uniform bearing.

End stiffeners should be placed so as to concentrate the reaction at the center of the bearing. Where four angles are used for the end stiffener detail, they should be placed back-to-back as shown in Fig. 6–13(*a*). The means of centralizing the bearings on the masonry or other support will be considered in the design of the bearing (Chap. 8). Figure 6–13 shows sole plates under the end of the girders, but it does not show the detail of bearings on the masonry.

Two details showing poor design for end stiffeners are shown in Fig.

6–13(*b*) and (*c*). In (*b*) the stiffeners are separated and placed with the angles toed toward each other. When the girder deflects under load, it may be expected that the stiffener angles nearest the center of the span will receive a majority, if not all, of the load. This is particularly true if care is not taken to use a pinned support or other type of centralizing bearing under the girder. In the case of (*c*), the bearing is centralized at the very end of the girder. Many stiffeners detailed in this manner have been found to have greatly reduced bearing because of a slight misalignment of the stiffeners with the end of the girder. Details, as shown in (*b*) and (*c*), are usually subject to considerable corrosion at the contact of the stiffeners with the bottom flange.

Since the clear depth of the web is more than 60 times the web thickness, intermediate stiffeners will be required. The clear distance between the stiffeners is controlled by the requirements that it be not more than 72 in., or not more than $d = 10{,}500t/\sqrt{s}$, in which d is the clear distance between stiffeners, t is the web thickness, and s is the unit shearing stress on the gross section of web (average).

At the end of the girder,

$$s = \frac{306{,}050}{30} = 10{,}200 \text{ psi},$$

and

$$d = \frac{10{,}500 \times 0.5}{\sqrt{10{,}200}} = 52 \text{ in.}$$

The point at which a clear spacing of 72 in. will satisfy the requirements will be where

$$s = \left(\frac{10{,}500 \times 0.5}{72}\right)^2 = 5{,}317 \text{ psi},$$

or where

$$V = 30 \times 5{,}317 = 159{,}500 \text{ lb.}$$

Examination of the maximum shear diagram indicates that a clear spacing of 72 in. might be used at a point about 10 ft from the center of the girder. The requirements for spacing could be met by varying the clear distance from 51 in. to 72 in. and with the use of 10 pairs of intermediate stiffeners. However, the additional layout work necessary and the variations in the details of lateral connections do not justify the variable spacing. Twelve pairs of uniformly spaced stiffeners will be used.

The length of the outstanding legs of intermediate stiffeners is governed by the empirical rule that they be at least 2 in. plus one-thirtieth of the depth of the girder, which would in this case require a minimum of a 4-in. outstanding leg. The thickness must be at least one-sixteenth of the outstanding leg, but not less than ⅜ in. Stiffener angles may be 4 x 3½ x ⅜. Fillers under the intermediate stiffeners will be 3½ x ⅞. No additional rivets are required on account of these fillers because the rivets do not carry computed stress. The rivet spacing will be made the same as for the end stiffeners to simplify the details and fabrication.

6–12. Support of Compression Flanges. The compression in the flange of a girder is accumulated gradually from increments of load applied at each rivet or through welds in the case of welded flanges. The stability of the flange against lateral buckling is a problem involving the manner of loading, which varies with the shape of the shear diagram for the girder. A column loaded with axial loading at its ends has a limit of load expressed by Euler's formula. If the load is gradually applied along the length of the column, this limit of load is increased; the largest buckling limit occurs in the condition in which the largest percentage of the column load is applied near the midpoint of the span.*

Plate girder flanges are usually varying in section along their length. This condition also affects the limit of the buckling load.

The AREA specifications permit unit stresses in the compression flanges of girders $= 18{,}000 - 5L^2/b^2$, in which L is the length of unsupported flange between lateral connections or knee braces and b is the flange width, both in inches.

For the girder under consideration, the maximum compressive stress is

$$\frac{3{,}773{,}000 \times 12}{3{,}098} = 14{,}615 \text{ psi.}$$

To determine the maximum permissible laterally unsupported length of compression flange, let $14{,}615 = 18{,}000 - 5L^2/b^2$. Since $b = 18$ in., the distance L between lateral supports may be as large as 472 in. Cross frames and lateral bracing will be provided for wind and other lateral forces and will support the compression flange at closer intervals than the maximum. It would appear that the compression flange might be reduced in size. This would be true except that the specifications require that the compression flange be as large as the tension flange to provide for the "effects of corrosion and possible crookedness."

6–13. Riveted Web Splices. Web splices in plate girders should be avoided whenever possible. Except in very deep girders, plates for webs can be obtained in widths and lengths sufficient to make splicing unnecessary. As an example, the AISC handbook lists 96 x ¾ Pls obtainable in lengths up to 70 ft.

When it is necessary to splice the web of a girder, consideration should be given to the fact that the web plate is included in the computation of the moment of resistance of any section; hence, it is calculated to resist its share of the bending moment as well as shear. The AREA, AISC, and

* Clifford D. Williams, "The Limit of Stress in the Compression Flanges of Beams," Preliminary Publication, 1952, International Association for Bridge and Structural Engineering, pp. 255.

Bethlehem Steel Co.

West Side Highway, New York City.

AASHO specifications require that web splices be designed to transmit both shear and bending moment.

Probably the simplest approach to accomplishing a full web splice is to splice the web for its full value in tension. Allowable shear stresses are lower than allowable tension stresses. The maximum shear occurs near the neutral axis of the web, and the maximum bending stresses occur near the top and bottom edges of the plate. It would appear that the splice need not be quite so complete near the center of the girder as at its edges, but uniformity of rivet spacing is a prerequisite for economical fabrication.

Examination of Fig. 6–10 indicates that there are certain sections along the length of the girder where cover plates provide excess moment of

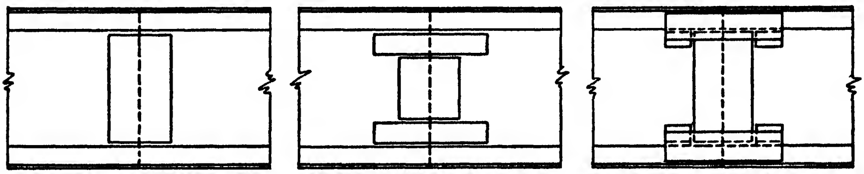


Fig. 6–14.

resistance; it is considered good practice to locate web splices at some such point. Even though the splice is computed to develop the full web, the streamline flow of stress in the region of the splice is interrupted with probable local stress increases not included in the calculations. For this reason, a little excess in the computed moment of resistance can do no harm.

Figure 6–14 shows three types of web splice frequently used. Type 1, known as a *shear splice,* is probably the oldest style of splice used. This splice is designed by making the splice plates of such thickness that the section modulus of the splice material will be equivalent to the section modulus of the web. The splice material is then attached to the web with sufficient rivets to transfer the computed moment and shear from the web to the splice plates.

Type 2, sometimes called the *moment splice,* is made of three plates on each side of the web plate. This splice has been designed as though the combined plates were acting as a single unit capable of taking both shear and that part of the bending moment normally taken by the web.

Objection is frequently made to both these splices. The maximum unit stress due to bending moment is at points farthest removed from the neutral axis. The parts of the web receiving the highest stress from bending are not directly spliced by either Type 1 or Type 2 splices. It is necessary for stress to travel a very circuitous route from the extreme fibers of the web to the splice material and then back. The probability of stress traveling the long route to the splice is rather remote. Undoubtedly the flange stress is increased in the vicinity of such splices, and they should be used only where considerable excess flange section exists. In the case of Type 2 splice, there is no connection between the elements of the splice; hence, there can be no transfer of horizontal shear except through the rivets used to transfer shear and moment from the web to the splice.

Type 3 splice has received considerable favor for many years because the material is placed more nearly where the maximum web stress occurs and because the elements are lapped to transfer shear.

The problem of tracing the transfer of shear and moment stresses to each rivet plane is rather tedious. However, it can be shown that a full depth splice of Type 3, attached to the web with sufficient rivets to develop the value of the plates in tension and with the required additional rivets for an indirect splice, will not have excess stress in the rivets. Type 3 splices will be used in the following illustration.

Using the girder designed in the preceding articles as an example, a web splice could be placed at about 12.33 ft from the centerline of the span. Some excess moment of resistance exists at this point and, since the flange rivet spacing was determined at a point about 16.15 ft from the centerline, there will be some excess value in the flange rivets at the splice point. The splice is required to transmit both shear and bending moment. Such a splice may be designed by providing splice material sufficient to replace the material in the web and rivet attachment adequate to develop the tensile value of the web. No further analysis is required, but the numerical example that follows illustrates a method of tracing the contribution of each rivet.

The web is 60 x ½ and, since ⅜-in. material is the minimum permitted under the specifications, the two required ⅜-in. splice plates will provide more section than is being spliced. The riveted attachment of the splice material will be the major consideration. The number of rivets required to develop the net section of the web in tension can be calculated for each inch of depth. Since the net web section between the flanges is determined by 4-in. vertical rivet spacing, there will be 0.75×0.5 sq in. of effective area per inch of height, and the number of rivets required to develop 18,000 psi tension per inch of height will be

$$\frac{0.75 \times 0.5 \times 18{,}000}{11{,}810} = 0.57 \text{ rivets.}$$

Two rows of rivets with 4-in. spacing will provide 0.5 rivets per inch, and three rows will provide 0.75 rivets per inch of depth. Two full rows at 4-in. spacing and a third row partly filled may be used.

Another approach is to determine the number of rivets which are required in the 44-in. distance between flanges to develop the tensile value of the net web.

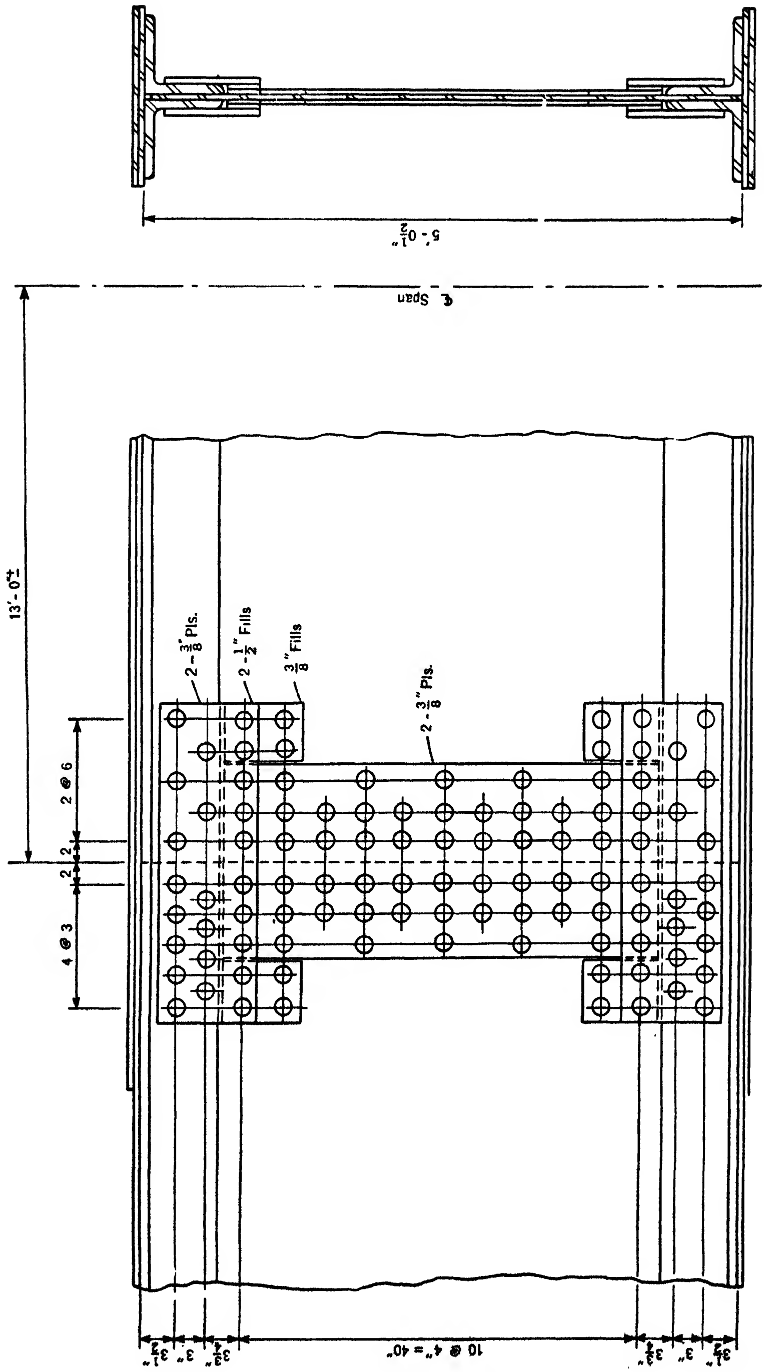

5'-0½"
℄ Span
13'-0±
2-⅜" Pls.
2-½" Fills
⅜" Fills
2-⅜" Pls.
2 @ 6
2
2
4 @ 3
3½"
3"
3¾"
10 @ 4" = 40"
3¾"
3"
3½"

FIG. 6-15.

Thus, $$\frac{(44 - 11) \times 0.5 \times 18{,}000}{11{,}810} = 26 \text{ rivets.}$$

Figure 6–15 shows the details of the web splice with 29 rivets attaching that part of the web splice between the flange angles. For the splice of that part of the web hidden by the flange angles, consideration should be given to the loads which would exist in those rivets if the web were not spliced.

In Art. 6–10 the spacing of flange rivets was determined to be $6\frac{1}{4}$ in. on two rows at 16.15 ft from the center of the span. At 12.33 ft from the center of the span the rivet requirement is somewhat less. At a splice, the normal rivet spacing must usually be varied to fit detail requirements. In Fig. 6–15 it will be noted that the normal spacing of $6\frac{1}{4}$ in. is changed to 6 in. This spacing simplifies the design and provides the desired 3-in. gage on vertical lines in the web splices. The net width of the 8 x $\frac{1}{2}$-in. strip of plate to be spliced will then be $(8 - 2) + (1.5)^2/(4 \times 3) = 6.19$ in. along the staggered line. The net section of the plate will require the development of section having a value of $6.19 \times 0.5 \times 18{,}000 = 55{,}710$ lb.

Since the rivets between the splice and the end of the girder receive local load in the same direction as the splice load, it will be necessary to add rivets to take care of the full 55,710 lb. Five additional rivets will be required. Between the splice and the center of the girder, the local load on the rivets is opposed to that from the splice shear load, and the splice plates will need to engage only five of the rivets provided for the flange.

As shown in Fig. 6–15, the plates splicing the edges of the web plate lap over the remainder of the splice to develop horizontal shear between the elements of the splice and to cause them to act as a single unit in resisting moment and shear.

The web splice shown in Fig. 6–15 provides for the development of the full net section of the web. For this reason it is not necessary to calculate the stress in individual rivets. However, in the following analysis, a better understanding of the action of the splice may be gained by a study of the detailed action of the material.

In the analysis of the splice as designed, it is necessary to determine the part of the bending moment and of the shear carried by the web plate and required to be transferred to the splice. The bending moment on the girder section at the splice is found to be 3,217,700 ft-lb. The unit stress at the extreme fiber of the web is, then,

$$\frac{3{,}217{,}700 \times 12 \times 30}{84{,}642} = 13{,}700.$$

The section modulus of the web is

$$\frac{0.5 \times 60 \times 60}{6} = 300 \text{ in.}^3.$$

The bending moment carried by the web is, then,

$$\frac{300 \times 13{,}700}{12} = 342{,}000 \text{ ft-lb},$$

or 10.6 per cent of the total bending moment at the section.

The shear on the girder at the point of the splice is found to be 183,000 lb. (See Fig. 6–12.) The shear on the web at any distance from the neutral axis is equal to VQ/I, where Q is the statical moment of the areas of the section farther removed from the neutral axis than the point for which the shear is desired. From Art. 6–10, the value of Q, computed for the flange with two cover plates, is 1,294 in.^{3}. The value of Q for the top 6.25 in. of web is $6.26 \times 0.5 \times 26.87 = 84$. The shear per inch of web at the inside row of flange rivets will then be

$$\frac{(1{,}294 + 84) \times 183{,}000}{84{,}642} = 2{,}980 \text{ lb per in.}$$

For the center of the web plate,

$$Q = 1{,}294 + (0.5 \times 30 \times 15) = 1{,}519 \text{ in.}^3,$$

and the shear is,

$$\frac{1{,}519 \times 183{,}000}{84{,}642} = 3{,}284 \text{ lb per in.}$$

The shear distribution on the web is shown in Fig. 6–16. The total shear taken by the web will be equal to the area of the shear diagram and is $(50.5 \times 2{,}980) + (47.5 \times 304 \times 2/3) = 160{,}130$ lb, or 87.6 per cent of the total shear at this section.

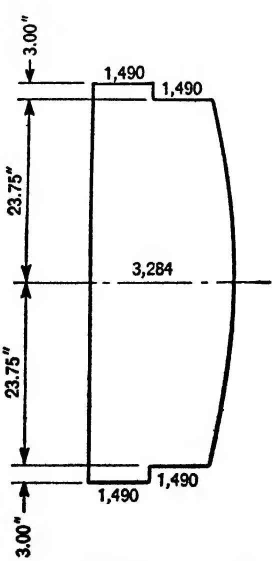


Fig. 6–16. Distribution of shear stress in the web.

Figure 6–15 shows the splice as designed, with 47 rivets to the left and 39 to the right or toward the center of the girder. Eight rivets used to attach fillers are not included, as they would be required by the specifications to reduce the bending in rivets caused by fillers. The properties of the group of 47 rivets to the left are found in the following computation:

No. of Rivets	x	Rx	Rx^2
13	2.0	26	52.0
2	3.5	7	24.5
13	5.0	65	325.0
2	6.5	13	84.5
9	8.0	72	576.0
2	9.5	17	161.5
2	11.0	22	242.0
2	12.5	25	312.5
2	14.0	28	392.0
47		275	2,170.0

No. of Rivets	y	y^2	Ry^2
3	0	0	0
4	4	16	64
6	8	64	384
4	12	144	576
6	16	256	1,536
6	20	400	2,400
8	23.75	564.1	4,513
10	26.75	715.6	7,156
47		I_x =	16,629

$$\bar{x} = \frac{275}{47} = 5.85 \text{ in.}$$

$$-5.85 \times 275 = -1{,}608.8$$

$$I_y = 561.2$$

From the above computations it will be seen that the center of the rivet group is at the centerline of the girder and 5.85 in. to the left of the splice center. The forces for which this group of rivets must be investigated are the 160,130-lb shear taken by the web and 10.6 per cent of the moment at 5.85 in. from the splice. The bending moment at 12.82 ft from the center of the girder is 3,217,700 − (0.49 × 183,000) = 3,128,000 ft-lb; the part of this moment taken by the web is 0.106 × 3,128,000 = 313,600 ft-lb. Since the maximum bending moment and maximum shear are not simultaneous a design using these values would be conservative.

The rivets farthest from the splice in the top and bottom rows will be the rivets receiving the largest loads from the transfer of bending moment. The polar moment of inertia of this group is

$$561 + 16{,}629 = 17{,}190.$$

That part of the bending moment taken by the web, 313,600 ft-lb, will then produce a load in the extreme rivet with a horizontal component equal to

$$\frac{313{,}600 \times 12 \times 26.75}{17{,}190} = 5{,}860 \text{ lb};$$

and the vertical component is

$$\frac{313{,}600 \times 12 \times 8.15}{17{,}190} = 1{,}790 \text{ lb}.$$

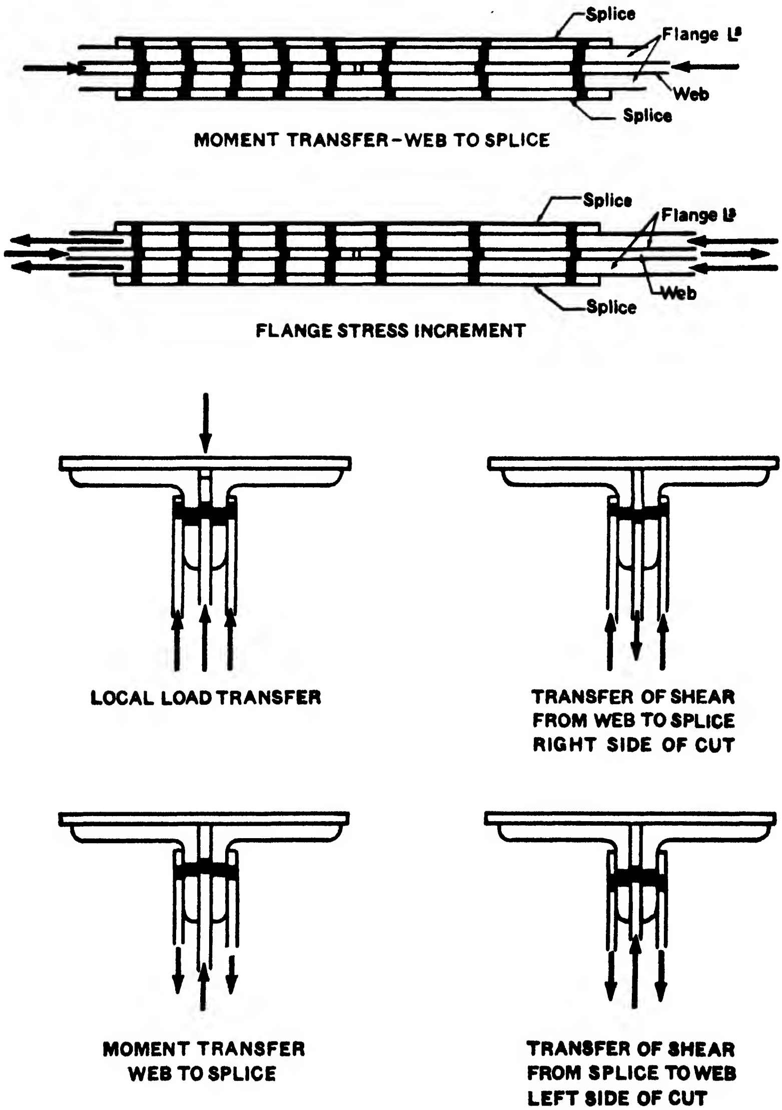


Fig. 6–17.

Figure 6–17 shows the nature of the shear loads coming to the rivets of the flange.

The shear load of 160,130 lb must be transferred through 47 rivets to the splice; hence, each rivet will receive 160,130/47 = 3,410 lb. The local vertical load was found in Art. 6–10 to be 1,822 lb per lin in. of flange, or $1.5 \times 1{,}822 = 2{,}733$ lb for each rivet in the top flange. This force is applied through the vertical legs of the flange angles, which are between the web and the splice plates; hence, one-half of the force is assumed to go into the splice plates and one-half into the web.

The transfer of load from the web to the flange is

$$\frac{VQ}{I} = \frac{183{,}000 \times 1{,}294}{84{,}642} = 2{,}796 \text{ lb per in.}$$

The load per rivet from this force is then $1.5 \times 2{,}796 = 4{,}194$ lb.

The forces acting on the end rivet of the top row may be summarized as follows:

	Horizontal Component	*Vertical Component*
Moment transfer to splice	5,860	1,790
Shear transfer	—	3,410
Increment of flange load	4,194	—
Local load	—	1,366
Total	10,054	6,566

The resultant load on the end rivet of this group is, then,

$$\sqrt{(10{,}054)^2 + (6{,}566)^2} = 12{,}008 \text{ lb.}$$

This load is practically equivalent to the 11,810-lb bearing value of a ⅞-in. rivet on the ½-in. web.

Examination of the calculations shows that all items in the summary will be the same for each rivet of the top row except the component due to moment transfer. All other rivets on this row will have a smaller total load than the rivet that was computed.

Rivets in the bottom flange do not receive the force from the local load of the track but otherwise receive similar forces to those in the top flange; the loads from transfer of moment and increment of flange load being in opposite direction to similar forces in the top flange.

For the side of the splice toward the center of the span the moments of inertia of the group of 39 rivets are found in the following calculation:

No. of Rivets	x	Rx	Rx^2
13	2.0	26	52
13	5.0	65	325
9	8.0	72	576
2	11.0	22	242
2	14.0	28	392
39		213	1,587

$$\bar{x} = \frac{213}{39} = 5.46 \text{ in.}$$

$$-(5.46 \times 213) = -1{,}163$$

$$I_y = 424$$

No. of Rivets	y	y^2	Ry^2
3	0	0	0
4	4	16	64
6	8	64	384
4	12	144	576
6	16	256	1,536
6	20	400	2,400
4	23.75	564.1	2,256
6	26.75	715.6	4,294
39		I_x =	11,510

The polar moment of inertia is 11,510 + 424 = 11,934.

The bending moment at the center of the right-hand group of rivets is 3,217,700 + (0.46 × 183,000) = 3,188,400 ft-lb; the part of this moment taken by the web is 0.106 × 3,188,400 = 338,000 ft-lb, and the shear in the web is 160,130 lb.

The vertical component of the load on the end rivet of the top row, due to moment, is

$$\frac{338{,}000 \times 12 \times 8.54}{11{,}934} = 2{,}900 \text{ lb};$$

and the horizontal component is,

$$\frac{338{,}000 \times 12 \times 26.75}{11{,}934} = 9{,}090 \text{ lb}.$$

The local load of 1,822 lb per in. of flange will cause a load of 3 × 1,822 = 5,466 lb per rivet, of which one-half will go directly to the web.

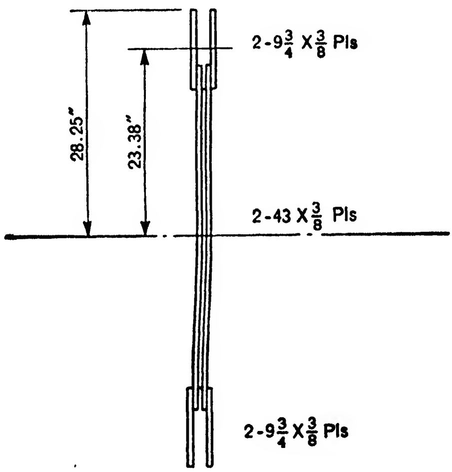


Fig. 6–18.

The increment of load transferred through rivets is 3 × 2,796 = 8,388 lb per rivet, and the shear load of 160,130 lb carried by the web causes a vertical load of 160,130/39 = 4,106 lb per rivet.

A summary of the loads coming to the end rivet in the top row to the right of the splice is as follows:

	Horizontal Component	*Vertical Component*
Moment transfer from splice	−9,090	2,900
Shear transfer	—	−4,106
Local load from track	—	2,733
Increment of flange load	8,388	—
Total	−702	1,527

The resultant load per rivet is then $\sqrt{(1{,}527)^2 + (702)^2} = 1{,}681$ lb, which is less than the allowable rivet load.

In order to determine whether the elements of the web splice are adequately attached to each other, the shear on the attaching rivets may be computed. The total shear taken by the web splice material was found to be 160,130 lb. The cross section of the splice material is composed of two 43 x ⅜ intermediate plates and four 9¾ x ⅜ plates, as shown in Fig. 6–18. The moment of inertia of the combined section is

$$\begin{aligned}
4 \times \frac{9.75^3}{12} \times \frac{3}{8} \qquad &= \quad 116 \\
4 \times 9.75 \times \frac{3}{8} \times 23.38^2 &= \quad 7{,}994 \\
2 \times \frac{43^3}{12} \times \frac{3}{8} \qquad &= \quad \underline{4{,}969} \\
I &= 13{,}079 \text{ in.}^4
\end{aligned}$$

For the two smaller plates above the plane of the attaching rivets,

$$Q = 2 \times 9.75 \times \frac{3}{8} \times 23.38 = 171 \text{ in.}^3$$

The shear between the elements of the splice is

$$\frac{VQ}{I} = \frac{160{,}130 \times 171}{13{,}079} = 2{,}100 \text{ lb per in. of connection.}$$

This force is resisted by six rivets in a length of 19 in., which is

$$\frac{2{,}100 \times 19}{6} = 6{,}650 \text{ lb per rivet.}$$

Since the double-shear value of a ⅞-in. rivet is 16,240 lb, the attachment of the elements of the splice to each other is ample to ensure that they act together to resist moment and shear. It also provides an excess value to compensate for the ⅝-in. filler at this line of contact. It should be noted that the shear computed is on a different plane than that transferring stress from the web to the splice and hence it may be considered separately.

6–14. Flange Splices. Angles are available in such lengths that splicing of flange angles seldom is necessary except in very long girders. Splices of flange angles should be located where cover plates provide an excess of moment of resistance. Although there is some argument for keeping the flange symmetrical about the vertical axis, it is usual to splice only one flange angle at any one point. This method reduces the amount of material cut and decreases the rivet grip necessary at the splice.

For the girder designed in previous articles, it would normally be possible to obtain full length angles. Assuming, however, that it is necessary to splice the flange angles, Fig. 6–19 shows a splice detail which could be used.

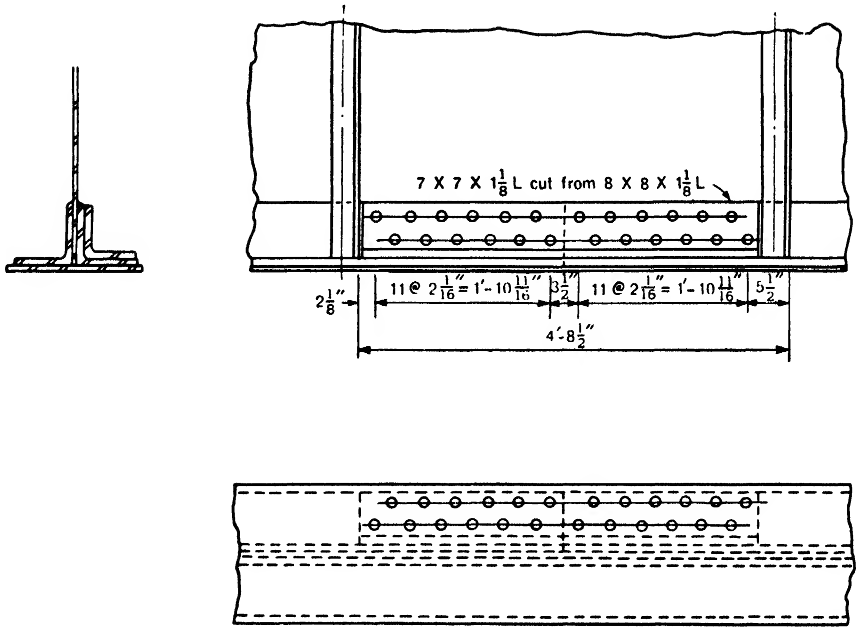


FIG. 6-19. Flange splice.

It is better not to have the outstanding leg of the splice angle extend beyond the flange angle; hence, a special angle providing sufficient area is usually fabricated. As shown in Fig. 6-19, a special 7 x 7 x 1⅛ angle is made by cutting 1 in. from the leg of an 8 x 8 x 1⅛ angle. The corner of the angle must be ground to clear the fillet of the flange angle, and the area of the special angle will be slightly larger than the area of the angle being spliced.

In Art. 6-8 two holes were deducted from each of the vertical legs of the flange angles and 2.24 holes from the two horizontal legs, making a total deduction of 6.24 holes from the two angles. The net section of the flange angles will not be further reduced if no more than 3.12 holes are deducted from each flange angle at the splice. Referring to Section 44 of the AREA specifications, the gross width of the angle will be 8 + 8 − 0.88 = 15.12 in. and the net width used in computing the cross section, 15.12 − 3.12 = 12.00 in. The rivets in the vertical leg of the angle are staggered as shown in Fig. 6-19. For this section the minimum stagger s will be such that

$$15.12 - 4.00 + \frac{s^2}{12} + \frac{s^2}{24.5} + \frac{s^2}{12} = 12.00,$$

or $s = 2.06$ in.

The number of rivets required to splice the net area of one flange angle may be computed.

$$\begin{array}{lcl} \text{Gross area of one flange angle} & = & 13.23 \text{ sq in.} \\ \text{Rivet holes, } 3.12 \times \frac{7}{8} & = & \underline{\ 2.73\ } \\ \text{Net area} & = & 10.50 \text{ sq in.} \end{array}$$

$$\text{Number of rivets required} = \frac{10.5 \times 18{,}000}{8{,}120} = 24$$

Figure 6–19 shows the minimum length of splice that might be used. This detail requires that the regular spacing of the stiffeners be increased (Art. 6–11) at the splice points. In cases where the stiffener spacing could not be altered sufficiently, it may be necessary to place the splice angle under the stiffener.

The transfer of local load and flange stress increment occurs on one plane through the rivet, while the splice load is transferred on a different plane; hence, the loads do not accumulate and the splice requirements may be met without addition of rivets for the transfer of flange stress to the web.

Since the center of gravity of the splice angle is somewhat closer to the center of the girder than that for the angle being spliced, there will be some reduction in the total moment of resistance. The excess value in the cover plate should be sufficient to offset the loss of section at the splice point due to the shift of material.

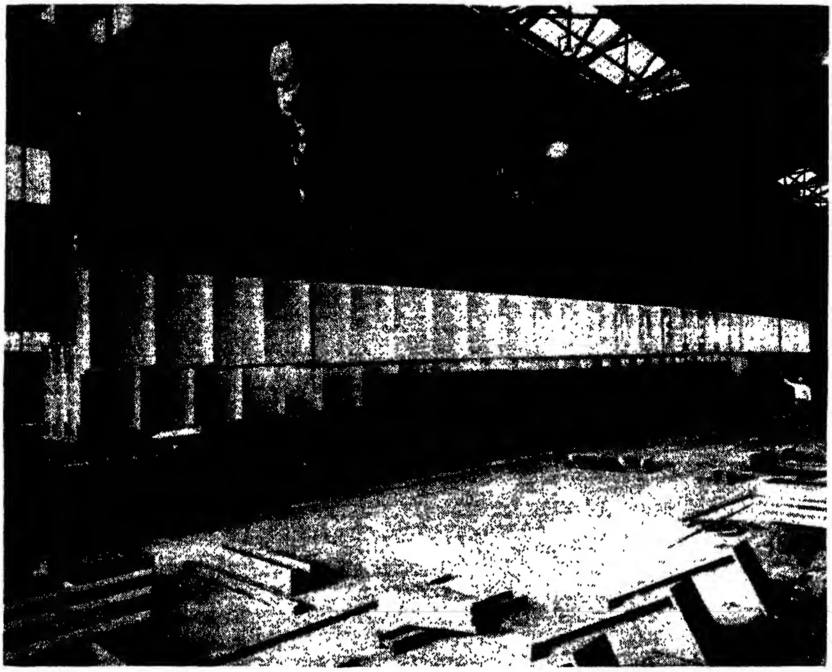
The splice illustrated in Fig. 6–19 is a bottom flange angle splice, but the top and bottom flange spices would usually be designed in the same manner.

6–15. Welded Flange and Web Sections. The choice of elements for the assembly of a welded plate girder is somewhat more flexible than for a riveted girder. Splices are easier to accomplish, and plates may be varied in thickness to suit the design requirements. Flanges of welded girders are usually assembled from flat plates or split I-beams.

Figure 6–4(a) shows a section of a girder with parts of a wide-flange beam used for the top and bottom flanges and with a plate inserted to complete the web. The plate is welded longitudinally to the partial webs of the beam. Figure 6–4(b) shows a girder with a web plate full height and flat plates for the flanges. Figure 6–4(c) shows a girder section in which parts of beams are used and cover plates added. The sections used in the flanges may be varied throughout the length of the girder by the use of transverse welded joints. Designs have been made in which the web also varied in thickness, being the thickest toward the end of the girder.

Considerable uncertainty about the fatigue properties of welded members has delayed the use of welded girders for railway bridges, but many such girders are used in buildings, industrial equipment, and highway bridges.

In the following discussion, a welded plate girder (Fig. 6–24) is designed for a loading condition such as might occur in building construction, using the AISC specification. It is assumed that a span of 66 ft center-to-center



Bethlehem Steel Co

WELDED PLATE GIRDERS FOR NEW YORK STATE THRUWAY
(58 IN. DEEP AND 100 FT. LONG).

of bearings is required. The load consists of a concrete floor of slab and beam construction, which brings concentrated floor loads at 6-ft centers to the top of the girder. The dead load at each concentration is 12,000 lb, and a live load of 18,000 lb may be at any or all beam points.

The maximum moment occurs at the center of the span when all points are fully loaded with live load. Assuming that the weight of the girder is 250 lb per lin ft, the maximum moment will be

$$\begin{aligned} 5 \times 30{,}000 \times 33 &= +4{,}950{,}000 \text{ ft-lb} \\ -5 \times 30{,}000 \times 15 &= -2{,}250{,}000 \\ \frac{250 \times 66 \times 66}{8} &= \underline{+\ \ 136{,}125} \\ & +2{,}836{,}125 \text{ ft-lb} \end{aligned}$$

Since the top flange will be supported at intervals of 6 ft, it is likely that the allowable compression will be equal to the allowable tension, or 20,000 psi. The depth of the girder should be between $L/15$ and $L/12$ for economical use of material, or approximately 5 ft out-to-out of flanges. Assuming that the flanges will be about 2 in. thick, the depth of the web will be approximately 56 in.

The maximum shear will be equal to the end reaction, or (5 × 30,000)

+ (250 × 33) = 158,250 lb. The web area needed will be 158,250/13,000 = 12.17 sq in. A web plate 56 x ¼ will provide 14.0 sq in., and ¼-in. web thickness is the minimum permitted under the specifications. However, the AISC specification for plate girders also requires that "plate girder webs shall have a thickness of not less than 1/170 of the unsupported distance between flanges." This requirement makes it necessary to use a web 56/170 = 0.33 in. thick, or a 56 x ⅜ web plate must be used.

With a depth of 58 in. center-to-center of flanges, a web of 56 x ⅜, and flange areas of A top and bottom, the moment of inertia of the girder will be $(2A \times 29 \times 29) + 5{,}488 = 1{,}682A + 5{,}488$. The extreme fiber stress will be

$$\frac{Mc}{I} = 20{,}000 \text{ psi}, \quad \text{or} \quad \frac{2{,}836{,}125 \times 12 \times 30}{1{,}682A + 5{,}488} = 20{,}000,$$

from which $A = 27.09$ sq in.

Figure 6–20 shows a cross section having 27.00 sq in. in each flange. The flanges are made from four thicknesses of plates butt-welded at their ends. The moments of resistance for the sections with the several thicknesses of flange plates are found as follows:

	A	y	I	ΣI	c	S	M (ft-lb)
56 x ⅜ web.......	21.00	0	5,490	5,490			
2—13.5 x ½ Pls...	2 × 6.75	28.25	10,774	16,264	28.50	570	950,000
2—13.5 x ½ Pls...	2 × 6.75	28.75	11,159	27,423	29.00	946	1,576,700
2—13.5 x ½ Pls...	2 × 6.75	29.25	11,550	38,973	29.50	1,321	2,201,700
2—13.5 x ½ Pls...	2 × 6.75	29.75	11,948	50,921	30.00	1,697	2,828,000

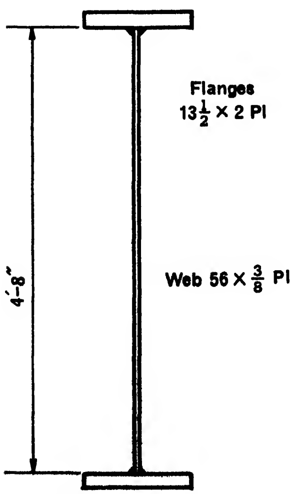


FIG. 6–20. Section of girder.

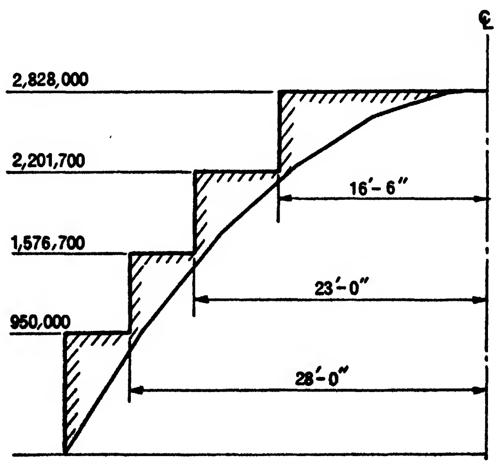


FIG. 6–21. Moment of resistance diagram.

The unit stress at the center of the span is

$$\frac{2{,}836{,}125 \times 12 \times 30.00}{50{,}921} = 20{,}060 \text{ psi.}$$

Figure 6–21 shows the graphical determination of the lengths of flange plates, in which the moments of resistance for each section are plotted on the moment diagram.

6–16. Welded Connections. The flanges will need to be attached to the web with sufficient weld to develop the horizontal shear.

The maximum live-load shear in each panel is approximated by assuming full live loads to occur at all panel points on one side only of the panel

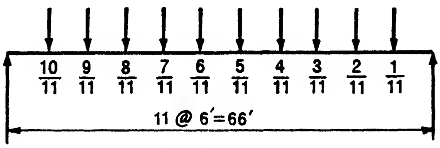


FIG. 6–22. Fraction of each load taken to left hand side of panel.

in question. Referring to Fig. 6–22, the live-load shear may be computed as follows:

Between beam No. 5 and the center: $\left(\frac{1}{11} + \frac{2}{11} + \frac{3}{11} + \frac{4}{11} + \frac{5}{11}\right) 18{,}000 = 24{,}600$ lb

Between beams No. 4 and No. 5:	$24{,}600 + 6(18{,}000)/11 = 34{,}400$
Between beams No. 3 and No. 4:	$34{,}400 + 7(18{,}000)/11 = 45{,}800$
Between beams No. 2 and No. 3:	$45{,}800 + 8(18{,}000)/11 = 58{,}900$
Between beams No. 1 and No. 2:	$58{,}900 + 9(18{,}000)/11 = 73{,}600$
Between the end and beam No. 1:	$73{,}600 + 10(18{,}000)/11 = 90{,}000$

The dead-load shear is computed with the concentrated load of 12,000 lb at all points and a uniform load of 250 lb per ft of girder. The totals of dead- and live-load shear are plotted in Fig. 6–23.

The shear between the support and the first load is 158,200 lb, and Q for the minimum section is $6.75 \times 28.25 = 191$ in.^{3} The shear load per inch of flange is

$$\frac{158{,}200 \times 191}{16{,}264} = 1{,}858 \text{ lb.}$$

The size of welds to be used in connecting the elements of the plate girder is determined by consideration of the thickness of the elements to be connected. Not less than ⅜-in. welds should be used when connecting a 2-in. plate (AISC specification, Section 24c). The section near the center of the span will require ⅜-in. welds for attachment of the flange plates to the web. Even though 5⁄16-in. welds might be used near the end of the girder, the design would be simplified by using the same size of weld for

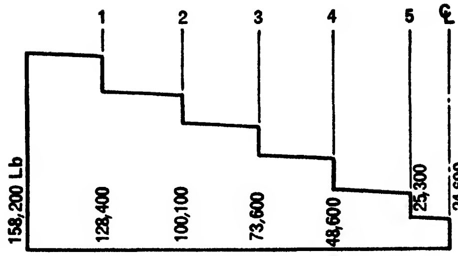


FIG. 6-23. Diagram of maximum shears.

the entire flange length. The value of $\frac{3}{8}$-in. welds placed on each side of the $\frac{3}{8}$-in. web plate is limited to $1.4 \times 3{,}600 = 5{,}040$ lb per lin in. of connection. (See Art. 5-11.) The spacing required to develop the flange plate connection in the end panel with 3 x $\frac{3}{8}$ welds on each side is then

$$\frac{3 \times 5{,}040}{1{,}858} = 8.1 \text{ in.}$$

Between beams No. 1 and No. 2, the maximum shear $V = 128{,}400$ lb, the value of $Q = 13.5 \times 28.5 = 385$ in.³, and $I = 27{,}423$ in.⁴ The shear per linear inch of flange is

$$\frac{128{,}400 \times 385}{27{,}423} = 1{,}800 \text{ lb},$$

and the spacing of two 3 x $\frac{3}{8}$ welds $= 15{,}120/1{,}800 = 8.4$ in. center-to-center.

Between beams No. 2 and No. 3, $V = 100{,}100$ lb; $Q = 20.25 \times 28.75 = 582$ in.³; $I = 38{,}973$ in.⁴; the shear per inch is

$$\frac{100{,}100 \times 582}{38{,}973} = 1{,}495 \text{ lb};$$

and the spacing of the two 3 x $\frac{3}{8}$ welds $= 15{,}120/1{,}495 = 10.1$ in.

Between beams No. 3 and No. 4, $V = 73{,}600$ lb; $Q = 27.0 \times 29.0 = 783$ in.³; $I = 50{,}921$ in.⁴; the shear per inch is

$$\frac{73{,}600 \times 783}{50{,}921} = 1{,}132 \text{ lb};$$

and the maximum spacing for two 3 x $\frac{3}{8}$ welds $= 15{,}120/1{,}132 = 13.4$ in.

The AISC specification requires that the space between intermittent fillet welds attaching elements in compression shall not exceed 16 times the thickness of the thinnest element. Hence, the maximum spacing for attaching the flange plates will be $(16 \times \frac{3}{8}) + 3 = 9$ in. center-to-center, and 8-in. spacing will be used in the 12 ft nearest to the end.

6-17. Stiffeners. Stiffeners for the girder of Fig. 6-24 will be of two kinds. At points of load concentrations and at end bearings the loads

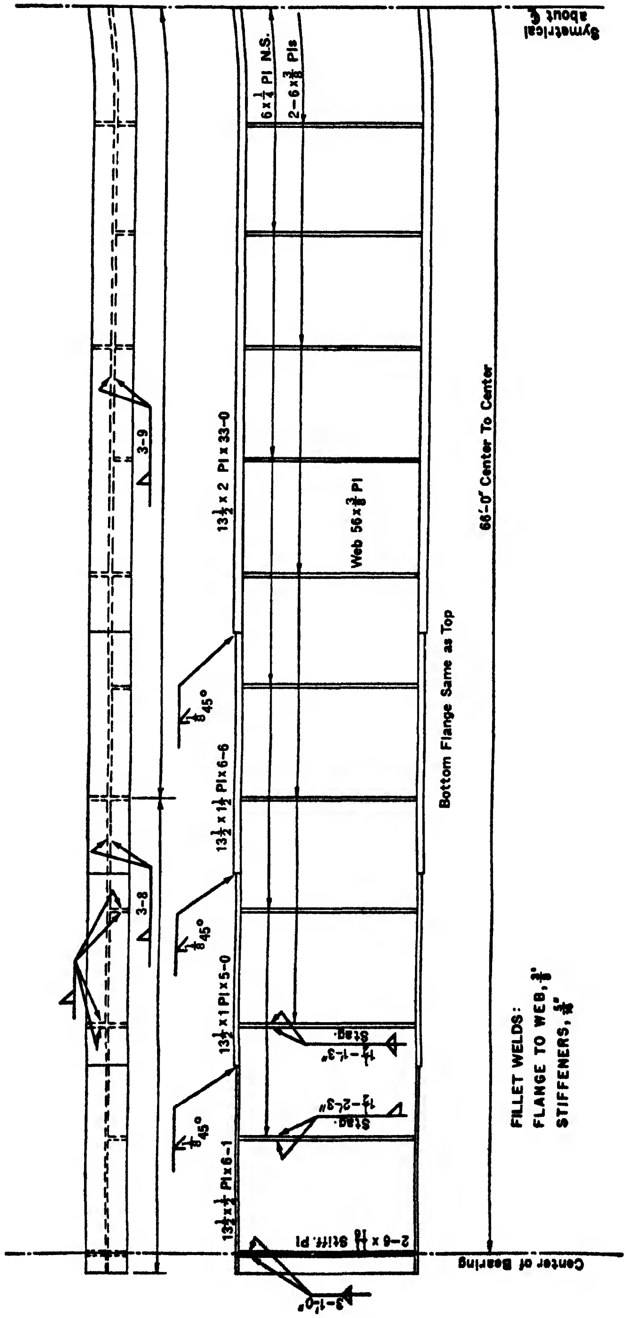

Symmetrical about ℄
6 x 1/4 Pl N.S.
2-6 x 3/8 Pls
13 1/2 x 2 Pl x 33-0
Web 56 x 3/8 Pl
66'-0" Center To Center
Bottom Flange Same as Top
3-9
3-8
13 1/2 x 1 1/2 Pl x 6-6
13 1/2 x 1 Pl x 5-0
13 1/2 x 1/2 Pl x 6-1
45°
Stag.
2-6 x 11/16 Stiff. Pl
Center of Bearing
FILLET WELDS:
FLANGE TO WEB, 3/8"
STIFFENERS, 5/16"

FIG. 6-24.

must be transferred to the web. These stiffeners will be spaced to agree with the loads, or at 6-ft centers.

When the clear depth between flanges is more than 70 times the thickness of the web, stiffeners will be used at points where v exceeds $64{,}000{,}000/(h/t)^2$,

in which h = the clear depth between flanges, in inches,

t = the thickness of the web, in inches,

v = the greatest shear stress in the panel, in psi.

At the end of the girder of Fig. 6–26,

$$Q = (10.5 \times 14) + (6.75 \times 28.25) = 338 \text{ in.}^3,$$

$$I = 16{,}264 \text{ in.}^4,$$

and

$$v = \frac{158{,}200 \times 338}{0.375 \times 16{,}264} = 8{,}770 \text{ psi}.$$

The value of

$$\frac{h}{t} = \frac{56}{0.375} = 149.3,$$

and

$$\frac{64{,}000{,}000}{(149.3)^2} = 2{,}870 \text{ psi}.$$

Hence, intermediate stiffeners are required (AISC Section 26e).

The clear distance between intermediate stiffeners should not be more than 7 ft, and not more than

$$\frac{11{,}000t}{\sqrt{v}} = \frac{11{,}000 \times 0.375}{\sqrt{8{,}770}} = 44.0 \text{ in.}$$

Intermediate stiffeners will need to be placed midway between the loaded points, which will provide for stiffener spacing of 3 ft throughout. Single plates may be used for the intermediate stiffeners, but the stiffeners at the load points should be on both sides of the web. All material should be at least $\frac{1}{4}$ in. thick. The bearing stiffeners should be wide enough to extend nearly to the edges of the cover plates, and their thickness should be not less than one-sixteenth of their widths.

The use of 6 x $\frac{1}{4}$ Pls for intermediate stiffeners may be checked against AISC Section 26e. The required $I_s = 0.00000016H^4$, in which H is the depth of the web and I_s is the moment of inertia of the 6 x $\frac{1}{4}$ Pl about its attached edge. Then, the required $I_s = 0.00000016(56)^4 = 1.57$ in.^{4} A 6 x $\frac{1}{4}$ Pl will have a value of $I_s = (6^3 \times 0.25)/3 = 18$ in.^{4} and will obviously satisfy this requirement. These stiffeners must be attached by welds with clear spacing of not more than 12 in. The attaching welds may be staggered on the two sides of the stiffeners with a spacing of 2 ft 3 in. on each side.

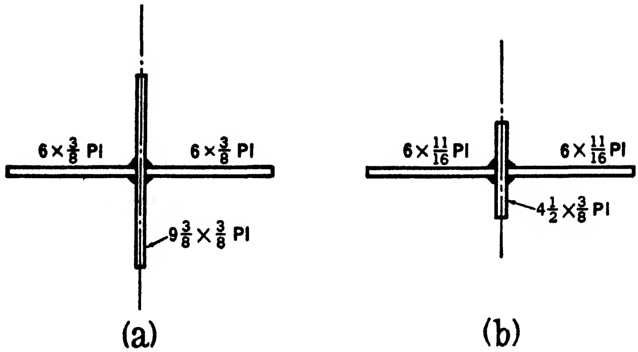


FIG. 6–25. Assumed column sections for bearing stiffeners.

Stiffeners at the load points must transfer 30,000 lb to the web and are to be considered as columns having a length of 3/4 of 56 = 42 in. The value of r is computed for a section including a web strip equal to $25t$ [see Fig. 6–25(a)]. Then, assuming two 6 x 3/8 Pls,

	A	y	Ay^2	I_0
2—6 x 3/8	4.50	3.19	45.79	13.50
1—9 3/8 x 3/8	3.52			
	8.02 in.²		45.79	13.50
			13.50	
			I = 59.29 in.⁴	

Thus,

$$r = \sqrt{\frac{59.29}{8.02}} = 2.72 \text{ in.}$$

The L/r is then 42/2.72 = 15.4, and the allowable compression is $17{,}000 - 0.485(15.4)^2 = 16{,}885$ psi. The computed stress in the assumed column section is 30,000/8.02 = 3,740 psi, which is less than the allowable. The compression on these stiffeners without consideration of a web strip will be 30,000/4.22 = 7,110 psi, which is less than the allowable 27,000-psi bearing stress. (Only that portion of the stiffener outside of the flange-to-web welds is considered effective in bearing, or 2 × 5.625 × 3/8 = 4.22 sq in.) The bearing stiffeners must be attached by welds with a clear spacing of not more than $16t$, or 6 in. If the welds are spaced 1 ft 3 in. on each side of the stiffeners and staggered, there will be 15 in. of weld with a value of 45,000 lb.

The end stiffeners are required to carry a reaction of 158,200 lb. The required stiffener will be assumed to be two 6 x 11/16 Pls. The web strip that is assumed to be included when computing the stiffener as a column will be 3/8 × 12 = 4.50 in. wide. [See Fig. 6–25(b).]

Aluminum Company of America

Aluminum Alloy Plate Girder Bridge Span over Grasse River, Massena, N. Y. (100-Ft. Span. Center-to-Center of Bearings.)

	A	y	Ay^2	I_0
2—6 x 1$\frac{1}{16}$	8.25	3.19	83.95	24.75
1—4$\frac{1}{2}$ x $\frac{3}{8}$	1.69			
	9.94 in.^{2}		83.95	24.75
			24.75	
			I = 108.70 in.^{4}	

Thus,

$$r = \sqrt{\frac{108.70}{9.94}} = 3.30 \text{ in.},$$

$$L/r = 42/3.30 = 12.7,$$

$$17{,}000 - 0.485(12.7)^2 = 16{,}920 \text{ psi},$$

$$158{,}200/9.94 = 15{,}910 \text{ psi compression},$$

and

$$158{,}200/7.73 = 20{,}470 \text{ psi bearing}.$$

The effective size of the $\frac{5}{16}$-in. welds connecting the end stiffeners to the web [arranged as shown in Fig. 6–25(*b*)] is only 0.7 × $\frac{3}{8}$, or 0.262 in. The strength per inch of $\frac{5}{16}$-in. weld is then only 0.707 × 0.262 × 13,600 = 2,520 lb. The weld required to attach the end stiffeners will be 158,200/2,520 = 62.8 in. Three-inch welds spaced 10$\frac{1}{2}$ in. center-to-center will satisfy this requirement.

6–18. Aluminum Plate Girder. During 1946, the Massena Terminal Railroad constructed a railroad bridge over the Grass River near Massena, N. Y.* (See accompanying illustration.) One of the 100-ft deck plate girder spans was constructed of 14S-T aluminum alloy with 17S-T rivets. The remaining spans were constructed of steel.

The aluminum span was designed according to the *Design Specifications for Bridges and Structures of Aluminum Alloy 27S-T*, by L. S. Moisseiff, and the steel spans according to the AREA specifications. The loading was Cooper's E-60.

The maximum section of the aluminum girders consisted of a web plate 120 x $\frac{3}{4}$, four 8 x 6 x $\frac{5}{8}$ angles, and four 14 x $\frac{5}{8}$ cover plates. The steel girder for the same span had a 108 x $\frac{1}{2}$ web plate, four 8 x 8 x $\frac{3}{4}$ angles, four 18 x $\frac{5}{8}$ cover plates, and two 18 x $\frac{9}{16}$ cover plates. The section modulus of the aluminum girder was 4,542 in.^{3} and that of the steel girder 5,642 in.^{3} The weight of the two aluminum girders with bracing was 53,000 lb compared with 128,000 lb for the steel span.

PROBLEMS

6–1. A girder is to be designed for a maximum bending moment of 2,836,000 ft-lb and an end reaction of 158,200 lb. The maximum depth available is 5 ft out-to-out.

* "Aluminum Span for E-60 Railroad Bridge," *Engineering News-Record*, November 28, 1946.

Select a section for riveted construction, using the AISC specification. Assume that no deduction of rivet holes is required. Compare the cross-sectional area with that for the welded girder of Art. 6–15.

6–2. Determine cross sections for riveted plate girders having depths of 32 in., 48 in. and 64 in. back-to-back of flange angles, when the maximum bending moment is 3,000,000 ft-lb and the maximum shear 300,000 lb. Use the AREA specification and compare the resulting cross-sectional areas.

6–3. A plate girder section is composed of a 48 x ½ web, 6 x 6 x ¾ flange angles, and one 14 x ½ flange plate top and bottom; it has a dimension of 4 ft ½ in. back-to-back of flange angles. If the shear load is 175,000 lb, determine the required rivet spacing to attach the flange angles to the web and the cover plate to the angles.

6–4. Develop a maximum bending moment diagram and a maximum shear diagram for a deck-plate girder highway span. The length of the span is 90 ft center-to-center of bearings; each girder supports one lane of highway; the dead load above each girder may be assumed to be 1,000 lb per lin ft and the weight of a girder 250 lb per lin ft; the live load will be H20-S16 according to the AASHO *Specifications for Highway Bridges.*

6–5. Design a cross section for the girder of Prob. 6–4, assuming that the head room will permit a depth of 6 ft for the girder and that riveted construction will be used.

6–6. Design an alternate cross section for the girder of Prob. 6–4, using welded construction and an over-all depth of 6 ft. Use flanges of wide flange beams for the girder flanges and change sections where it will be economical to do so, splicing the entire flange by means of butt welds.

6–7. Design the end and intermediate stiffeners for the girder of Prob. 6–5.

6–8. Design the end and intermediate stiffeners for the girder of Prob. 6–6.

6–9. Determine the flange rivet spacing for the girder of Prob. 6–5 at the end and at the one-quarter point.

6–10. Find the moment of inertia of a girder having the following section:

3—14 x ½ cover plates top and bottom,
4—6 x 6 x ¾ flange angles,
1—40 x ½ web plate,
Depth = 40.5 in. back-to-back of flange angles.

(a) Reduce the tension flange for one ¾-in. rivet through each leg of the flange angles, two rivets through the cover plates, and 4-in. spacing in the web. Compute the moment of inertia.
(b) Compute the moment of inertia without any reduction for rivet holes.
(c) If a computed unit stress of 20,000 psi is found from the moment of inertia of (b), what tension unit stress would be found with the use of the moment of inertia of (a)?

CHAPTER 7

TRUSSES

7-1. The Use of Trusses. When the span length becomes too great for the economical use of beams or plate girders, trusses are frequently employed. The dividing line between economical span lengths for rolled beams, plate girders, and trusses cannot be definitely stated for the general case. The available length of rolled beam is frequently a factor determining its maximum span. Conditions of continuity affect the economical span lengths, increasing the economy in the use of rolled beams and plate girder sections. The use of rigid-frame construction has greatly increased the economical spans for rolled or built-up beams. The available space for depth of structure sometimes determines whether a truss may be used.

Trusses are used for supporting roofs of buildings for spans from 40 or 50 ft to spans of 300 or 400 ft. Simple-span truss bridges for highway or railway use are frequently built from 40 ft to 300 or 400 ft in length. Trusses for industrial equipment of various types, such as ore handling equipment, conveyor equipment, and crane booms, are built in spans from 25 ft to 300 ft. Transmission towers, radio towers, and supporting structures for tanks are usually of truss-type construction.

Trusses are structures having the functions of beams and carrying loads which produce bending of the structure as a whole together with shear, but which resist the bending by means of chords and the shear by means of a web system. Trusses are usually considered to be formed with a system of members attached together at their ends to form triangles. The Vierendeel truss is an exception to this general definition of a truss, being an open-web type of beam providing its load-carrying capacity by reason of the resistance of its members to bending. The Vierendeel truss is usually not considered to be a true truss because it lacks the characteristics of a triangulated system of members.

7-2. Types of Trusses. Trusses are classified by the manner in which various systems of triangulation are combined, and they frequently have been named after the engineer first using the particular type of truss. The top and bottom chords may be parallel or inclined, the truss may be simple span or continuous, and the end members may be vertical or inclined.

Trusses may be named by their use, such as highway truss, railway truss, or roof truss. A highway or railway truss may be further defined by the manner in which it supports the load it carries, such as a deck truss, through truss, or half-through truss. The deck truss is built entirely below its load. The load passes between the trusses of a through bridge and below an overhead bracing system. A pony truss is a type of a half-through truss

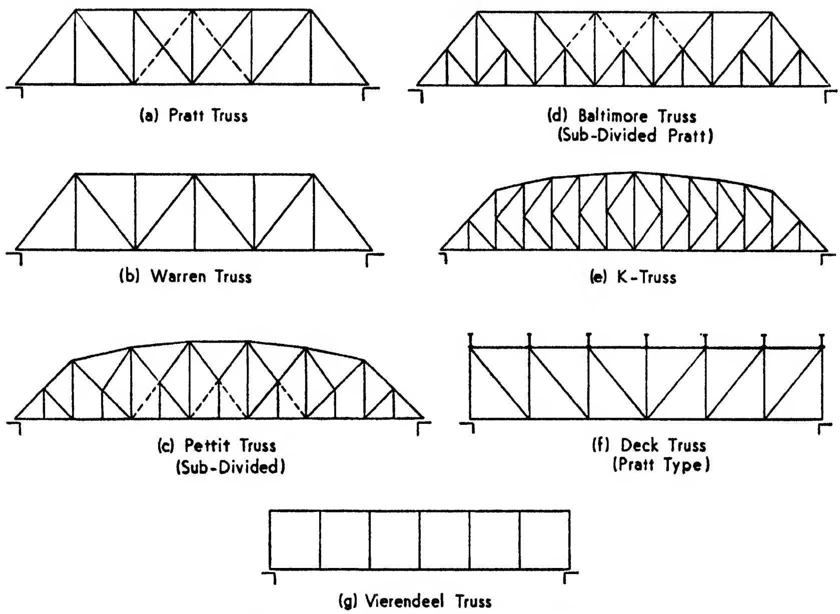


FIG. 7-1. Types of bridge trusses.

bridge which, because of its shallow depth, does not have an overhead bracing system.

A few of the more common types of trusses used for highway and railway bridges are shown in Fig. 7-1. Figure 7-2 shows common types of

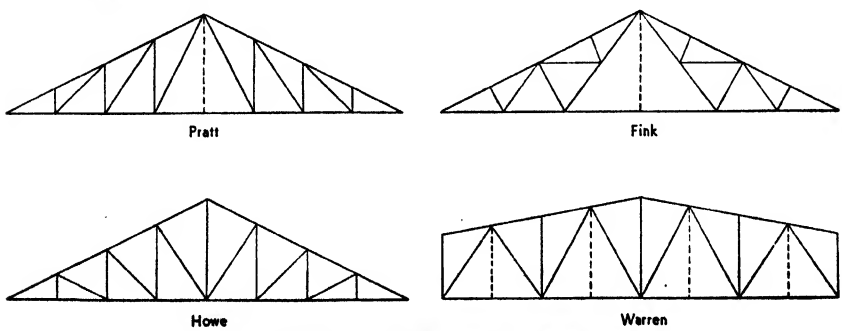


FIG. 7-2. Types of roof trusses.

roof trusses. Figure 7-3 shows a few applications of truss types to bridge construction.

Trusses are further classified by the manner in which their members are formed or attached, as, for example, riveted trusses, welded trusses, single-plane and double-plane trusses, and pin-connected trusses.

Figure 7-10 shows design drawings for a double-plane riveted Pratt truss used as a highway through bridge. Figure 7-4 shows cross sections

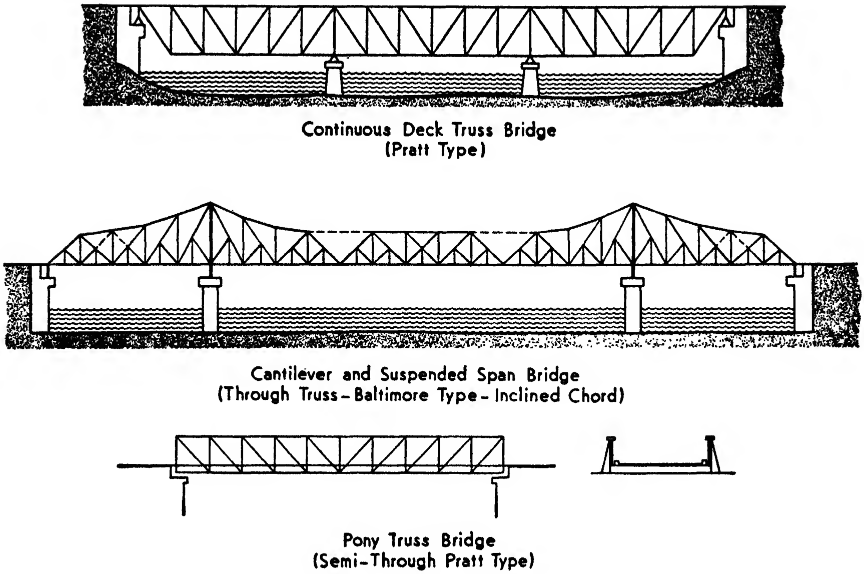


FIG. 7-3.

of a few typical truss members which are either single-plane or double-plane according to whether they are connected together with gusset plates in one or in two planes. Figure 7-5 is a representation of several types of connections used in riveted, welded, or pin-connected trusses.

7-3. Selection of Truss Type. The selection of the type of truss to be used is a matter of filling the requirements of the conditions to be met and of using materials and processes in an economical manner. The type of triangulation to be selected may depend somewhat on the designer's preference, but an important factor to be considered is the angle between intersecting members and its effect on the details. The panel length is usually determined by an economical design for a floor or roof system which the truss supports. The depth of the truss may be determined by the desired pitch of the roof or by an economical span-to-depth ratio for a bridge truss. The depth may be dictated by the clearances necessary and by the necessity for overhead bracing.

When a depth and a panel length have been selected, the angle at which the members meet will depend on the type of triangulation used. For ease in detailing welded or riveted joints, it is desirable that the intersecting angles be between 30 and 60°. For instance, if a bridge truss has been determined to have panels of 20 ft and a depth of 60 ft, the Pratt truss would have angles between vertical and inclined members too small to provide satisfactory gusset plate details. On the other hand, a truss of the subdivided types, such as the Baltimore or Pettit trusses or the K-truss, would provide better inclination of diagonal members for this ratio of panel and depth.

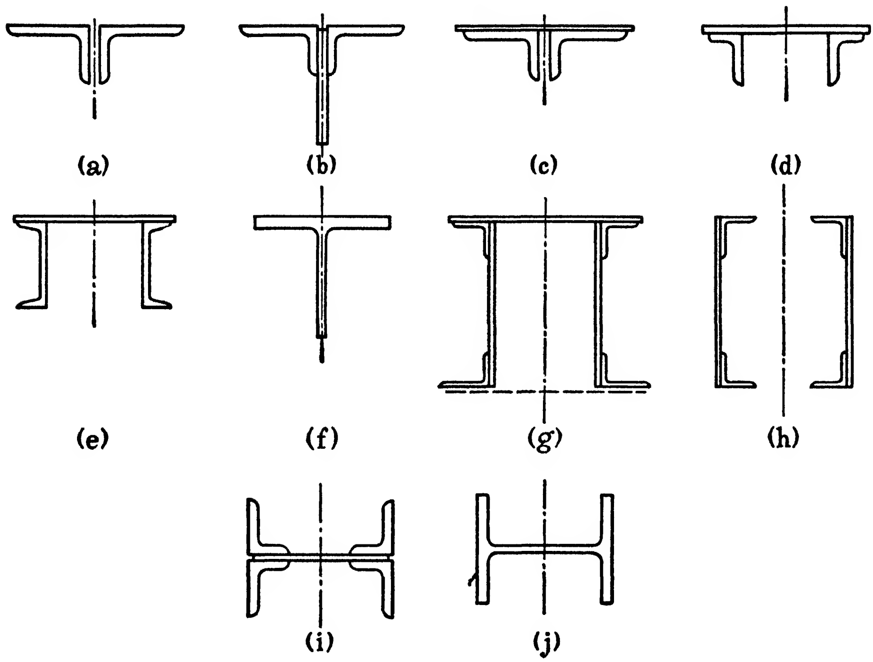


FIG. 7-4.

Prior to about 1912 a majority of American bridge trusses were pin-connected. Because of the unsatisfactory results which came about by wear of the pins in the pin holes and the resulting looseness of joints, the pin-connected truss has become almost obsolete except for very heavy and long spans. Shop equipment for the fabrication of eye-bars is almost non-existent because of the small use of such members. Since 1912 the majority of trusses have been riveted. Recently, welded trusses have been used more and more, particularly for roof construction. Some highway trusses and most industrial trusses are being constructed with welded connections.

7-4. Truss Specifications. Many details of design, as well as limiting dimensions, are set by specifications. The AASHO and the AREA specifications are very specific in the loadings to be used, the manner of distributing those loads, the impact allowances to use, truss depths, and the actual design of the members and details of the trusses used for highway and railway bridges. The AISC specification is usually used for roof truss design.

While these specifications may have many similar provisions, there are details specified in each case that are applied in different manners. Changes in the specifications are made by an evolutionary process and result in closer agreement of many of the provisions. The designer should study the design specifications very carefully because much of the detail is specified very completely. This completeness of specifications has come about

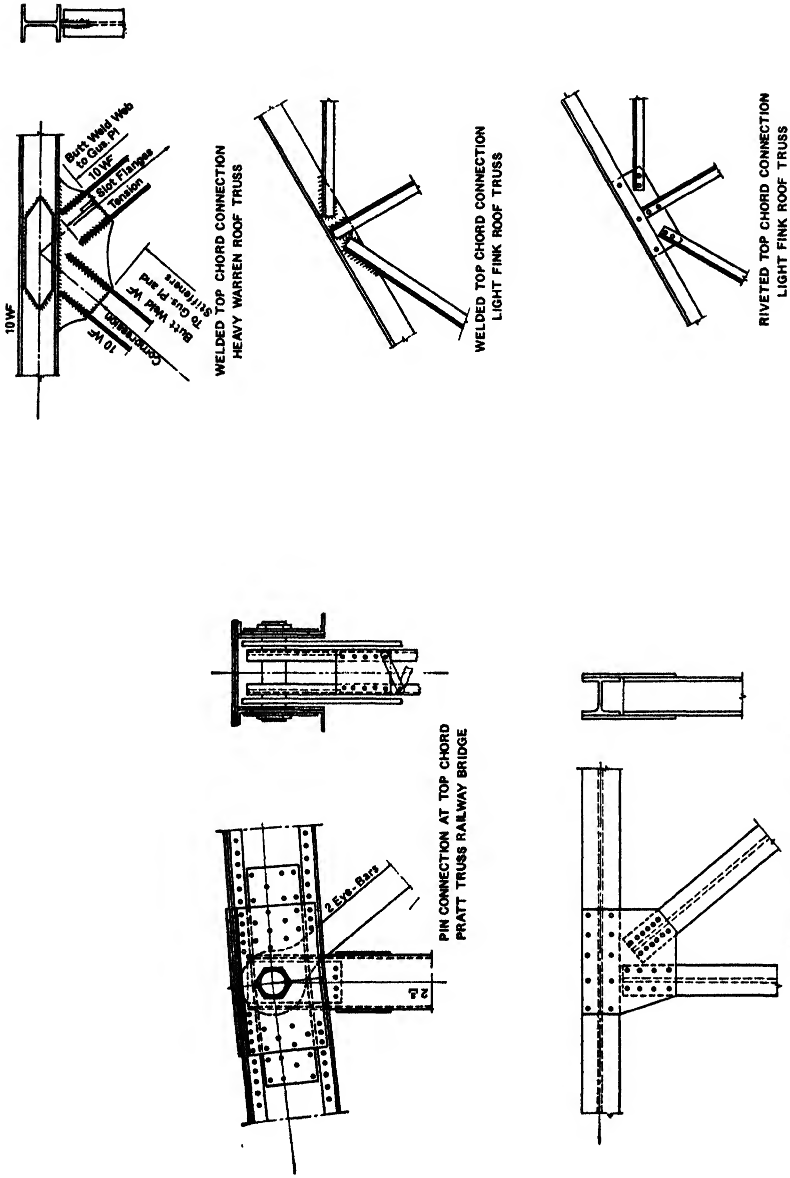
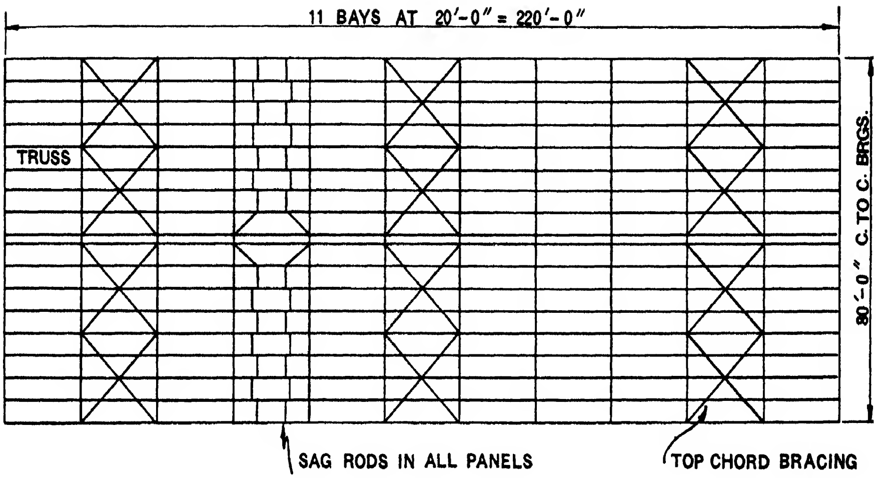
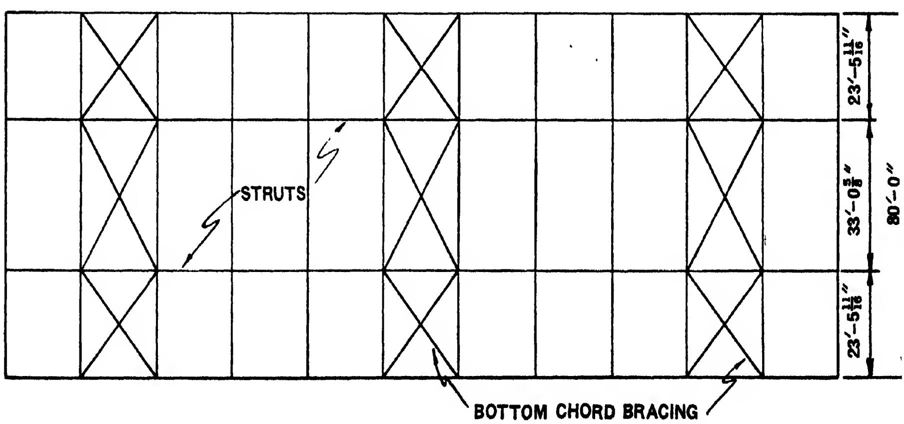


FIG. 7-5. Typical truss connections.



PLAN AT TOP CHORD OF TRUSSES



PLAN AT BOTTOM CHORD OF TRUSSES

Fig. 7-6.

by long use and experience, and full use should be made of this experience. Improvements are to be encouraged, but, in the case of doubt, past experience suggests a safe procedure.

7-5. Roof Steel. Figure 7-6 shows line diagrams of the roof for an industrial building 80 ft by 220 ft. In general, the problem of design consists of the selection of truss type, roof slope, truss spacing, purlin design, truss design, and design of bracing.

The truss selected for illustration is a Fink truss with an 80-ft span. The trusses are spaced 20-ft centers. Figure 7-7 shows the design of the truss and the purlin spacing. The estimated roof loads are as follows:

$$
\begin{array}{lrl}
\text{Roofing} & = & 15.5 \\
\text{Purlins} & = & 3.0 \\
\text{Truss} & = & 3.0 \\
\text{Live load, } \dfrac{20 \times 10}{10.83} & = & 18.5 \text{ (20 lb per sq ft of horizontal projection)} \\
\text{Total} & = & \overline{40.0} \text{ lb per sq ft of roof}
\end{array}
$$

Each purlin carries an area of roof with dimensions 5 ft 5 in. by 20 ft. The total live and dead load above the purlins is then $5.42 \times 37 = 200.5$ lb per lin ft of purlin. This load is vertical and has components parallel and normal to the roof surface. The normal component is $200.5/1.083 = 185$ lb per ft. The component parallel to the roof is $185 \times 0.417 = 77$ lb per ft of purlin.

The design of the purlin depends on the manner in which it is braced. The force parallel to the roof surface must be carried in transverse bending of the purlin. Since the section modulus in this direction is small for most sections, it is usually a more economical design to brace the purlins with sag rods that serve as intermediate supports for the transverse loading. As shown in Fig. 7-7, these sag rods should be placed near the tops of the purlins to support the top flanges. The ridge purlin may be one member directly at the ridge, or it may be two members tied together, as shown in Figs. 7-6 and 7-7. A distinct disadvantage of the single ridge purlin is that it must take additional vertical loads equal to the vertical components of loads on the sag rods on each side of the roof. Figure 7-6 shows the upper ends of the sag rod system terminating at the purlin supports. In this way the ridge purlins are independently braced and carry only the loads coming directly to them.

The purlins should have a depth from 1/24 to 1/30 of their span, preferably about 1/24 of the span. This would indicate that 10-in. purlins could be used.

Assuming that the tie rods will be placed at 4.5 ft from the ends of the purlins, the stresses in the top flange may be compared with the allowable stresses. The transverse moment in the top flange at the ties will be $(77 \times 4.5^2)/2 = 780$ ft-lb. The transverse moment at the center of the purlin will be $[(77 \times 11^2)/8] - 780 = 385$ ft-lb. The bending moments

due to normal forces will be $(185 \times 20^2)/8 = 9{,}250$ ft-lb at the center, and $9{,}250 - 0.55^2(9{,}250) = 6{,}452$ ft-lb at the tie rod.

A 10B17 is shown for the purlins in Fig. 7–7. The allowable compression in the top flange is

$$\frac{12{,}000{,}000}{Ld/bt} = \frac{12{,}000{,}000 \times 4 \times 0.33}{132 \times 10} = 12{,}100 \text{ psi.}$$

The top flange will have compression equal to the transverse moment divided by the section modulus of the top flange plus the normal moment divided by the section modulus of the entire section. The computed stresses are as follows:

At the tie points,

$$\frac{780 \times 12}{1.72} + \frac{6{,}452 \times 12}{16.2} = 10{,}220 \text{ psi.}$$

At the center of the purlin,

$$\frac{385 \times 12}{1.72} + \frac{9{,}250 \times 12}{16.2} = 9{,}537 \text{ psi.}$$

The computed stresses are less than the allowable stress and it may be shown that the next lighter section, 10B15, will not be adequate.

It will be noted that one-half of the section modulus about the y-axis is used in the calculation of stress due to transverse load. The flange that resists the transverse moment must be the flange supported by the sag rods.

Figure 7–8 is a "stress sheet" showing the forces in the truss members caused by the vertical live and dead loads. The panel loads acting on the truss are $(40 \times 20 \times 43.33)/4 = 8{,}667$ lb. One-half of this load will be applied at each purlin. The purlins that are between the panel points bring the loads to the top chord and produce bending moment. The bending-moment stress must be combined with stress from the direct force in computing the unit stress in these members.

In general, purlins should be at the panel points of the trusses. This can be accomplished by subdividing each top chord panel with additional truss members. When the panel length is short, however, it may be cheaper to provide for bending in the chord than to provide the additional members.

The wind loading is shown on Fig. 7–9. The normal loads are computed in accordance with the ASCE recommendations, as shown in Fig. 1–6. The truss bearings provide for expansion at one end, and the horizontal component of the wind forces must be resisted at the other end. In cases where the truss is supported by steel columns, or when expansion is not provided, it is usual to divide this force between the supports.

The bottom chord is designed for a maximum live- and dead-load tension of 72,700 lb. The wind force may be compression, but it cannot exceed the dead-load tension. During erection, however, the truss will be lifted by attachment to intermediate panel points, and the bottom chord will be

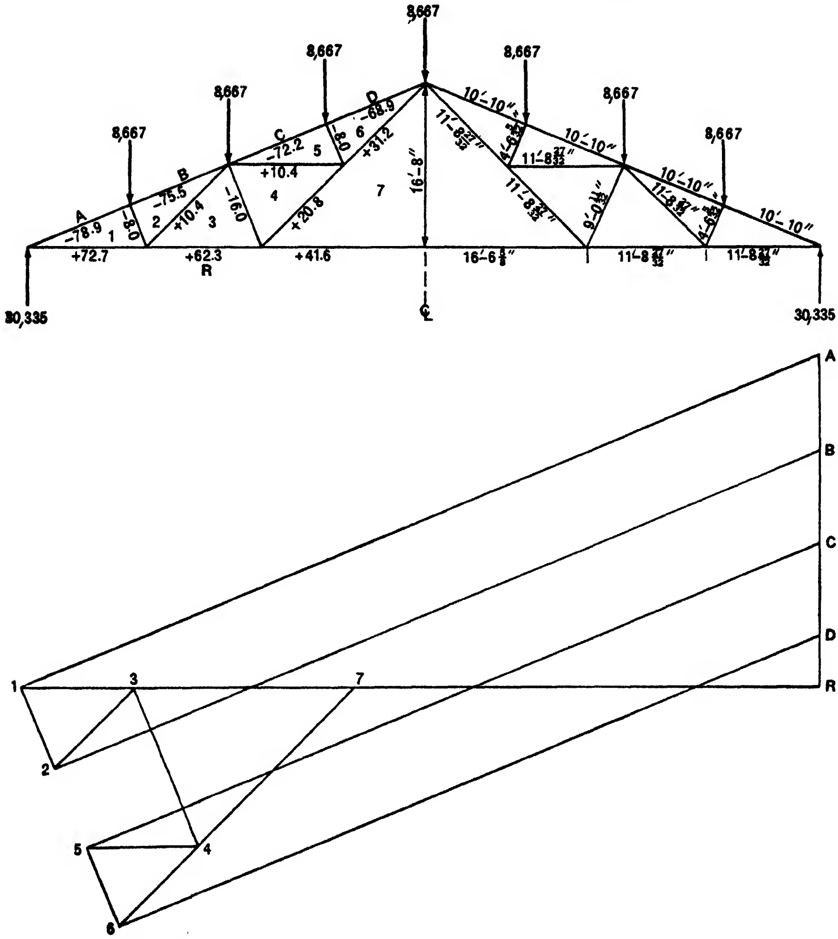


FIG. 7-8. Stress sheet. Dead load and live load on full span.

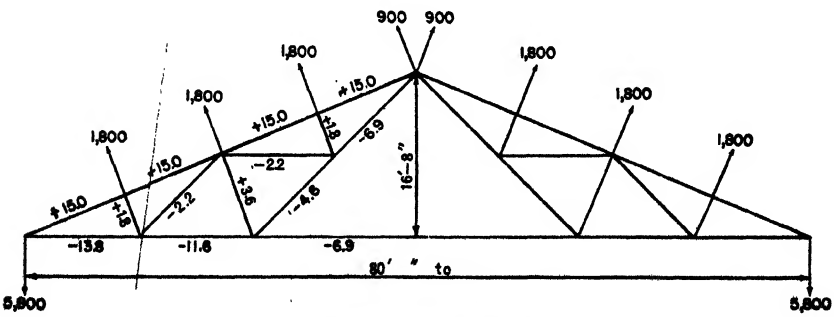


FIG. 7-9. Stress sheet. Wind load.

under a small compressive force. This makes it desirable to consider the L/r of the full span length. If this L/r does not exceed 500 to 600, it is not likely that any trouble will result. The bottom chord should be braced between trusses, as shown in Fig. 7–6, and the maximum unsupported length will be 33 ft ⅝ in. after the truss is erected. If the L/r is 240 when the truss is erected, $r = 397/240 = 1.65$, and the L/r for the full span will be $960/1.65 = 582$.

The top chord will be designed for a dead- and live-load force of 78,900-lb compression. The wind-load force will not need to be considered because it produces tension in the top chord (see Section 15e, AISC specification). The top chord is continuous and the bending moment produced by the 4,333-lb purlin reaction will be about $(4{,}333 \times 10)/8 = 5{,}410$ ft-lb. The top chord is supported at intervals of 5 ft 5 in. against horizontal buckling and at 10 ft 10 in. intervals against vertical buckling. The properties of two 6 x 3½ x ½ angles with the 6-in. legs vertical and ½ in. between backs of the angles are: $r_x = 1.92$ in., $r_y = 1.45$ in., $S_x = 8.5$ in.^{3}, and area = 9.0 sq in. The stresses are then

$$\frac{P}{A} = \frac{78{,}900}{9.0} = 8{,}780 \text{ psi},$$

$$\frac{M}{S} = \frac{5{,}410 \times 12}{8.5} = 7{,}630 \text{ psi}.$$

The allowable bending stress is 20,000 psi; the allowable direct stress is

$$17{,}000 - 0.485\left(\frac{130}{1.92}\right)^2 = 14{,}780 \text{ psi}.$$

Then

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{8{,}780}{14{,}780} + \frac{7{,}630}{20{,}000} = 0.976.$$

Table 7–1 shows the properties of the selected sections. Two angles are used for the light web members to prevent the twisting that accompanies the attachments of one-angle members.

The bracing for the bottom chords is controlled by the requirement for limiting L/r for tension and compression members. The r for the struts must be at least $240/200 = 1.2$, for the long diagonals $465/300 = 1.55$ about the horizontal axis, and for the short diagonals $370/300 = 1.23$. The r about the other axes must be not less than 0.77 in. for the long diagonals, or 0.66 in. for the short diagonals. The struts may be two 4 x 3 x ¼ angles; the long diagonals, one 5 x 3½ x ¼ angle; and the short diagonals, one 4 x 3 x ¼ angle. The diagonal bracing, usually made from round rods, in the plane of the top chord is used to resist the wind loads against the end of the building and to aid in the erection.

7–6. The Highway Truss Bridge Problem. In the example selected, it is required to design a three-lane highway bridge for a 160-ft simple span to carry the H20 loading specified by the AASHO specification. Riveted construction has been selected as most common to this type of structure.

TABLE 7-1

PROPERTIES AND DESIGN OF SECTIONS

Member	Design Load	Length (In.)	Min. r	¾ Rivets Required	Section	Area (Sq. In.)	$\frac{P}{A}$	Allowable $\frac{P}{A}$
Bottom chord.....	+72,700	141 397	0.59 1.65	6	2—3½ × 3 × ⅜ Ls	3.94 net	18,400	20,000
Top chord ..	−78,900	130 65	1.08 0.54	6	2—6 × 3½ × ½ Ls	9.00	8,780	9,130*
U_1L_1 U_3M_3	− 8,000	54	0.45	2†	2—2½ × 2 × ¼ Ls	2.12	3,780	14,680
L_1U_2 U_2M_3	+10,400	141	0.59	2	2—2½ × 2 × ¼ Ls	1.68 net	6,190	20,000
U_2L_2.......	−16,000	109	0.91 0.55‡	2	2—2½ × 2 × ¼ Ls	2.12	7,550	7,760
L_2U_4.......	+31,200	141 282	0.59 1.18	3	2—3 × 2½ × ¼ Ls	2.18 net	14,300	20,000
L_2M_3.......	+20,800	....	......	2	....	......	....	....

* An allowance made for $\frac{M}{S}$: (1.00 − 7,630/20,000) 14,780.

† See Section 21(a) of the AISC specification.
‡ SeeSection 16(b) of the AISC specification.

Vertical clearance over the roadway not less than 14 ft and overhead bracing at least 5 ft in depth are required. (See Articles 3.1.8 and 3.6.68, AASHO specification.) Allowing from 3 to 4 ft for the thickness of the floor system, the truss depth is estimated to be from 22 to 24 ft. The minimum depth of one-tenth of the span (Article 3.6.11) will undoubtedly be satisfied by observing the clearance requirements.

Diagonal inclinations of approximately 45° will provide the easiest details for the connections of members. Dividing the 160-ft span into eight spaces will give 20-ft panels, nine spaces 17.8-ft panels, and seven spaces 22.9-ft panels. Very little difference in cost will be likely to result between panels varying from 17.8 to 22.9 ft. As the panel length increases, the stringers and floorbeams become heavier, and the number of floorbeams and truss members decreases. For convenience in design, an even number of panels will be found to be more satisfactory. Hence, a tentative design will be based on eight panels of 20 ft each. (See Fig. 7-10, Sheet 1.)

7-7. Design of Stringers. The width of roadway for a three-lane highway will be 26 + 12 = 38 ft, according to Article 3.1.8, to which must be added 3 ft for curbs. The clearance between trusses must then be 41 ft and, with an allowance of about 2 ft for the width of a truss, the distance center-to-center of trusses will be about 43 ft. To provide for working clearance in erection, the first stringer should be about 1 ft 6 in. from the truss, or the outside stringers will be about 38 ft apart. Dividing the 38 ft into six equal stringer spacings of 6 ft 4 in. will result in a slab about 6 in. thick, which is about the minimum that should be used. A crowned surface varying from 1 to 4 in. will be used. The stringer design will follow. According to Article 3.3.1(*b*), when S is 6.33 ft, the part of a wheel carried by one interior stringer will be $6.33/5 = 1.266$, and the maximum live-load bending moment will be $8{,}000 \times 10 \times 1.266 = 101{,}300$ ft-lb. The dead load will be $0.83 \times 150 = 125$ lb per sq ft, or $6.33 \times 125 = 791$ lb per lin ft, plus the weight of the stringer, estimated to be 60 lb per lin ft. The impact is maximum on the floor system and is limited by these specifications to 30 per cent as a maximum. The design bending moment will then be as follows:

$$
\begin{array}{lll}
\text{Live load,} & & = 101{,}300 \text{ ft-lb} \\
\text{Impact,} & 30\% & = 30{,}390 \\
\text{Dead load,} & \dfrac{851 \times 20^2}{8} & = \underline{42{,}550} \\
 & \text{Total} & = 174{,}240 \text{ ft-lb.}
\end{array}
$$

The section modulus required is

$$\frac{174{,}240 \times 12}{18{,}000} = 116 \text{ in.}^3$$

and a 21 WF 62 provides an $S = 126.4$ in.^{3}, which should be satisfactory for the interior stringers. The concrete floor slab provides continuous lateral support for the top flanges of the stringers and prevents the necessity of considering transverse loads or reduction of the compressive stress.

The outside stringer receives a load of $0.58 \times 150 \times 6.33/2 = 275$ lb from the slab, and allowing 150 lb per ft for the curb and 55 lb per ft for the stringer, gives a total load of 480 lb per lin ft. The 16,000-lb rear wheel can be placed 2 ft from the curb, and the part of the wheel carried by the outside stringer will be $4.33/6.33 \times 16{,}000 = 10{,}950$ lb. The bending moment received by the outside stringer will be

$$
\begin{array}{lll}
\text{Live load,} & \dfrac{10{,}950 \times 10}{2} & = 54{,}750 \text{ ft-lb} \\
\text{Impact,} & 30\% & = 16{,}425 \\
\text{Dead load,} & \dfrac{480 \times 20^2}{8} & = \underline{24{,}000} \\
 & \text{Total} & = 95{,}175 \text{ ft-lb.}
\end{array}
$$

The required section modulus is

$$\frac{95{,}175 \times 12}{18{,}000} = 63.5 \text{ in.}^3,$$

and a 16 WF 40 will be ample in size for the outside stringers.

The maximum end reaction in the stringer will occur with the 16,000-lb wheel load at the end of the stringer and the 4,000-lb wheel load at 6 ft from the opposite end. Little, if any, distribution of a wheel load placed

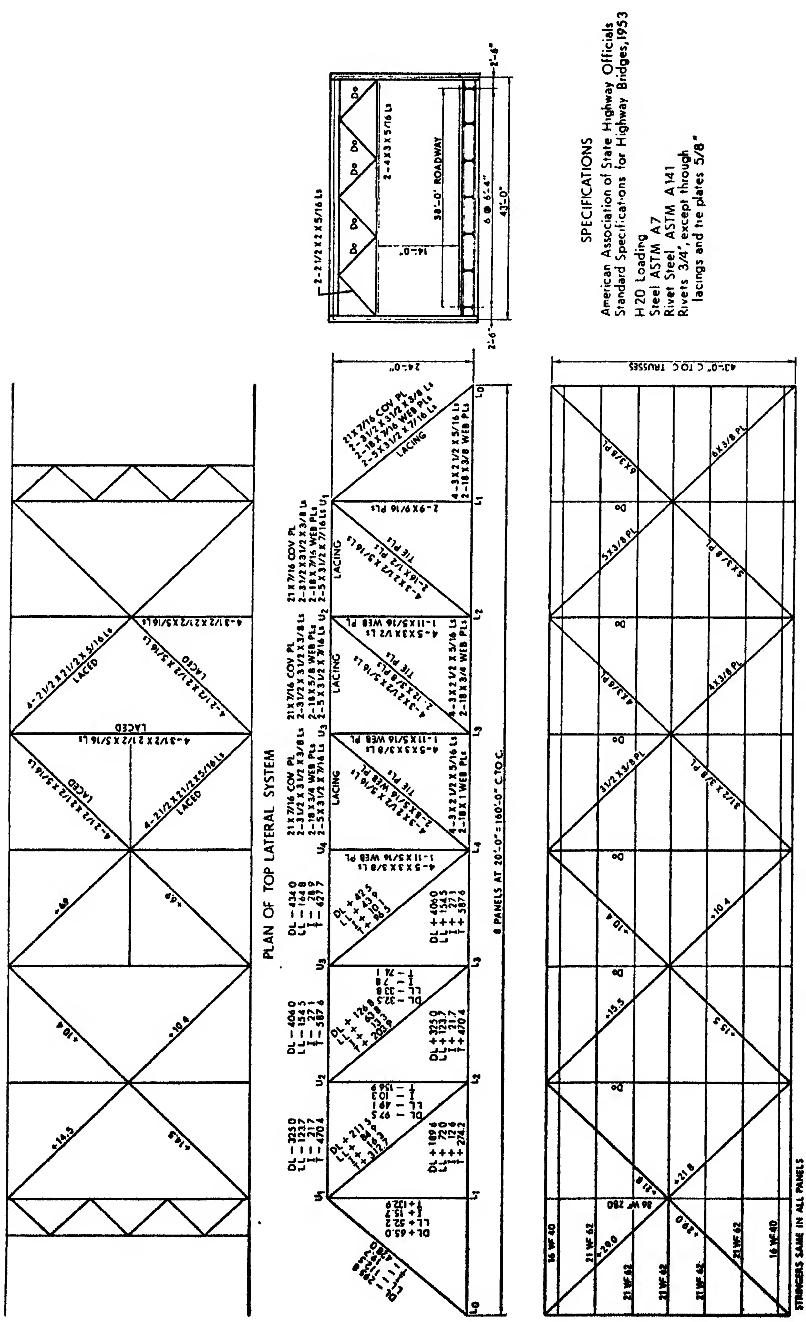


FIG. 7-10. SHEET 1. 160-ft Pratt truss highway bridge, three-lane through type. Stress sheet.

near the end of the stringer can occur. Since wheels of adjacent trucks are only 4 ft apart, and an interior stringer may be midway between two 16,000-lb wheels, the end reaction for an interior stringer will be

$$2 \times 16{,}000 \times \frac{4.33}{6.33} + 1.266 \times 4{,}000 \times \frac{6}{20} = 23{,}420 \text{ lb, live load}$$
$$30\% = 7{,}030 \text{ lb, impact}$$
$$853 \times 10 = \underline{8{,}530} \text{ lb, dead load}$$
$$\text{Total} = 38{,}980 \text{ lb}$$

Since ¾-in. rivets will be used in the design, the number of rivets required to attach the stringer to the floorbeam will be 38,980/5,960 = 7, and the number required to attach the angles to the web of the stringer will be 38,980/8,100 = 5. The shear stress on the web of the interior stringers will be 38,980/(0.4 × 21) = 4,640 psi average.

The end reaction for an outside stringer will be

$$16{,}000 \times \frac{4.33}{6.33} = 10{,}950 \text{ lb, live load, rear wheel}$$
$$\frac{6}{20} \times 4{,}000 \times \frac{4.33}{6.33} = 820 \text{ lb, live load, front wheel}$$
$$30\% = 3{,}530 \text{ lb, impact}$$
$$465 \times 10 = \underline{4{,}650} \text{ lb, dead load}$$
$$\text{Total} = 19{,}950 \text{ lb}$$

The number of rivets required to attach the outside stringers to the floorbeams will be 19,950/5,960 = 4, and the number required to attach the connection angles to the web of the stringers 19,950/6,210 = 4. The average unit shear in the web of the outside stringers will be

$$19{,}950/(0.31 \times 16) = 4{,}020 \text{ psi.}$$

The standard beam connections detailed in the AISC handbook provide sufficient rivets for the end reactions and may be used.

7–8. Design of Floorbeams. The floorbeam will receive its maximum live load from the truck wheels. According to Article 3.2.6, each lane

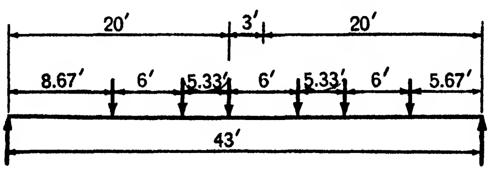


FIG. 7–11.

would occupy 12.67 ft of width. For bending moment the wheels will be placed, as shown on Fig. 7–11, with the center of gravity 20 ft from one end and the point of maximum moment at 20 ft from the other end. According to Article 3.2.9, the live load must be reduced 10 per cent when the three lanes are loaded. This reduction is to provide for the probability that all three lanes will not receive maximum loads at the same time.

The ratio 6/20 of the front wheel loads will be carried by the floorbeam. The concentrations on the floorbeam will then be

$$0.90\left[16{,}000 + \frac{6}{20}(4{,}000)\right] = 15{,}480 \text{ lb.}$$

Stringer dead loads will be 17,060 lb at interior stringers and 9,300 lb at exterior stringers. The maximum bending moment will then be

$$\begin{array}{llrr}
\text{Live load,} & 6 \times 15{,}480 \times \frac{20}{43} \times 20 = & 864{,}000 & \\
 & -30{,}960 \times 8.33 \qquad = & \underline{-257{,}900} & 606{,}100 \text{ ft-lb} \\
\text{Impact,} & 30\% & = & 181{,}800 \\
\text{Dead load,} & 51{,}950 \times 21.5 = 1{,}116{,}900 & & \\
 & 9{,}300 \times 19.0 = -176{,}700 & & \\
 & 34{,}120 \times 9.5 = -324{,}100 & & \\
 & 260 \times 43^2/8 = \underline{\quad 60{,}100} & & \underline{676{,}200} \\
 & & & 1.464{,}100 \text{ ft-lb}
\end{array}$$

The required section modulus for the floorbeam is

$$\frac{1{,}464{,}100 \times 12}{18{,}000} = 976 \text{ in.}^3$$

A 36 WF 280 provides a section modulus of 1,031 in.^{3} and may be used. A plate girder design might be used to save some weight, but a probable increase in unit price per pound would offset some of the economy.

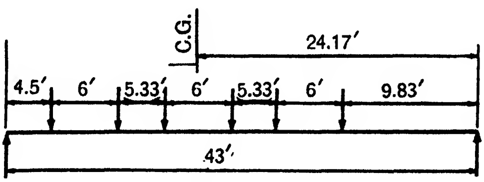


FIG. 7-12.

The end reaction for the floorbeam will be maximum with the live loads located as shown in Fig. 7-12. The reaction computations follow:

$$\begin{array}{llrr}
\text{Dead load,} & \text{Curb: } 20 \times 150 = & 3{,}000 & \\
 & \text{Wearing surface: } 20 \times 0.22 \times 150 \times 19 = & 12{,}550 & \\
 & \text{Slab: } 20 \times 0.50 \times 150 \times 20 = & 30{,}000 & \\
 & \text{Stringers: } 2.5 \times 20 \times 62 = & 3{,}100 & \\
 & \qquad 1.0 \times 20 \times 40 = & 800 & \\
 & \text{Floorbeam: } 20.5 \times 280 = & \underline{5{,}740} & 55{,}190 \text{ lb} \\
\text{Live load,} & 6 \times 15{,}480 \times \frac{24.17}{43} & = & 52{,}210 \\
\text{Impact,} & 30\% & = & \underline{15{,}660} \\
 & & & 123{,}190 \text{ lb}
\end{array}$$

The floorbeam connection must have at least 123,190/5,960 = 21 rivets (¾-in.).

The end connections of floorbeams to the truss cannot be laid out until more information has been developed about the truss. The connections of stringers to floorbeams can be drawn as shown in the cross section of Fig. 7–10, Sheet 3. It is usually better not to cope the ends of stringers, if that can be prevented. In this case, the top of the stringer is dropped 2½ in. below the top of the floorbeam. This will provide for 3½ in. of concrete over the floorbeam flange, which should be sufficient for protection of the flange.

7–9. The Truss Live Load. The "stress sheet" for a structure is a drawing showing the forces acting on each member and the makeup of the various members. Such a drawing is included as Sheet 1 of Fig. 7–10.

The H20 loading may be applied as trucks or as an equivalent lane loading, whichever produces the larger load in the member under consideration. For the truss members it is found that the lane loading is critical, except for member U_1L_1. The lane loading is a uniform load of 640 lb per lin ft spread over a 10-ft width. A concentrated load of 18,000 lb in addition to the uniform load is used for computing bending moment and a 26,000-lb load for shear. Since all members of a truss may be computed by a method of moments, it is interpreted that the 18,000-lb concentration is to be used for the calculation of live load in truss members.

The maximum live load will be carried by a truss when three lanes are loaded as shown in Fig. 7–13. A reduction of 10 per cent is applied to the loading because it is unlikely that all three lanes will be loaded with the maximum load at the same time. The maximum live load carried by a truss will be $(22.83/43) \times 0.90 \times 3 = 1.43$ lanes, and

$$1.43 \times 640 = 915 \text{ lb uniform live load per ft of truss,}$$

$$1.43 \times 18{,}000 = 25{,}740 \text{ lb concentrated load for moment,}$$

$$20 \times 0.915 = 18.3 \text{ kips uniform live load per panel.}$$

The per cent of impact is considered to depend on the loaded length of the span and is equal to $50/(L + 125)$; the maximum impact is limited to 30 per cent of the live load.

The maximum forces in the chord members are computed with the lane load on the entire span. The concentrated load is placed at the panel point about which moments are computed. The forces in the vertical and diagonal web members are computed from the maximum shear in the several panels. Maximum shear occurs in a panel when that panel has as much load as the average panel load for the entire span. For instance, member U_2L_2 receives its maximum load when panel 2–3 has its greatest shear.

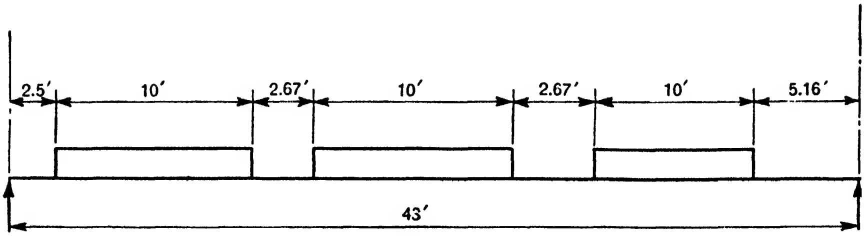


FIG. 7-13. Lane loading.

Maximum live-load shear occurs in panel 2–3 when that panel has 14.3 lin ft of lane loaded and the load extends to the right end of the truss. The average length of loaded lane per panel would then be 114.3/8 = 14.3 ft. The concentrated load would be placed at point L_3.

The computations for the live-load forces in the members of the truss are as follows:

L_0U_1: $18.3 \times 3.5 = 64.0$
$25.74 \times \frac{7}{8} = \underline{22.5}$
Net reaction $= 86.5$ kips

$86.5 \times 1.3 = -112.5$ kips

$$I = \frac{50}{160 + 125} = 17.55\%$$

L_0L_2: $86.5 \times 0.833 = +72.0$ kips

U_1U_2: $18.3 \times 3.5 = 64.0$
$25.74 \times \frac{3}{4} = \underline{19.3}$
Net reaction $= 83.3$ kips
$I = 17.55\%$

$83.3 \times 40 = 3{,}324$
$18.3 \times 20 = \underline{-363}$
$2{,}969$ ft-kips
$2{,}969/24 = -123.7$ kips

L_2L_3: $+123.7$ kips

U_2U_3: $18.3 \times 3.5 = 64.0$
$25.74 \times \frac{5}{8} = \underline{16.1}$
Net reaction $= 80.1$ kips
$I = 17.55\%$

$80.1 \times 60 = 4{,}806$
$36.6 \times 30 = \underline{-1{,}098}$
$3{,}708$ ft-kips
$3{,}708/24 = -154.8$ kips

L_3L_4: $+154.8$ kips

U_3U_4: $18.3 \times 3.5 = 64.0$
$25.74 \times \frac{1}{2} = \underline{12.9}$
Net reaction $= 76.9$ kips
$I = 17.55\%$

$76.9 \times 80 = 6{,}152$
$54.9 \times 40 = \underline{-2{,}196}$
$3{,}956$ ft-kips
$3{,}956/24 = -164.8$ kips

U_1L_2: Loaded length $= 137.14$ ft.

$$\frac{125.4 \times 68.57}{160} = 53.7$$

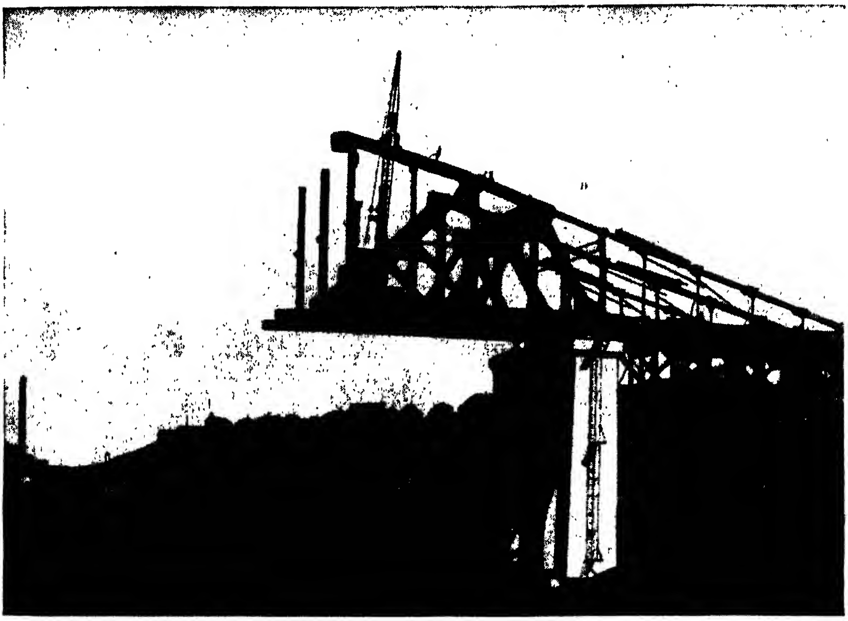
$$25.74 \times \tfrac{3}{4} = 19.3$$

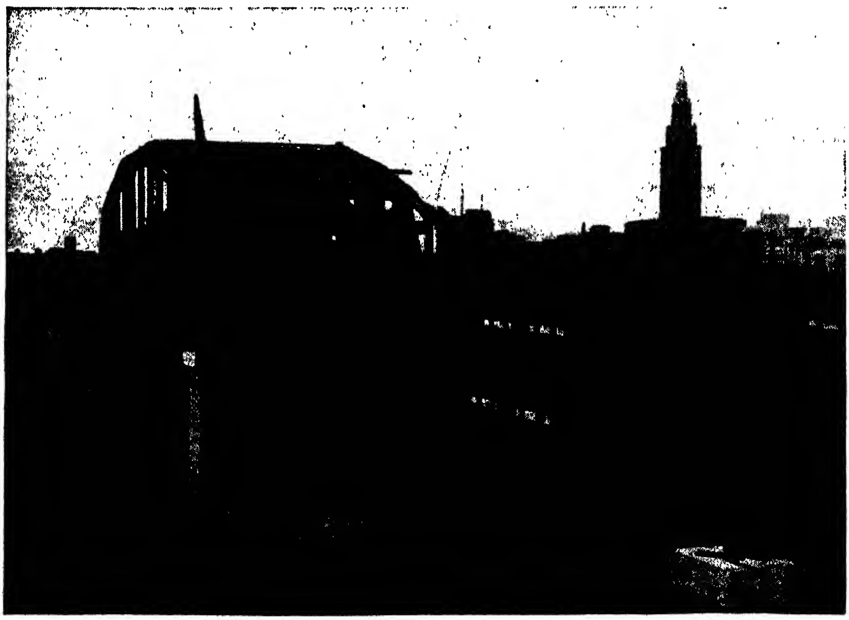
$$\frac{17.14 \times 0.915 \times 8.57}{20} = \underline{-6.7}$$

Panel shear $= 65.3$ kips

$137.14 \times 0.915 = 125.4$ kips
$65.3 \times 1.3 = 84.9$ kips

$$I = \frac{50}{137 + 125} = 19.15\%$$





American Bridge Co.

Erection of New York Central Bridge over Cuyahoga River, Cleveland, Ohio.

U_2L_2: Loaded length = 114.3 ft $114.3 \times 0.915 = 104.6$ kips

$$\frac{104.6 \times 57.6}{160} = 37.7 \qquad I = \frac{50}{114 + 125} = 20.9\%$$

$$25.74 \times \tfrac{5}{8} = 16.1$$

$$\frac{14.3 \times 0.915 \times 7.15}{20} = -4.7$$

Panel shear = 49.1 kips

U_2L_3: $49.1 \times 1.30 = +63.8$ kips

U_3L_3: Loaded length = 91.4 ft $91.4 \times 0.915 = 83.6$ kips

$$\frac{83.6 \times 45.7}{160} = 23.9 \qquad I = \frac{50}{91.4 + 125} = 23.1\%$$

$$25.74 \times \tfrac{1}{2} = 12.9$$

$$\frac{11.4 \times 0.915 \times 5.7}{20} = -3.0$$

Panel shear = 33.8 kips

U_3L_4: $33.8 \times 1.30 = +43.9$ kips

U_4L_3: Loaded length for counter stress = 68.6 ft $68.6 \times 0.915 = 62.8$ kips

$$\frac{62.8 \times 34.3}{160} = 13.5$$

$$25.74 \times \tfrac{3}{8} = 9.7$$

$$\frac{8.6 \times 0.915 \times 4.3}{20} = -1.7$$

Panel shear = 21.5 kips (Less than dead-load shear)

U_1L_1: For wheel loads see Fig. 7-12

$15{,}480 \times 6 \times 24.17/43 = 52.2$ kips

$I = 30\%$

7-10. The Truss Dead Load. The dead load of the floor system comprises the major part of the dead load in wide highway bridges with concrete floors. The forces in the main truss members may be determined for the live load and the floor dead load. The dead load of the truss itself must be estimated.

After live load and impact forces have been computed for the H20 loading, the floor dead load is computed and an allowance is made for the truss. This value of dead load may be used to estimate the dead load forces for a few main members, and a closer value for the weight of the truss may be obtained.

The panel floor load to one truss will be the floorbeam dead-load reaction, or 55,190 lb. Assuming that the truss, portals, and lateral bracing weigh 18 per cent of the floorbeam reaction, a panel dead load of about 65,000 lb results. A panel of main members near the center of the bridge will have forces as follows:

U_3U_4:	Dead load, $-6.67 \times 65{,}000 =$		$-434{,}000$ lb
	Live load		$-164{,}800$
	Impact, 17.6%		$-28{,}900$
		Total	$-627{,}700$ lb
L_3L_4:	Dead load, $6.25 \times 65{,}000 =$		406,000 lb
	Live load		154,500
	Impact		27,100
		Total	587,600 lb
U_3L_4:	Dead load, $0.65 \times 65{,}000 =$		42,300 lb
	Live load		43,900
	Impact		10,100
		Total	96,500 lb
U_3L_3:	Dead load, $-0.50 \times 65{,}000 =$		$-32{,}500$ lb
	Live load		$-33{,}800$
	Impact		$-7{,}800$
		Total	$-74{,}100$ lb

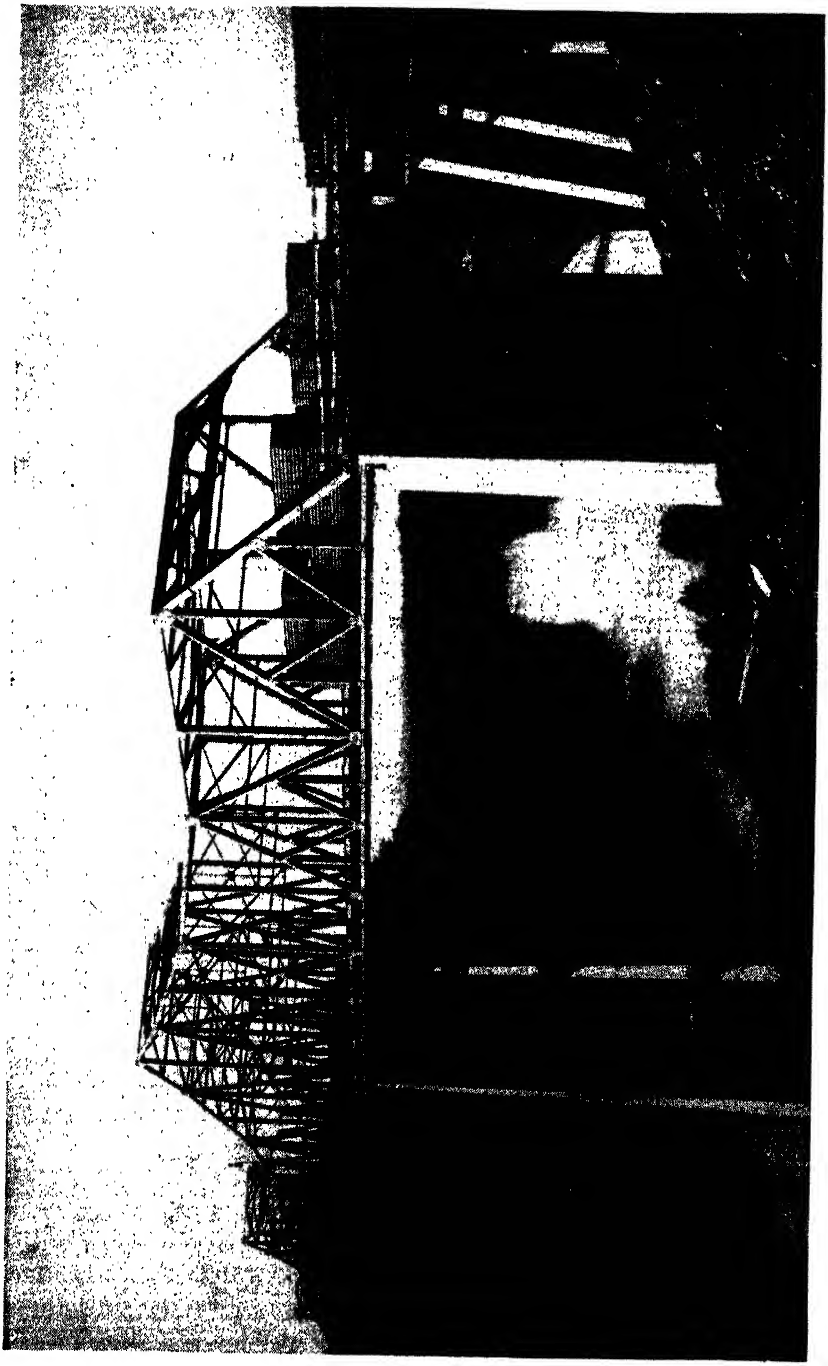
Tension members will be designed for 18,000 psi on the net section. Approximately 20 per cent of the net section is usually taken out of the gross section for rivet holes. Hence, the average stress will be about 15,000 psi on the gross section. For heavy top chords, the average allowable compression will be about 14,500 psi, and for the longer and more slender vertical posts an average stress of about 11,000 psi will be likely. On this basis, the probable areas of members may be estimated and the weights may be figured by multiplying by 3.4, or the weight of a square inch of steel per linear foot.

Member	Force	Stress	Area	Wt/Ft	Length	Weight
U_3U_4	$-627{,}700$	14,500	43.2	147	20	2,940 lb
L_3L_4	$+587{,}600$	15,000	39.2	133	20	2,660
U_3L_4	$+96{,}500$	15,000	6.4	22	31.2	680
U_3L_3	$-74{,}100$	11,000	6.7	23	24	550
			Weight of truss members			6,830
			Floor dead load			55,190
			30% of weight of truss for details			2,050
						64,070 lb

It is seen that the value of 65,000 lb per panel is about as close as the dead load may be estimated in advance of a complete design. An error of 50 per cent in the estimated weight of details will amount to an error of less than 2 per cent in the total dead load.

The stress sheet is completed by the use of a panel load of 65,000 lb and a computation of total forces for each member.

7–11. Selection of Members. Most short-span bridges are designed with their members as principally wide-flange sections. However, the example that follows makes use of built-up members of the type used with longer-span bridges. This is done in the interest of familiarizing the student with



Brown's Bridge over Chattahoochee River.
Four-span continuous through truss bridge illustrating the use of rolled sections for the principal members, designed by the author.

this more difficult design. Bottom chords may be made from angles, plates, combination of angles and plates, or channels, and the top chord should be a covered section.

Examination of the stress sheet of Fig. 7–10, Sheet 1, shows that the top chord is loaded with maximum loads varying from −470,400 lb to −627,700 lb, which will require cross sections from 34 to 45 sq in. Because of the sizes of members required, it is assumed that a double-plane truss will be used.

The bottom chord will require areas varying from about 18 to 40 sq in. on the basis of an average stress of 15,000 psi on the gross section. A double-plane tension member must have its segments tied together with tie plates at intervals; hence, it is desirable to use a combination of plates and angles, with the horizontal legs of the angles providing for riveting.

The floorbeam is much deeper than the probable depth of the bottom chord, and a web connection will be required. It is then desirable that the outer surfaces of the bottom chord be smooth. The angles will be turned with the outstanding legs toward each other, as shown in Fig. 7–14, using minimum legs of 2½ in. to provide working clearance between the toes of the angles. If the four angles are made 3 x 2½ x 5/16, their area will be 6.48 sq in., and two Pls 18 x ⅜ will provide 13.50 sq in.; this section would be about right for L_0L_2. At L_3L_4 the webs could be increased to provide about 22 sq in. additional area, and the webs would be 18 x 1. Clearance between the toes of the angles should be at least 4 in. to provide for fabrication and painting; hence, the gusset plates would need to be at least 11 in. apart.

Proceeding to the top chord, if gusset plates are 11 in. apart and a cover plate is used, that plate would need to be from 20 to 21 in. wide. It is required that rivets ¾ in. or larger be used. This makes it desirable to use angles with 3 or 3½ in. legs. Referring to Article 3.6.15 of the AASHO specification, the thickness of cover plates must be at least 1/40 of the unsupported distance between connecting rivets. With a 21-in. cover plate, the rivet lines would be not more than 18 in. apart, or the thickness of the cover plate must be 18/40 = 0.45 in., which is equivalent to about 7/16 in. A top cover plate 21 x 7/16 will be assumed; two top connecting angles 3½ x 3½ x ⅜ will be assumed; the bottom angles should be somewhat larger than the top angles to lower the center of gravity, so two 5 x 3½ x 7/16 angles are assumed (see Article 3.6.17). The bottom plane of the top chord will be laced between gusset plates, and it is desirable to have the inner surfaces of the chord section free from obstruction to the gusset plates. The sections selected provide an area of 21.21 sq in. The smallest chord section requires about 34 sq in., and two 18 x 7/16 webs will provide 15.75 sq in. A truss looks better if the top and bottom chords have the appearance of being about the same size. The distance between attaching

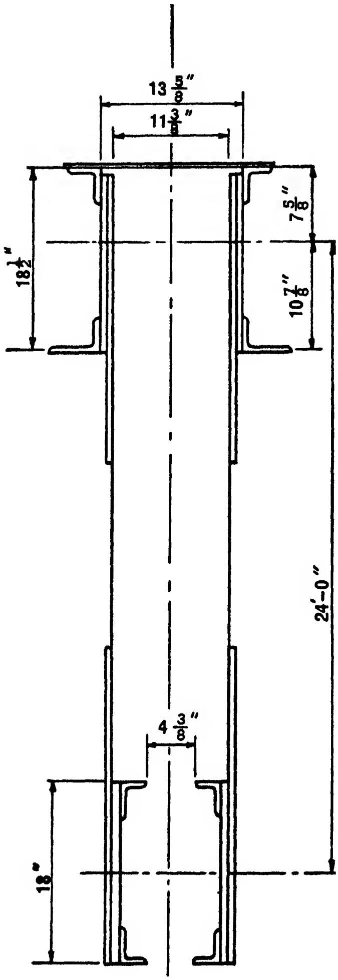


FIG. 7-14.

rivets through the top chord webs will be about 14.5 in. and the minimum thickness according to Article 3.6.15 is 14.5/32 = 0.45 in., or about 7/16 in.

The maximum chord section for U_4U_4 can be obtained by increasing the web thickness to two 18 x 3/4 Pls, which together with the cover plate and angles will give 48.21 sq in.

Certain working dimensions must now be established. It is necessary that the in-to-in dimension between gusset plates be constant in order that the design of web members may be simplified. This dimension should be as large as reasonable. It would, of ocurse, be possible to increase the width of the cover plate, but its thickness would also need to be increased; the webs would then become smaller and the general proportions of the chord would not be so good. On the basis of the tentative sections, the maximum in-to-in dimension of the gusset plates is found by subtracting the dimensions of the legs of the two top angles, the thickness of the maximum webs, and the thickness of the two minimum gusset plates; this will be 21.0 − 7.0 − 1.5 − 0.75 = 11.75 in. Some clearance between the edges of the cover plates and the toes of the angles is desirable; hence, a dimension of 11⅜ in. will be used for the in-to-in of gusset plates. At the bottom chord, a dimension of 11⅜ in. in-to-in of the gusset plates will provide 11⅜ − 5 − 2 = 4⅜ in. between the toes of the angles, which is satisfactory.

To obtain continuity of the angles of the top chord, it is desirable to have the back-to-back dimensions established. At the center of the span the top chord webs are maximum, but gusset plate thickness requirements are minimum (see Article 3.6.14). At this point the gusset plate plus the web will be ¾ + ⅜ = 1⅛ in. The distance back-to-back of angles will then be 11⅜ + 2¼ = 13⅝ in.

It will be noted that as the end of the span is approached, the thickness of gusset plates should be increased, since the loads transferred from the web system to the chord are becoming larger. The thickness of webs decreases toward the end of the span, so that at U_1U_2 the 1⅛-in. thickness of gusset plus web will provide for a 7/16-in. web plus an 11/16-in. gusset. This will likely be ample, but a check may be necessary later in the design.

When the general dimensions of the members have been established, it is necessary to obtain the properties of each truss member and to determine the unit stresses. The members for a truss can frequently be selected entirely from one series of wide-flange beams, such as the 14-in. or 12-in. series. Even though the members may be of the same series, the actual depths will vary, requiring the use of fillers.

7–12. Properties of Truss Members. The following computations include the properties of the selected members, the resulting unit stresses, and the allowable unit stresses in accordance with the specifications.

U_1U_2:

			Axis x–x		
Section	*A*	*d*	*Ad*	*Ad²*	I_o
21 x 7/16	9.19	18.72	172.1	3,222	...
2—18 x 7/16	15.75	9.25	145.8	1,348	425
2—3½ x 3½ x ⅜ Ls	4.96	17.49	86.6	1,513	6
2—5 x 3½ x 7/16 Ls	7.06	0.88	6.1	5	7
	36.96		410.6	6,088	438
				438	
				6,526	
			36.96 × 11.11² =	−4,560	
			I_x =	1,966 in.⁴	

$$y = \frac{410.6}{36.96} = 11.11 \text{ in.}$$

$$r_x = \sqrt{\frac{1{,}966}{36.96}} = 7.29 \text{ in.}$$

Section	A	Axis y–y: d	Ad^2	I_o
21 x 7/16	9.19	....	.....	338
2—18 x 7/16	15.75	6.59	685	...
2—3½ x 3½ x 3/8 Ls	4.96	7.82	303	6
2—5 x 3½ x 7/16 Ls	7.06	8.45	504	18
	36.96		1,492	362
			362	
			I_y = 1,854 in.⁴	

$$r_y = \sqrt{\frac{1{,}854}{36.96}} = 7.08 \text{ in.}$$

U_2U_3:

Section	A	Axis x–x: d	Ad	Ad^2	I_o
U_1U_2	36.96	11.11	410.6	6,088	438
+18 x 3/8	6.75	9.25	62.5	578	182
	43.71		473.1	6,666	620
				620	
				7,286	
			$43.71 \times 10.82^2 =$	−5,117	
			$I_x =$	2,169 in.⁴	

$$y = \frac{473.1}{43.71} = 10.82 \text{ in.}$$

$$r_x = \sqrt{\frac{2{,}169}{43.71}} = 7.05 \text{ in.}$$

Section	A	Axis y–y: d	Ad^2	I_o
U_1U_2	36.96	....	1,492	362
+18 x 3/8	6.75	6.27	266	...
	43.71		1,758	362
			362	
			I_y = 2,120	

$$r_y = \sqrt{\frac{2{,}120}{43.71}} = 6.96 \text{ in.}$$

U_3U_4:

Section	A	Axis x–x: d	Ad	Ad^2	I_o
U_1U_2	36.96	11.11	410.6	6,088	438
+18 x 5/8	11.25	9.25	104.0	962	304
	48.21		514.6	7,050	742
				742	
				7,792	
			$48.21 \times 10.67^2 =$	−5,490	
			$I_x =$	2,302 in.⁴	

$$y = \frac{514.6}{48.21} = 10.67 \text{ in.}$$

$$r_x = \sqrt{\frac{2{,}302}{48.21}} = 6.91 \text{ in.}$$

Section	A	Axis y–y: d	Ad^2	I_o
U_1U_2	36.96	....	1,492	362
+18 x 5/8	11.25	6.21	433	...
	48.21		1,925	362
			362	
			I_y = 2,287 in.⁴	

$$r_y = \sqrt{\frac{2{,}287}{48.21}} = 6.89 \text{ in.}$$

It will be noted that the gravity axis of the top chord varies from 11.11 to 10.67 in. above the base of the section. The working line for the top chord is located at 10⅞ in. above the base, which will provide the minimum effect from eccentricity. The top chord members must be calculated for eccentricity, the P/A stress, and bending stress due to the weight of the member. Since the members are all continuous, it is assumed that the bending moment due to the weight is $wL^2/12$. The maximum combined stress occurs at the ends of the members, and the limiting stress for riveted members applies equally at the ends and center. Appendix B of the AASHO specification gives a limiting value for unit stress composed of direct stress and bending, as

$$f_s = \frac{\dfrac{f_y}{n}}{1 + 0.25 + \dfrac{ec}{r^2}},$$

in which $f_y = 33{,}000$, $n = 1.76$, and e = end eccentricity. All combined stresses in compression members must be compared with this limiting value. The P/A stress must not exceed the value

$$15{,}000 - \frac{1}{4}\left(\frac{L}{r}\right)^2.$$

The minimum section used for the top chord, that for member U_1U_2, will also be used for the end post, L_0U_1.

U_1U_2:

$$\frac{P}{A} = \frac{470{,}400}{36.96} = 12{,}730 \text{ psi} \qquad \frac{ec}{r^2} = \frac{0.24 \times 11.11}{7.29 \times 7.29} = 0.05$$

$$\frac{Mc}{I} \text{ (from ecc.)} = \frac{470{,}400 \times 0.24 \times 11.11}{1{,}966} = 640$$

$$\frac{Mc}{I} \text{ (from D.L.)} = \frac{125 \times 20^2 \times 11.11}{1{,}966} = \underline{280} \qquad \frac{\dfrac{33{,}000}{1.76}}{1 + 0.25 + 0.05} = 14{,}420 \text{ psi, allowable } f_s$$

$$13{,}650 \text{ psi}$$

$$15{,}000 - \frac{1}{4}\left(\frac{240}{7.08}\right)^2 = 14{,}710 \text{ psi, allowable } P/A$$

U_2U_3:

$$\frac{P}{A} = \frac{587{,}600}{43.71} = 13{,}440 \text{ psi} \qquad \frac{ec}{r^2} = \frac{0.06 \times 8.12}{7.05 \times 7.05} = 0.01$$

$$\text{Ecc.} = \frac{587{,}600 \times 0.06 \times 8.12}{2{,}169} = 140$$

$$\text{D.L.} = \frac{145 \times 20^2 \times 8.12}{2{,}169} = \underline{220} \qquad \frac{\frac{33{,}000}{1.76}}{1 + 0.25 + 0.01} = 14{,}900 \text{ psi}$$

13,800 psi (top side at the center)

$$15{,}000 - \frac{1}{4}\left(\frac{240}{6.96}\right)^2 = 14{,}700 \text{ psi}$$

U_3U_4:

$$\frac{P}{A} = \frac{627{,}700}{48.21} = 13{,}000 \text{ psi} \qquad \frac{ec}{r^2} = \frac{0.20 \times 8.27}{6.91 \times 6.91} = 0.04$$

$$\text{Ecc.} = \frac{627{,}700 \times 0.20 \times 8.27}{2{,}302} = 450$$

$$\text{D.L.} = \frac{160 \times 20^2 \times 8.27}{2{,}302} = \underline{230} \qquad \frac{\frac{33{,}000}{1.76}}{1 + 0.25 + 0.04} = 14{,}520 \text{ psi}$$

13,680 psi (top side at the center)

$$15{,}000 - \frac{1}{4}\left(\frac{240}{6.91}\right)^2 = 14{,}700 \text{ psi}$$

L_0U_1:

$$\frac{P}{A} = \frac{428{,}000}{36.96} = 11{,}570 \text{ psi} \qquad \frac{ec}{r^2} = \frac{0.24 \times 11.11}{7.29 \times 7.29} = 0.05$$

$$\text{Ecc.} = \frac{428{,}000 \times 0.24 \times 11.11}{1{,}966} = 580$$

$$\text{D.L.} = \frac{125 \times 1.56 \times 20^2 \times 11.11}{1{,}966} = \underline{440} \qquad \frac{\frac{33{,}000}{1.76}}{1 + 0.25 + 0.05} = 14{,}420 \text{ ps}$$

12,590 psi

$$15{,}000 - \frac{1}{4}\left(\frac{375}{7.08}\right)^2 = 14{,}300 \text{ psi}$$

L_0L_2:

4—3 × 2½ × 5/16 ∟s = 6.48 sq in.
2—18 × 3/8 Pls = 13.50
Gross area = 19.98 sq in.
Holes:
8—3/8 × 7/8 = 2.63
4—5/16 × 7/8 = 1.09 −3.72
Net area = 16.26 sq in.

$$\frac{P}{A} = \frac{274{,}200}{16.26} = 16{,}700 \text{ psi}$$

L_2L_3:

4—3 × 2½ × 5/16 ∟s = 6.48 sq in.
2—18 × ¾ Pls = 27.00
Gross area = 33.48 sq in.
Holes:
8—¾ × 7/8 = 5.25
4—5/16 × 7/8 = 1.09 −6.34
Net area = 27.14 sq in. (See Art. 7-18)

$$\frac{P}{A} = \frac{470{,}400}{27.14} = 17{,}300 \text{ psi}$$

L_3L_4:

4—3 × 2½ × 5/16 Ls	=	6.48 sq in.	
2—18 × 1 Pls	=	36.00	$\dfrac{P}{A} = \dfrac{587{,}600}{34.39} = 17{,}100$ psi
Gross area	=	42.48 sq in.	
Holes:			
8—1 × 7/8 = 7.00			
4—5/16 × 7/8 = 1.09		−8.09	
Net area	=	34.39 sq in.	

U_1L_2:

4—3 × 2½ × 5/16 Ls	=	6.48 sq in.	
2—16 × ½ Pls	=	16.00	$\dfrac{P}{A} = \dfrac{312{,}700}{18.77} = 16{,}660$ psi
Gross area	=	22.48 sq in.	
Holes:			
6—½ × 7/8 = 2.62			
4—5/16 × 7/8 = 1.09		−3.71	
Net area	=	18.77 sq in.	

U_2L_3:

4—3 × 2½ × 5/16 Ls	=	6.48 sq in.	
2—12 × 3/8 Pls	=	9.00	$\dfrac{P}{A} = \dfrac{203{,}900}{12.42} = 16{,}420$ psi
Gross area	=	15.48 sq in.	
Holes:			
6—3/8 × 7/8 = 1.97			
4—5/16 × 7/8 = 1.09		−3.06	
Net area	=	12.42 sq in.	

U_3L_4:

4—3 × 2½ × 5/16 Ls	=	6.48 sq in.	
2—8 × 5/16 Pls	=	5.00	$\dfrac{P}{A} = \dfrac{96{,}500}{9.30} = 10{,}380$ psi
Gross area	=	11.48 sq in.	
Holes:			
8—5/16 × 7/8		−2.18	
Net area	=	9.30 sq in.	

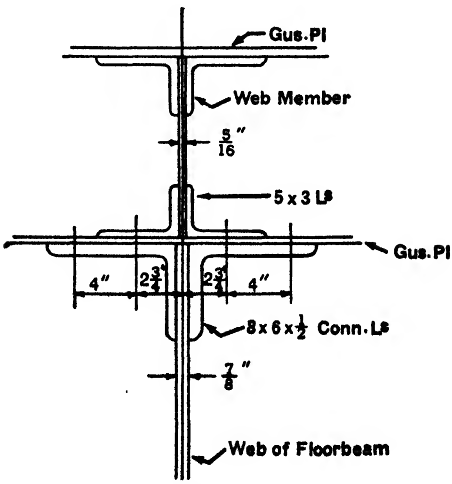
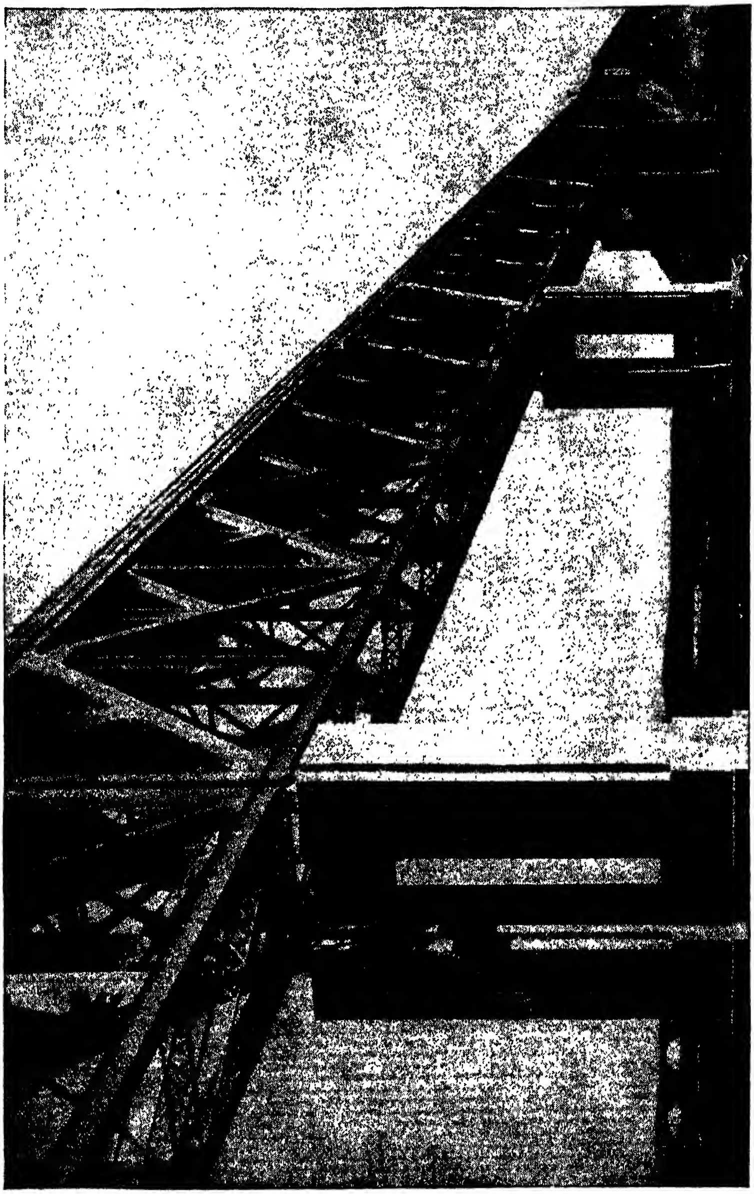


Fig. 7-15.



American Bridge Co.

CIMARRON RIVER BRIDGE, ARKALON, KANSAS.

The design of the vertical members depends on the floorbeam end connection. A little investigation will show that it will be difficult to space the required 21 rivets on two rows of 11 each. Furthermore, the 21 rivets are required to resist shear alone. Stiffness of the vertical members, and deflection of the floorbeam, will induce end moment at the connection of the floorbeam to the truss; hence, it would be better to have a few excess rivets at this connection. For these reasons, four rows of rivets are considered necessary. Two of these rows will pass through the vertical member above the bottom chord. Figure 7–15 shows dimensions of the connection which should be satisfactory.

Article 3.6.9 provides that a portion of a compression member may be neglected in computing the radius of gyration, if the area neglected is not included when computing the unit stress. Members U_2L_2 and U_3L_3 are made from four angles and a web. When the entire section is included in the computations, the L/r is found to exceed the allowable value of 120. However, if only the angles are included, the L/r ratio is satisfactory.

U_2L_2:

Section	A	d	Ad^2	I_o
4—5 x 3 x ½ Ls	15.00	1.90	54.1	38.0
1—11 x 5/16 Pl	3.44	....	....	....
	18.44		54.1	38.0
			38.0	
		$I =$	92.1 in.^{4}	

$$r = \sqrt{\frac{92.1}{18.44}} = 2.24 \text{ in.}$$

or

$$r = \sqrt{\frac{92.1}{15.00}} = 2.48 \text{ in.}$$

$$\frac{P}{A} = \frac{156,900}{15} = 10,460 \text{ psi}$$

$$15,000 - \frac{1}{4}\left(\frac{288}{2.48}\right)^2 = 11,630 \text{ psi}$$

U_3L_3:

Section	A	d	Ad^2	I_o
4—5 x 3 x 3/8 Ls	11.44	1.85	39.2	29.6
1—11 x 5/16 Pl	3.44	....	....	....
	14.88		39.2	29.6
			29.6	
		$I =$	68.8 in.^{4}	

$$r = \sqrt{\frac{68.8}{14.88}} = 2.15 \text{ in.}$$

or

$$r = \sqrt{\frac{68.8}{11.44}} = 2.46 \text{ in.}$$

$$\frac{P}{A} = \frac{74,100}{11.44} = 6,480 \text{ psi}$$

$$15,000 - \frac{1}{4}\left(\frac{288}{2.46}\right)^2 = 11,580 \text{ psi}$$

U_1L_1:

2—9 x $\frac{9}{16}$ Pl

Gross area	=	10.12 sq in.
Holes:		
4—$\frac{7}{8} \times \frac{9}{16}$	=	1.97
Net area	=	8.15 sq in.

$$\frac{P}{A} = \frac{132{,}900}{8.15} = 16{,}310 \text{ psi}$$

7–13. Erection Procedure and Its Effect on Design. One of the first requirements for a proper design is that it be possible to construct the structure. Before a truss design is put on drawings, a well-planned erection procedure should be outlined. The gusset plates should be shop-

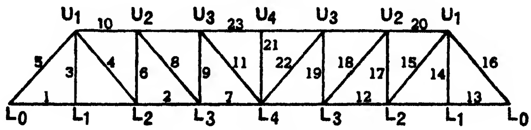


FIG. 7–16.

riveted to the correct members so that they will be at hand in the proper order of erection. In general, the gussets will be shop-riveted to the chords rather than to the web members in order to facilitate the field driving of rivets and to reduce the number of field rivets.

The truss of Fig. 7–10 is laid out with the plan of erecting the bottom chord members on falsework. During the erection procedure, 50 per cent of the open holes are to be filled with bolts and pins. One-half of the filled holes are to be filled with cylindrical pins with diameters 1/32-in. less than the holes. After the members are all in place, the field holes are filled with driven rivets. Assuming that the bottom chord is supported on falsework, the truss of Fig. 7–10 will be erected in the order shown on Fig. 7–16. Floorbeams and stringers can be filled in as fast as vertical members are erected.

After the erection plan has been decided, the joints can be designed with the proper connections indicated for field rivets.

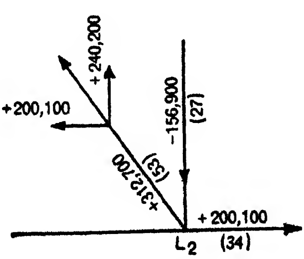
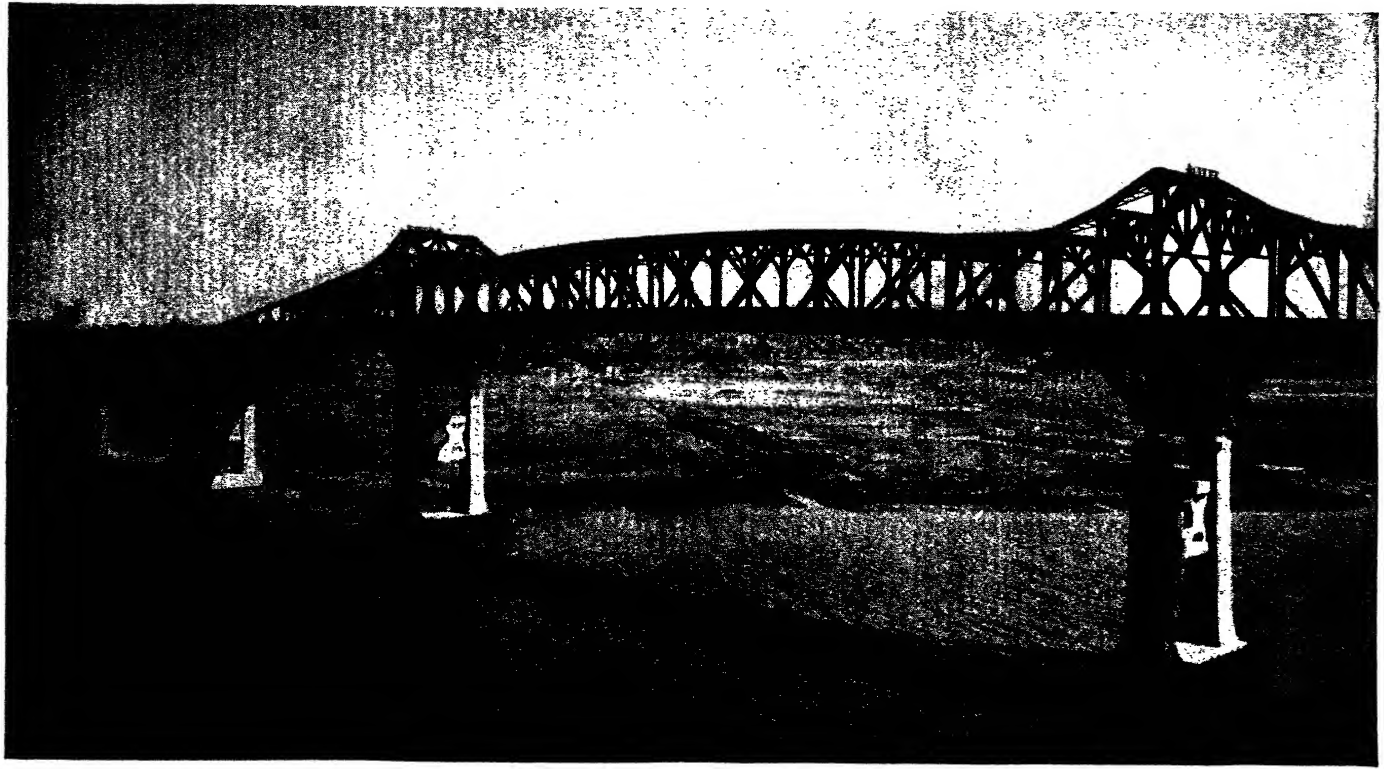


FIG. 7–17.



Bethlehem Steel Co.

BRIDGE CROSSING OVER HACKENSACK RIVER, NEW JERSEY PULASKI SKYWAY.

7-14. Joint L_2. The truss joint at L_2 is typical of bottom chord connections. Figure 7-17 shows the system of forces for which connections must be sufficient. The forces shown are not simultaneous because the maximum force in U_2L_2 occurs under a different position of live load than that causing the maximum force in U_1L_2. The maximum transfer of vertical load from U_2L_2 is $-156{,}900$ lb, which requires 27—¾-in. rivets in single shear. The maximum increase in tension from member L_1L_2 to member L_2L_3 will occur when the tension is U_1L_2 is maximum and will be equal to 200,100 lb, which means that the gusset plates must be attached to the chord with at least 34—¾-in. rivets.

The gusset plate will be shop-riveted to member L_2L_3. Gage lines for rivets should be laid out on all members with the idea of using the same lines throughout the length of the members. This will permit the material to be fabricated with the use of multiple punches. The pitch of rivets along these lines may be varied to suit the need at each point.

Open holes through the gusset plate, U_2L_2, and the bottom chord must be provided for the floorbeam connection. Article 3.6.95 of the AASHO specification requires diaphragms in the truss at the ends of floorbeams. This diaphragm serves to carry one-half of the floorbeam load to the outside plane of the truss and to maintain a right section through the bottom chord. Vertical gage lines on member U_2L_2 are laid out to agree with Fig. 7-15 and are extended across the bottom chord. The bottom flange of the floorbeam should be flush with the bottom of the chord to permit the lateral connection plates to attach to both chord and floorbeam. The bottom gage line on the chord cannot be used for field rivets in the floorbeam connection because the bottom flange of the floorbeam interferes with driving clearance. This row will be shop-riveted with countersunk rivets through the gusset plate and chord. It is then found that 16 rivets will be provided through the floorbeam connection and the bottom chord. Eight of these rivets engage the diaphragm and transfer one-half of the load to the opposite gusset plate. The member U_2L_2 requires 27 rivets, one-half of which are in the near-side and one-half in the far-side gusset plates. Eight of the rivets through the member U_2L_2 and the gusset plate also pass through the floorbeam connection. The outside gage line of the floorbeam connection should be used, and a total of 32 rivets are provided in the floorbeam connection. It will be noted that certain of the rivets required on the near-side plane of the truss are omitted from the far-side plane and that all rivets through the far side of the bottom chord are shop rivets. All floorbeam connections should be alike, and the pattern developed for L_2 may now be laid on all bottom chord joints.

The member U_1L_2 must be attached to the gusset plates with 53 rivets, one-half of which are in the near plane and one-half in the far plane. These rivets should be laid out symmetrically about the centerline of the member, and care should be taken not to reduce the net section more than was assumed in the computation of the unit stress. It is found that five rows of five rivets and one row of three rivets will be convenient. The net section was computed with six holes out of the member. Three holes are shown out of each segment of the member on the first row through the gusset plate, and five on the second row. A check must be made to determine whether the three rivets attaching the gusset plate are sufficient to develop the difference between the net area at the first row of five rivets and the row of three rivets. The value of three rivets is $3 \times 5{,}960 = 17{,}880$ lb. The value of the section taken out by two holes through the $\frac{1}{2}$-in. plate will be $\frac{7}{8} \times \frac{1}{2} \times 18{,}000 = 7{,}880$ lb. It is obvious that enough of the gusset plate tension value has been developed by the three rivets to permit the additional reduction made by the two extra holes through the web at the second row. The group of 28 rivets must be laid out on the member at sufficient distance from the L_2 connection to clear the floorbeam connection. After the rivet group for members U_2L_2 and U_1L_2 has been laid out, the outline of the gusset plate can be drawn. The gusset plate need only enclose the rivet groups, but there are certain rules of design that help in the appearance of the gusset plates. The appearance

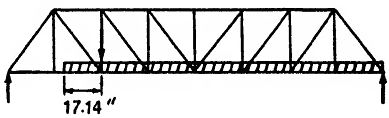


FIG. 7–18.

is usually improved if the edges of the gusset plates cross the members perpendicular to the centerlines of the members. The outline of the gusset plate should have no re-entrant angles, and the external angle between any member and the edge of a gusset plate should be 90° or more.

It is usually found that there is sufficient room on the gusset plate to provide all the rivets necessary through the chord. In the example used, it will be noted that the full number of 36 rivets is provided in excess of those connecting the floorbeam.

7–15. Gusset Plate Investigation. The gusset plate at joint L_2 must be investigated for stress. A critical section through this plate is along the top line of rivets through the bottom chord. Figure 7–19 shows the forces acting on the section at this line. The live load was placed in the position shown in Fig. 7–18 for maximum force in U_1L_2. With the live load in this position, the force in U_1L_2 is $+312{,}700$ lb, and in U_2L_2 the force is $-123{,}000$ lb. The vertical component of the force in U_1L_2 is $+240{,}200$ lb. This vertical component is equal to the force in U_2L_2 combined with the floorbeam reaction of 117,200 lb under the condition of loading used. One-half of the floorbeam reaction is transferred to the gusset plates below the section under consideration, and the other half above that section. The horizontal component of 200,100 lb is a shearing force at the section and represents the increment of bottom chord force produced by the member U_1L_2.

From the system of forces shown in Fig. 7–19, it is seen that the section being investigated is subjected to shear, direct force, and bending moment. The amount of bending moment depends on the location of the neutral axis of the acting section. The section of gusset plate taking the forces is

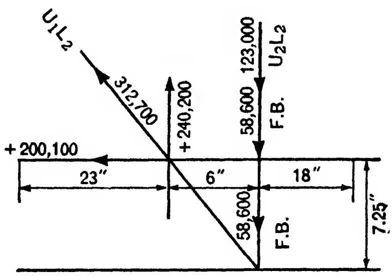


Fig. 7–19.

composed of two 47 x ½ Pls with holes at about 6-in. centers. The net area on the tension side of the plates is equivalent to a plate without holes but of a thickness equal to (6 − ⅞)/6 = 0.85 in. Holes are not deducted from the compression area; hence, the center of gravity of the acting section may be found as x, when

$$\frac{0.85x^2}{2} = \frac{(47 - x)^2}{2},$$

or when $x = 24.4$ in. from the edge of the plate on the tension side. The acting section will then be as shown in Fig. 7–20, and the area of this section

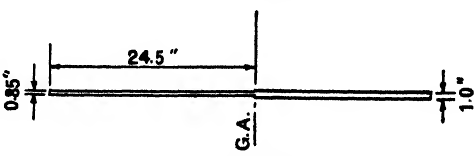


Fig. 7–20.

is $(0.85 \times 24.4) + (1.0 \times 22.6) = 43.3$ sq in. The moment of inertia of the acting section is

$$\frac{0.85 \times 24.4^3}{3} + \frac{1.0 \times 22.6^3}{3} = 7{,}960 \text{ in.}^4$$

The section is acted upon by a direct force of 58,600 lb and a clockwise moment of $(240{,}100 \times 1.4) + (181{,}600 \times 4.6) = 1{,}171{,}500$ in.-lb about the center of gravity of the acting section.

The maximum tension on this section is calculated to be

$$\frac{P}{A} + \frac{Mc}{I} = \frac{58{,}600}{43.3} + \frac{1{,}171{,}500 \times 24.4}{7{,}960} = 4{,}950 \text{ psi,}$$

which is a very low value of stress.

7–16. Top Chord Splice at U_3. Top chord splices can best be made at a panel point, thus utilizing the gusset plate as a part of the splice material. The members U_2U_3 and U_3U_4 must each be attached to the splice material with sufficient rivets to transfer their loads to the splice, and the splice material must be sufficient to take the loads. Rivets transferring the loads must be located properly to transfer the portion of the load carried by each element of the member.

The cover plate and outstanding legs of the top angles are spliced with a cover plate splice; the outstanding legs of the bottom angles are spliced with separate plates; and the remainder of the section is spliced with gusset plates and additional vertical splice plates. The distribution of rivets to transfer the total load is as follows:

Section	*Area*	*Rivets*
½ of two 3½ × 3½ × ⅜ ∟s	= 2.48	
Cov. Pl 21 × 7/16	= 9.19	
	11.67 sq in.	$\frac{11.67 \times 14{,}700}{5{,}960} = 29$ rivets
2—5 × 7/16 outstanding legs	= 4.38	$\frac{4.38 \times 14{,}700}{5{,}960} = 11$
2—18 × ¾ web Pls	= 27.00	$\frac{27.00 \times 14{,}700}{5{,}960} = 67$
2—18 × ⅝ web Pls	= 22.50	$\frac{22.50 \times 14{,}700}{5{,}960} = 56$
Vertical legs of ∟s	= 5.02	$\frac{5.02 \times 14{,}700}{5{,}960} = 13$

The cover plate splice must have at least 11.67 sq in. of area and must be attached with at least 29 rivets to each chord section. The bottom flange splice plate must have 2.19 sq in. and must be attached with six rivets on each side. The remaining splice material must be equal to the area of U_3U_4 minus the material otherwise provided; that is, $48.21 - 16.05 = 32.16$ sq in. The thickness of web splices will be $32.16/18 = 1.79$ in., or 0.89 in. in each plane. The ⅜-in. gusset plates will provide part of this material, but two ½-in. plates must be added. These ½-in. plates should pick up

$$\frac{0.5 \times 18 \times 14{,}700}{5{,}960} = 23 \text{ rivets.}$$

7–17. Joint U_1. At the hip of the truss, joint U_1, the cover plate cannot be spliced in the same manner as at U_3 because of the change in angle between the top chord and the end post. Stress from the cover plate in the top chord must be transferred to the gusset plate and out again to the cover plate of the end post. This is accomplished by means of an angle on the inside of the chord section attached to the cover plate and to the gusset plate. The force in the cover plate, the top angles, and a portion of the web plates must be transferred to the gusset plates through the top row of rivets on each side, most of which are in double shear. To ensure that the rivets are properly distributed, the center of gravity of all acting shear areas should coincide with the working line of the section, and the total number should be enough to develop the section.

Rivets	*Arm*	
20 ×	16.5	= 330
13 ×	12.75	= 166
11 ×	9.25	= 102
11 ×	5.75	= 63
9 ×	2.0	= 18
64		679

Center of gravity of shear areas:

$$\frac{679}{64} = 10.6 \text{ in. above base.}$$

The number of rivet shears required to develop the entire section is

$$\frac{36.96 \times 14{,}700}{5{,}960} = 92.$$

In the two planes, 128 single shears have been provided. The excess is largely because of the requirement of Article 3.6.21 that the maximum pitch in the ends of compression members should not exceed four diam-

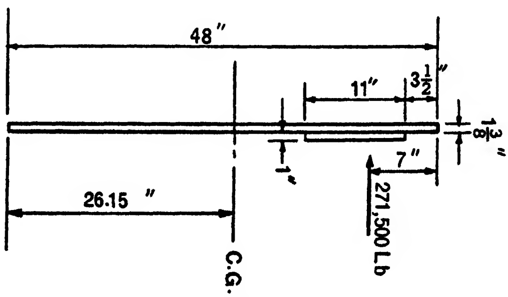
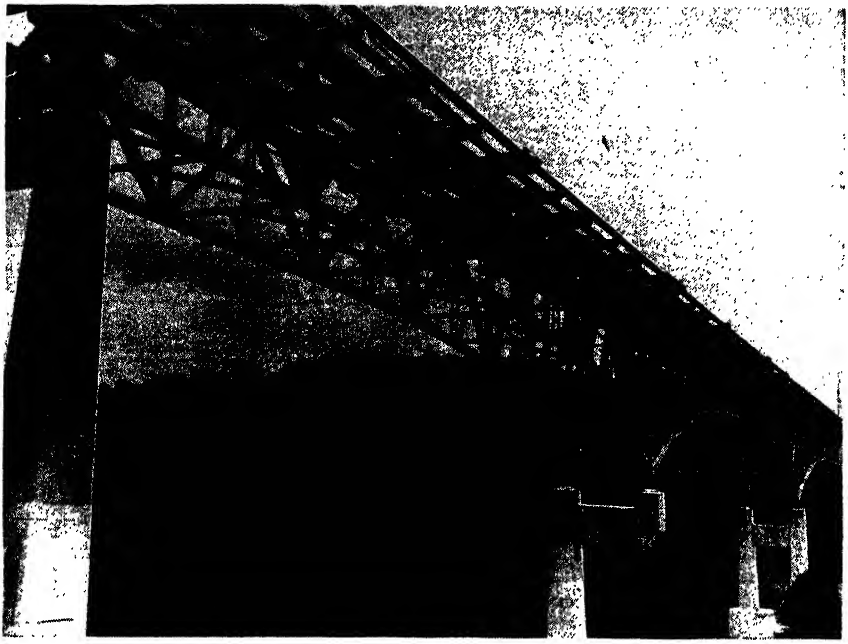


FIG. 7–21.

eters of the rivets for a distance of 1½ times the maximum width of the member. (This article also requires that beyond this distance the pitch should be gradually increased for a distance of 1½ times the maximum width of the member until the maximum pitch is obtained.) When the connection has been made to members U_1L_1 and U_1L_2 and the outline of the gusset plate has been drawn, it will be seen that more rivets are engaged than necessary to develop the member. The small eccentricity of



American Bridge Co.

DECK TRUSS OVER BEAVER RIVER, BEAVER COUNTY, PA.

the rivet group, indicated by the location of the center of gravity of the rivets, is not important, since there is a considerable excess of rivets.

The gusset plates at U_1 should be investigated for unit stress at a vertical section where all chord material is cut. At this section the gussets are about 48 in. deep and are $^{11}\!/_{16}$ in. thick. There are two ½-in. reinforcing plates on the outside of the chord. The section is at the intersection of the members and is shown in Fig. 7–21. The load on the section is equal to the top chord force minus the horizontal component of the load in U_1L_2, placed eccentrically with respect to the acting section. The properties of this section are found as follows:

Section	*A*	*d*	*Ad*	*Ad*²	I_o
48 x 1⅜	66	24	1,584	38,000	12,672
11 x 1	11	39	429	16,750	111
	77		2,013	54,750	12,785
				12,785	

$$\bar{y} = \frac{2{,}013}{77} = 26.15$$

$$I \text{ about base} = 67{,}533 \text{ in.}^4$$
$$77 \times 26.15^2 = -52{,}654$$
$$I = 14{,}879 \text{ in.}^4$$

The maximum force in U_1U_2 occurs when the bridge is fully loaded. The horizontal component of the simultaneous load in U_1L_2 is 198,900, and 470,400 − 198,900 = 271,500. The unit stress at the top edge of the gusset plate is

$$\frac{P}{A} + \frac{Mc}{I} = \frac{271{,}500}{77} + \frac{271{,}500 \times 14.85 \times 21.85}{14{,}879} = 3{,}630 + 5{,}920 = 9{,}550 \text{ psi.}$$

At the top of the reinforcing plate,

$$\frac{P}{A} + \frac{My}{I} = 3{,}630 + \frac{271{,}500 \times 14.85 \times 18.35}{14{,}879} = 8{,}500 \text{ psi.}$$

At the bottom edge of the reinforcing plate,

$$\frac{P}{A} + \frac{Mc}{I} = 3{,}630 + \frac{271{,}500 \times 14.85 \times 7.35}{14{,}879} = 5{,}510 \text{ psi.}$$

The force taken by the reinforcing plate is

$$\frac{(8{,}500 + 5{,}510)5.5}{2} = 38{,}500 \text{ lb.}$$

Rivets required to attach the reinforcing plate are 38,500/5,960 = 7.

7-18. Bottom Chord Splice. Two bottom chord splices are shown in the truss design of Fig. 7-10. The splices are similar and are designed to provide for the full net value of the bottom chord.

The horizontal legs of the angles are spliced with a ⅜-in. tie plate, and the vertical legs and web are spliced with a web splice plate. For the splice between L_2 and L_3 the splice plate is $^{13}\!/_{16}$ in. The actual net area of the chord L_2L_3 is

$$33.48 - (6 \times \tfrac{7}{8} \times \tfrac{3}{4}) - (4 \times \tfrac{7}{8} \times \tfrac{5}{16}) = 28.45 \text{ sq in.}$$

The net section has a tension value of 28.45 × 18,000 = 511,000 lb. Twelve ⅝-in. rivets at 4,140 lb = 49,860 lb, and 82—¾-in. rivets at 5,960 lb have a value of 488,720 lb. The total rivet value is then 538,600 lb.

7-19. Joint L_0. The member L_0U_1 is connected to the gusset at joint L_0 in a manner similar to the connection at U_1. The cover plate must, however, have its stress transferred to the inside splice angles before the gusset plate is reached. The size of the gusset plates is determined by the connection of L_0U_1 and the floorbeam. It is necessary to place the bearing below the bottom chord in order to have room for the end floorbeam connection above the pin. The bearing area on the pin is furnished by the gusset plate and the angles of the diaphragm.

The end reaction of 376,200 lb requires 15.7 sq in. of bearing area at 24,000 psi. Assuming a 6-in. diam pin and angles ½ in. thick, the bearing area will be

$$(6 \times 1) + (5\tfrac{5}{8} \times 1) + (6 \times 1\tfrac{3}{8}) = 19.87 \text{ sq in}$$

The web should be bored with the other material and will provide a small amount of additional bearing. See Chap. 8 for further information on this point.

Trim angles and cover plates usually follow the outline of the ends of the gusset plates to close the space and to keep dirt and water from falling through to the bearing.

7–20. Laterals and Portal Bracing. The top and bottom lateral systems are designed for wind load. In the case of a truss bridge located on a curved roadway, it would be necessary to design for centrifugal forces.

The bottom lateral system is usually plates or angles. When angles are used, the limiting lengths specified in Article 3.6.9 of the AASHO specification must be observed. Article 3.6.68 specifies the type of lateral and portal systems to be used. The members of the lateral and portal systems are usually selected to meet limiting slenderness requirements.

When the distance between trusses is considerably greater than the panel lengths, the bottom lateral panel length may be made twice the panel length of the main truss; the laterals will then cross a floorbeam at its center. At this point a lateral plate must be designed to splice the laterals. This plate will be attached to the bottom flange of the floorbeam. The floorbeam should be investigated to determine whether sufficient section has been provided to permit rivet holes through the bottom flange; if not, the lateral plate may be welded to the bottom flange.

7–21. Camber. Trusses having horizontal bottom chords are usually cambered. If the chords are straight when erected, the truss will have the appearance of being "sagged" at the center. In order to avoid this appearance of excessive deflection, it is customary to erect the truss with the bottom chord panel points raised to an elevated position, such that the dead-load deflection plus some part of the live-load deflection will not cause the bottom chord to deflect below a straight line. Camber is not provided because of any stress consideration but is purely for appearance.

Camber may be accomplished in several ways. The tension members may be made shorter and the compression members longer than the geometrical lengths by amounts equal to the stretch or the shortening due to stress. Then, when the truss members receive their computed stresses, the truss will take the desired geometrical shape.

Another method of accomplishing camber is to lengthen the top chord members by an amount equal to their shortening under load. The bottom chord members are not changed in length but are blocked up to the required camber during erection.

Camber for highway bridges is usually provided for the full amount of the dead-load and live-load deflection, without impact. Railway bridges are usually designed for camber equal to the dead-load deflection plus a part of the live-load deflection.

PROBLEMS

7-1. Compute the wind forces in accordance with the Duchemin formula for the truss of Fig. 7-9 when $p = 20$ lb per sq ft. Investigate the truss for the resulting forces and redesign any members necessary.

7-2. A bottom chord is composed of 12—18 x ½ Pls (two segments of six plates each). Design a lap splice for the net section that would result when six holes for ⅞-in. rivets are taken out of each segment (see Art. 5-30).

7-3. Investigate the top and bottom lateral systems of the highway truss of Fig. 7-10 for conformity with the AASHO specification.

7-4. Investigate the bottom chord splice in member L_1L_2, Fig. 7-10.

7-5. Design a welded roof truss for the loading of Figs. 7-8 and 7-9.

CHAPTER 8

BEARINGS

8–1. Beam Bearing Plates. At the ends of beams supported by direct bearing on masonry or concrete, it is usually necessary to use bearing plates. The chief purpose of the plate is to distribute the beam reaction over a large enough area so that the masonry is not overstressed in compression. A bearing plate also facilitates erection by providing a smooth surface, easily adjusted to the desired elevation, on which the beam may be rested.

A typical bearing plate is illustrated by Fig. 8–1. If the masonry is extended over the beam (as indicated by dotted lines), some form of wall anchor is used. The type shown is called a *government anchor* and consists of a bent steel rod. A pair of short clip-angles, connected to the web, might be used instead of the rod. No structural strength is assumed to be provided by the anchor. Its purpose is to prevent longitudinal movement of the beam with respect to the wall. However, if longitudinal loads of appreciable proportions are expected, anchor bolts extending vertically into the masonry and bonded to it would probably be used instead of the anchor shown.

The beam reaction causes pressure between the masonry and the bearing plate. The intensity of pressure p is assumed for design purposes to be uniform over the area of the plate. A small upward deflection of the outer edges of the plate does occur, of course, relieving the pressure near the outer edges and increasing the pressure near the center, as shown in Fig. 8–1(*b*). The end of the beam rotates, causing an increase in pressure along the edge of the bearing plate nearest the center of the beam span and a corresponding decrease at the edge near the end of the beam. The stress may vary in a longitudinal direction as shown in Fig. 8–1(*c*). With greater rotation or smaller reaction, the distribution may be triangular. With even greater rotation a portion of the plate near the end of the beam may be ineffective. These distributions assume, of course, elastic action of the wall or pedestal material. In overstressed portions the masonry may undergo plastic deformations which tend to make the pressure distribution nearly uniform. In any case, the pressure is usually assumed to be uniform, and the specification allowable average unit compressive stresses in the masonry are low in order to compensate for the nonuniformities of pressure distribution.

Two methods of analysis for bearing plates are in general use. In the first method, the bearing pressure of the beam flanges on the plate and the bearing pressure of the plate on the concrete are both considered to be

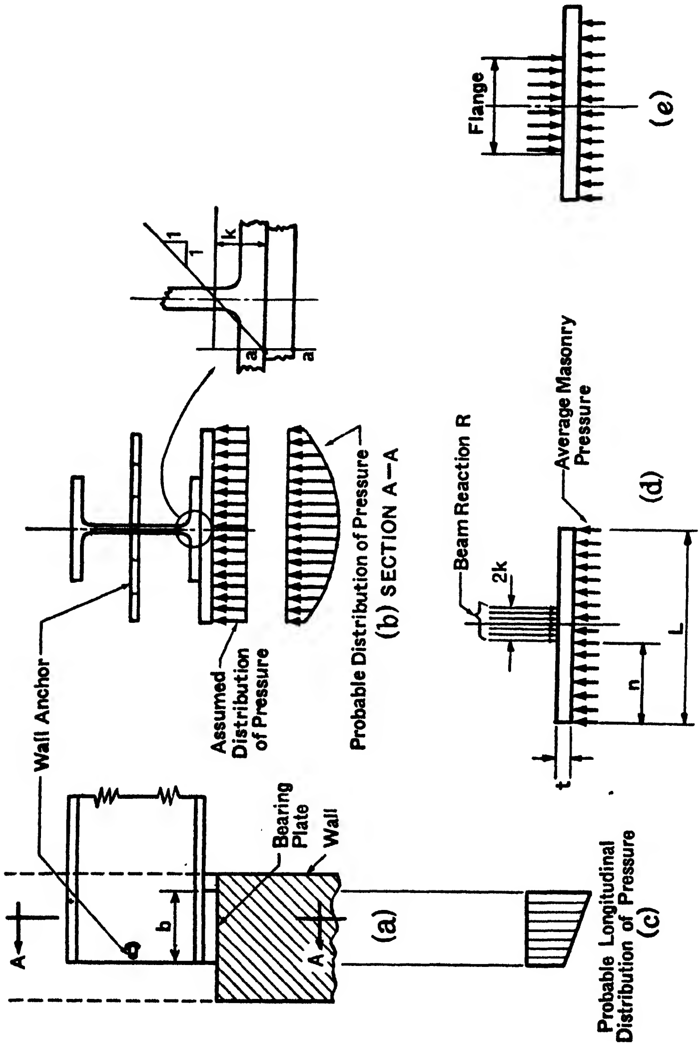

Wall Anchor
Bearing Plate
Wall
b
A
(a)
Probable Longitudinal Distribution of Pressure
(c)
Assumed Distribution of Pressure
Probable Distribution of Pressure
(b) SECTION A—A
a
k
1
Beam Reaction R
2k
Average Masonry Pressure
t
n
L
(d)
Flange
(e)

FIG. 8-1.

uniformly distributed. The critical sections of the bearing plate are at the edge of the beam flange and near the center of the beam. The larger bending moment at either section may control the bearing plate thickness. This method of design appears to assume that there is no deflection of the flange and bearing plate, and it would seem to be justified when web stiffeners are used over the bearing, thus distributing the load over the width of the flange. The bending moment at the center of the bearing plate is equal to one-eighth of the reaction times the difference between the plate width and the flange width. [See (Fig. 8–1(*e*).]

When bearing stiffeners are not used, the outer portions of the flange may bend upward, reducing the pressure between the beam and the plate near the outer edges and increasing it near the center. An appreciable convex curvature of the flange is often observed on rolled sections. Thus, either through bending or through initial curvature, the pressure of the beam on the plate will be more nearly as shown in Fig. 8–1(*d*). The empirical design method of AISC uses this assumed distribution.

In the AISC method it is assumed that the bearing plate provides the entire resistance to the bending moment and that the critical section is at a distance k from the center of the web. The value of k is shown as a property of the beam sections and is equal to the flange thickness plus the fillet radius. Figure 8–1(*d*) shows the distribution of load on the bearing plate as assumed by the AISC method.

Example 8–1. A 16 W 36 beam having an end reaction of 25 kips rests on a masonry wall. If the allowable bearing pressure is 300 psi on the masonry, design a bearing plate in accordance with the AISC specification.

Area of bearing required is 25/0.3 = 83.3 sq in.

The dimension b required to prevent web crippling (see Chap. 4) is

$$\frac{R}{24t\ (\text{web})} - k = \frac{25}{24 \times 0.299} - 0.94 = 2.54 \text{ in.}$$

A stiffener is not required to prevent web crippling if a bearing greater than 2.54 in. is provided, measured in the direction of the beam. A practical bearing dimension in the direction of the beam span would be 7 in. on a wall consisting of three rows of bricks. (This will allow a 1-in. setback of the plate from the edge of the wall. Such a setback is desirable in order to avoid local failure of the edge of the masonry wall. With larger loads, even more setback might be desirable.)

If one dimension of the bearing plate is 7 in., the other dimension must be 83.3/7 = 11.9 in. A plate 7 x 1-0 would be satisfactory. The average masonry pressure would then be

$$\frac{25{,}000}{84} = 298 \text{ psi.}$$

The dimension $k = 0.94$ in. and $n = 5.06$ in. [Fig. 8–1(*d*)]. Hence, in accordance with the AISC method, the bending moment at the critical section is

$$298 \times 5.06 \times \frac{5.06}{2} = 3{,}820 \text{ in.-lb per linear in. of plate.}$$

The required section modulus for the plate is

$$\frac{3{,}820}{20{,}000} = 0.191 \text{ in.}^3$$

The required thickness of plate is then

$$\sqrt{6 \times 0.191} = 1.07 \text{ in.}$$

A $1\frac{1}{16}$-in. Pl could be used. A $1\frac{1}{8}$-in. Pl would probably be used, however, since it is a more commonly stocked thickness.

EXAMPLE 8–2. Redesign the bearing plate for the beam of Ex. 8–1, knowing that the beam will have bearing stiffeners at the center of the bearing length. The bearing area required is the same as before. Try a plate measuring 7 x 1–0.

Since stiffeners are used over the bearing, it is expected that the bearing pressure between flange and bearing plate will be practically uniform. This pressure would be

$$\frac{25{,}000}{7 \times 7} = 510 \text{ psi.}$$

The maximum bending moment in the bearing plate would occur at the centerline of the beam and would be

$$(298 \times 6 \times 3) - (510 \times 3.5 \times 1.75) = 2{,}240 \text{ in.-lb per linear in.}$$

The required section modulus for the plate would be

$$\frac{2{,}240}{20{,}000} = 0.112 \text{ in.}^3,$$

and the thickness of the plate would be

$$\sqrt{6 \times 0.112} = 0.82 \text{ in.}$$

Hence, a 13/16-in. Pl may be used if web stiffeners are also used.

8–2. Column Base Plates. A typical column base plate detail is shown in Fig. 8–2. The plate may be welded to the column as illustrated in (*b*), or it may be riveted by means of connecting angles to the flanges or to the flanges and the web of the column. If the column end is milled, its connection to the plate need be strong enough only to hold the parts together. If the column is not milled, the connection is required to transmit the entire column load to the plate.

For an undeformed plate, the upward pressure p would be uniformly distributed over the entire area $A \times B$. Because of elastic deformation of the plate and the supporting material, there may be a reduction of pressure near the edges of the plate and an increase near the portion beneath the column flanges and web. As with beam bearing plates, the

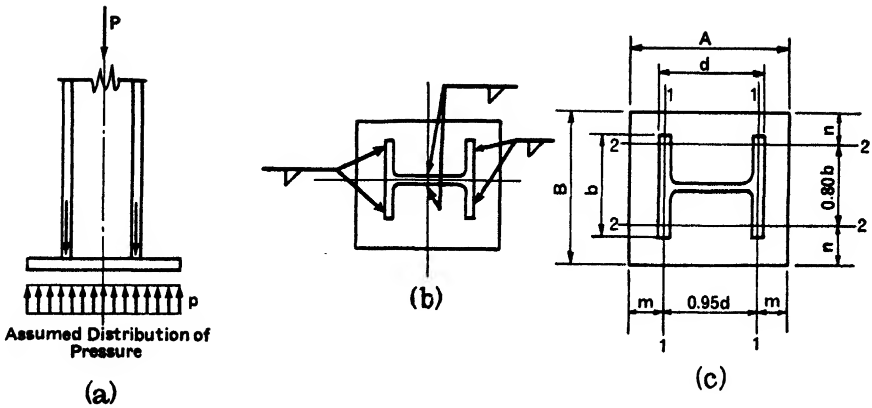


FIG. 8-2.

masonry bearing pressure allowed by specifications is kept low, so that pressure distribution can be safely assumed to be uniform. The plate design based on this assumption is conservative.

The downward force P is distributed over the area of contact between the column and the plate. The assumed uniform pressure p tends to bend upward those portions of plate which project beyond the outline of the column, as well as the portion included between the beam flanges. In the AISC method the portion between the flanges is ignored, while the strength of the projecting portions is assumed to be critical. The projecting portions are considered as uniformly loaded cantilever beams. The bending moment is maximum near the point where the flange contacts the plate. It is usually assumed that the maximum moment occurs on the sections marked 1–1 and 2–2 in Fig. 8–2(c). The values $0.95d$ and $0.80b$ are arbitrary but are selected to meet average values.

For section 1–1,

$$M = \frac{pm^2}{2} \text{ per inch strip of plate (average),}$$

or

$$M = \frac{Bpm^2}{2} \text{ for the entire overhanging portion of plate.}$$

Since

$$= \frac{M}{S}, f = \frac{pm^2}{2} \times \frac{6}{t^2} = \frac{3pm^2}{t^2}.$$

Similarly, for section 2–2,

$$f = \frac{3pn^2}{t^2}.$$

For a satisfactory base design, the flexural stress f must not exceed the allowable for the material projecting from either section 1–1 or section 2–2.

Using the 1949 AISC allowable flexural stress of 20 kips per sq in., the above equations may be rewritten as follows:

$$t^2 = 0.15pn^2,$$

and

$$t^2 = 0.15pm^2,$$

in which p is the bearing pressure in kips per sq in.

The required thickness is the larger value indicated by these equations. (For allowable unit flexural stresses other than 20 kips per sq in., the equations are similar but have a constant other than 0.15.)

EXAMPLE 8–3. Design a column base to develop the full capacity of a 10 WF 49 column, 18 ft 8 in. long, laterally supported at the top and bottom only. Use the allowable unit stresses of the 1949 AISC specification for steel and an allowable bearing stress of 500 psi on the concrete footing.

The column $L/r = 224/2.54 = 88.2$. The allowable column stress is

$$\frac{P}{A} = 17{,}000 - 0.485 \times (88.2)^2 = 13{,}220 \text{ psi.}$$

The allowable column load is then

$$P = 13{,}220 \times 14.40 = 190{,}000 \text{ lb, or } 190 \text{ kips.}$$

When the bearing pressure $p = 0.5$ kips per sq in., the bearing area required is

$$A = \frac{190}{0.5} = 380 \text{ sq in.}$$

A plate measuring 19 x 20 in. will provide the necessary area. The position of the plate will be that for which the plate bending moments are the least, unless the position is otherwise controlled by detail requirements, such as the location of anchor bolts. If the flange width is not greater than the depth of the column section, the better position of the plate is with its long dimension parallel to the web. In Fig. 8–2 it will be seen that

$$m = \frac{20}{2} - \left(0.95 \times \frac{10}{2}\right) = 5.25 \text{ in.,}$$

$$n = \frac{19}{2} - \left(0.80 \times \frac{10}{2}\right) = 5.50 \text{ in.}$$

Since n is greater than m, the maximum bending moment will occur along sections 2–2 on Fig. 8–2(c). Thus,

$$t^2 = 0.15 \times 0.5 \times (5.5)^2 = 2.27.$$

The required thickness is then

$$t = \sqrt{2.27} = 1.51 \text{ in.}$$

A plate 20 x 1½ x 1–7 will be used.

8–3. Column Bases to Develop End Moment. Columns are frequently loaded eccentrically, or may have lateral loads which cause end moments. The column bases are then required to transmit end moment and perhaps shear as well as direct load into the supporting foundations. Such bases may be divided broadly into two classes as follows:

1. Attached bases, in which the plate is firmly connected to the column, as in Fig. 8–3.
2. Unattached bases, in which the column merely rests on the base plate, the two being either entirely unjoined or attached by light riveting which serves only to keep the plate and column properly aligned with each other prior to completion of construction. This type is shown in Fig. 8–6.

The actions of the two types of base are different, and so are the analysis or design procedures. The two types will be considered separately in Arts. 8–4 and 8–5.

8–4. Moment Resisting Bases, Plate Attached. It is quite common with smaller sizes of column to connect the base plate to the column in the fabricating shop. Welding is the most common method of connection used, although bases can also be connected by riveting. When the welding or riveted detail is strong enough to transmit the column load and moment, the plate and column act as a unit and are so considered in the analysis.

There are many methods of analysis for such column bases. In all the methods, however, pressure variations between the base plate and the foundation, as described in the next two paragraphs, are recognized.

When the column is loaded with axial loads alone, the upward pressure of the foundation on the plate is assumed to be uniform, as shown in Fig. 8–3(a). The total downward load on the base is the sum of the column load and the initial tensile loads T_1 in the anchor bolts. The unit upward pressure is the total downward load divided by the area of the base plate.

The effect of a small end moment M is to increase the pressure on one side of the center and to decrease it on the other, as shown in Fig. 8–3(b). The pressure at any point is equal to $p = P/A \pm Mx/I$, in which x is the distance of the point from the center of gravity line of the plate, measured in the plane of the moment M. By using this same equation for larger values of M, the distribution of pressure shown in Fig. 8–3(c) is obtained. Such a distribution is not possible, however, because tensile stress cannot exist between the foundation material and the plate. The tensile force required to maintain equilibrium is provided by the anchor bolt near the left edge of the plate illustrated. The distribution of pressure is shown in Fig. 8–3(d). The base is subject to the simultaneous application of force P and the end moment M on the column, as well as to the initial tension in the bolts and to the force C (resultant of bearing pressures). Note that in Fig. 8–3(d) the tension T_1 in the right bolt has

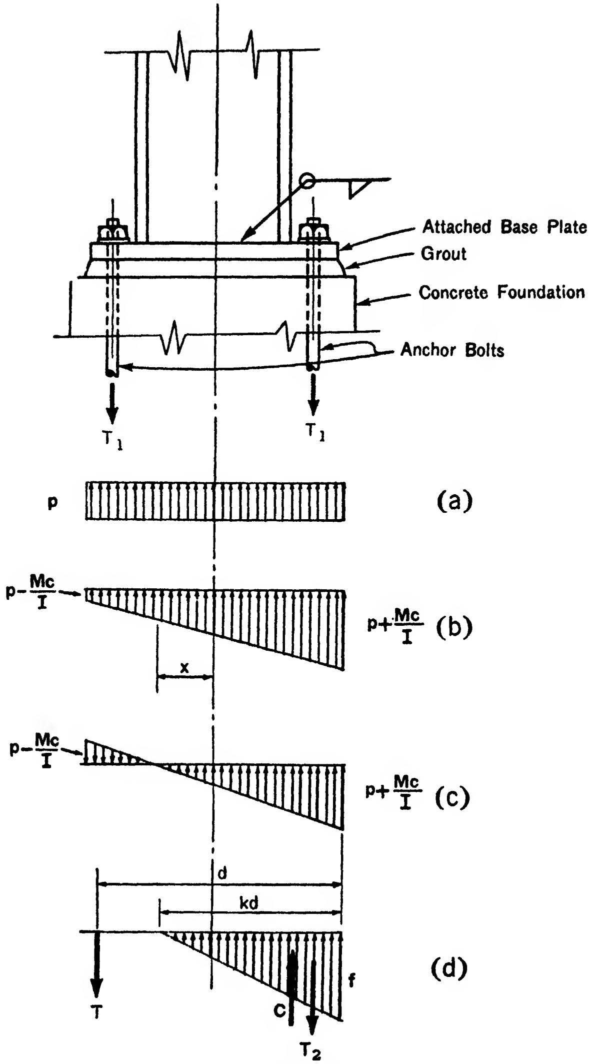


FIG. 8-3.

been reduced to T_2. Elastic shortening of the concrete on the right side caused by the increased pressure at that edge of the base plate permits the right bolt to shorten, losing some of its initial elastic strain, and therefore also losing some of its initial stress. The loss is slight, however, and safe results may be obtained if the loss is ignored.

The method of analysis to be used depends on the degree to which the moment affects the final pressure distribution. Should the moment be small, it may be possible to design a practical size of base in which anchor bolts will not be required to resist moment. A base of this type will be

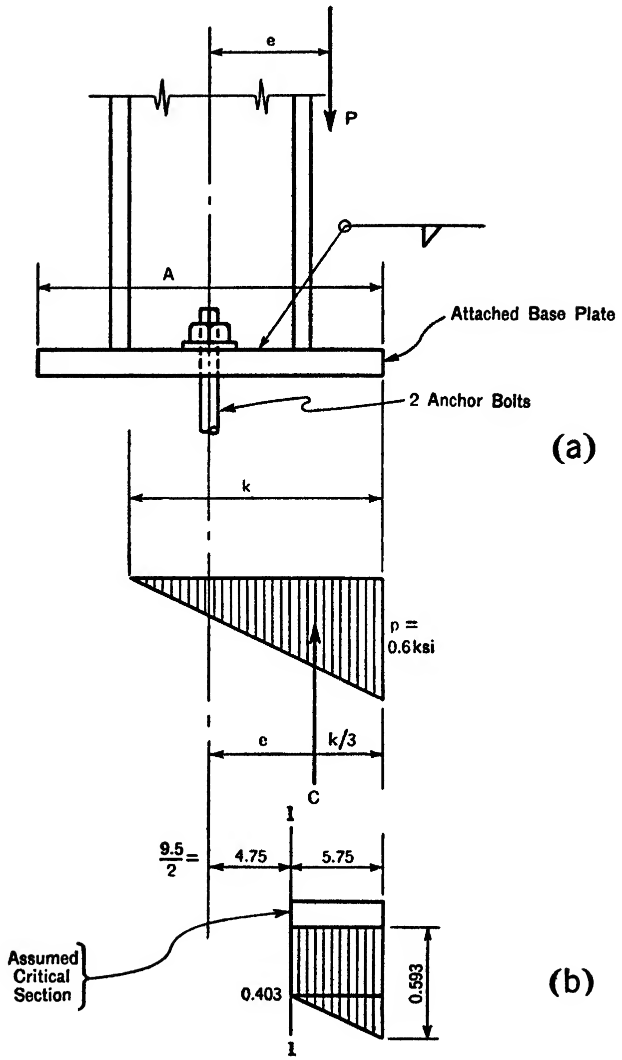


FIG. 8–4.

designed in Ex. 8–4. On the other hand, the moment might be larger so that a base designed without relying on anchor bolts would be so large as to be impractical. In such cases, the anchor-bolt tension is considered and a method of analysis similar to that for reinforced concrete members is used. This method will be illustrated by Ex. 8–5.

EXAMPLE 8–4. Design an attached base for a 10 W 49 column having an axial load of 80 kips and an end moment of 30 ft-kips, and bearing on a concrete footing. Use the AISC allowable stresses.

The moment is small for this size of column, so an attempt will be made to design a base in which the anchor bolts need not be depended upon for moment resistance.

As shown in Fig. 8–4(a), the axial load and moment are replaced by an eccentric load P at distance e from the column centerline. Base plate length A and width B are to be selected so that the maximum pressure does not exceed the allowable of 600 psi.

$$e = \frac{12 \times 30}{80} = 4.50 \text{ in.}$$

$$C = 80 \text{ kips} = \frac{1}{2}kB\,(0.6) = 0.3kB$$

Various combinations of k and B are possible. For dimension B of 15 in.,

$$k = \frac{80}{0.3 \times 15} = 17.8 \text{ in.}$$

$$\text{Dimension } A \text{ required} = 2\left(e + \frac{k}{3}\right) = 20.84 \text{ in.}$$

A plate measuring 15 x 21 in. will be tried. This is slightly larger than required, so that the pressure p at the right edge is recomputed. The location of the resultant pressure C is at distance e from the column centerline, so that

$$k = 3\left(\frac{21}{2} - e\right) = 18.0 \text{ in.}$$

$$p = \frac{2 \times 80}{15 \times 18} = 0.593 \text{ ksi}$$

The portion of plate to the right of the column flange is shown by Fig. 8–4(b). The critical section in bending is assumed to occur at section 1–1, as was shown on Fig. 8–2(c). The intensity of pressure at the critical section is 0.593 × (12.25/18) = 0.403 ksi. The bending moment per inch of plate width is

$$m = \frac{0.403(5.75)^2}{2} + \frac{0.190(5.75)^2}{3} = 8.75 \text{ in.-k.}$$

$$\text{Required } S = \frac{8.75}{20} = 0.437 \text{ in.}^3$$

$$t \text{ required} = \sqrt{6S} = 1.62 \text{ in.}$$

Use 1 Pl—15 × 1⅝ × 1–9.

The size of anchor bolts for this base cannot be computed. They would be of a size which experience shows to be suitable for holding the column in place during erection and for resisting accidental side blows during erection. Bolts smaller than ¾-in. diam are seldom used. Since the bolts are not needed for resisting the design moment, they would probably be located as shown in Fig. 8–4(a).

EXAMPLE 8–5. Design a connected base for a 10 WF 49 column bearing on a concrete footing and subject to an axial load of 50 kips and an end moment of 41.6 ft-kips. Use the AISC allowable stresses.

Four anchor bolts will be located as shown in Fig. 8–5. Their effect in resisting

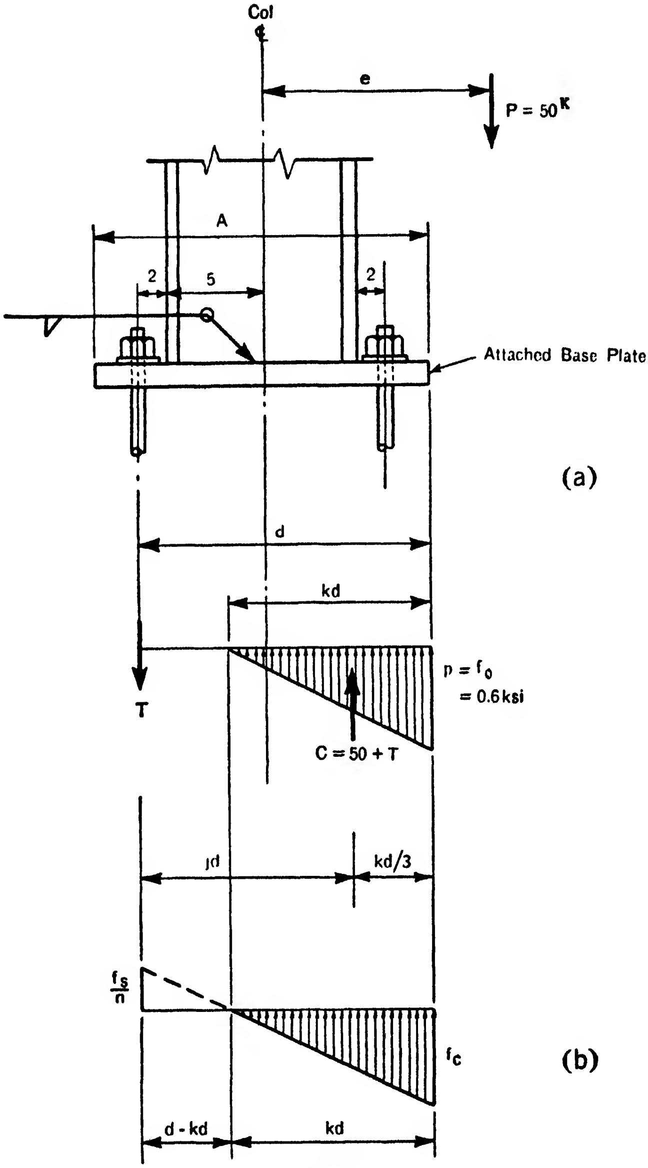


FIG. 8–5.

end moment will be considered. The anchor bolts will be assumed to have negligible initial tension.

The method of analysis that will be demonstrated is similar to that for reinforced concrete. The ratio of moduli of elasticity E_s/E_c is assumed to be $n = 10$. The dimension symbols in Fig. 8–5 are those familiar in reinforced concrete design.

The moment and axial load are replaced by an eccentric load.

$$e = \frac{12 \times 41.6}{50} = 10 \text{ in.}$$

The unknowns are the plate length A and width B, the bolt size, and the ratio k. By means of complex equations, these can be related to the applied load and moment. Such equations normally would be solved by assuming some dimensions and solving for the others. This can be done just as easily by following a few basic principles and eliminating the burden of the equations. To start the solution, plate dimensions A and B will be assumed to be 22 in. and 18 in., respectively.

$$d = \frac{A}{2} + 7 = 18 \text{ in.}$$

When $f_c = 0.6$ ksi,

$$C = \frac{1}{2} \times 0.6 \times Bkd = \frac{1}{2} \times 0.6 \times 18 \times 18k = 97.2k.$$

By summation of moments about a point on the line of force T,

$$Cjd = Cd\left(1 - \frac{k}{3}\right) = 50 \times 17,$$

$$97.2k(18)\left(1 - \frac{k}{3}\right) = 850,$$

$$1{,}750k - 583k^2 = 850.$$

Solving this equation gives $k = 0.61$,

$$C = 97.2(0.61) = 59.3 \text{ kips.}$$

By summation of vertical forces,

$$T = C - 50 = 9.3 \text{ kips.}$$

Figure 8–5(b) shows the variation of compressive and tensile stress on the "transformed" section. (See Art. 4–39.) By similar triangles, when f_c is 0.6 ksi,

$$f_s = nf_c\left(\frac{d - kd}{kd}\right) = 10 \times 0.6\left(\frac{0.39}{0.61}\right) = 3.84 \text{ ksi.}$$

Although the bolts may be capable of resisting much more stress, when the concrete stress is 0.6 ksi, the stress on the gross area of the bolts will be only 3.84 ksi. The required gross area is computed accordingly.

$$\text{Required gross area} = \frac{9.3}{3.84} = 2.42 \text{ sq in.}$$

Try two $1\frac{1}{4}$-in. bolts on each flange. The net area at the root of the threads is 0.89 sq in. per bolt. The tensile stress on the net area is

$$f = \frac{9.3}{2 \times 0.89} = 5.22 \text{ ksi.}$$

This is less than the allowable of 20 ksi; therefore the bolts selected are satisfactory.

(The required area of 2.4 sq in. is that required to keep f_c at the allowable value. Should greater bolt area be used, f_c will be less than the allowable.)

As a final step in the design, the thickness of the plate would be computed in the same manner as shown in Ex. 8–4. If the resulting complete design is uneconomical or impractical, further trials would be made using different plate sizes.

In the example above, the anchor bolts were assumed to have no initial tension. By an elastic analysis in which changes of bolt load due to elastic deformation in the concrete, as well as those due to rotation of the base plate, are considered, it can be shown that the moment resisting capacity of the base is increased by the presence of initial tension in the anchor bolts. However, the determination of the exact amount of initial tension in anchor bolts is difficult, and much dependence should not be placed on the value of initial tension. Furthermore, plastic deformation of the concrete may cause a reduction of stress to occur with passage of time. The solution shown above is recommended as safe, yet not so conservative as to be uneconomical.

8–5. Moment Resisting Bases, Plate Unattached. Figures 8–6 and 8–7 show columns with unattached base plates. The plate in this type of base does not affect the stability of the column in any way, nor does it affect the load on the anchor bolts. Rather, the plate acts somewhat as a loose "filler" between the column and the concrete.

Bases of this type may also be grouped as follows:

1. Those in which the moment is small and in which the anchor bolts are not depended upon to help resist the moment. Example 8–6 shows the design of this type.

2. Those in which the moment is larger and in which the bolts must serve to help resist the end moment in the column. The design of this type is shown by Ex. 8–7.

Figure 8–6(a) shows a base in which the bolts are so located as to be ineffective in resisting moment. The maximum moment that will be resisted by the column is that moment required to tip the column about its edge, or a moment equal to Pa, when a is one-half the column width. A safe moment value will be less than this amount because, when the column is on the point of tipping, the stress between the column and the plate would be great enough to cause bearing failure. The design limit of end moment should then be the moment that will produce the allowable bearing pressure. Care should be taken in the design of such column bases that provision is made for possible uncertainties. It is recommended that designs should provide for at least one-third of the width to be in compression, when tension bolts are not provided.

When the end moment is such that bearing between plate and column exists over the entire cross section of the column, the maximum

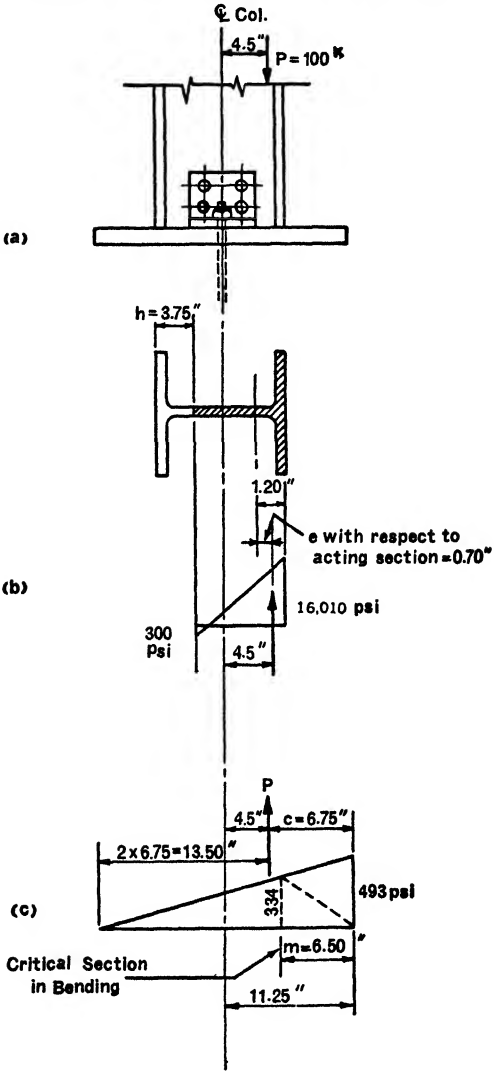


FIG. 8-6.

pressure is $p = P/A + Mc/I$. In this case the end moment would be $M = (p - P/A)\ I/c$, in which I/c is the section modulus of the cross section of the column. When positive bearing exists over only a part of the column section, only that part of the section is used in computing the section modulus.

EXAMPLE 8-6. Design an unattached base for a 10 WF 49 column having an axial load of 100 kips and an end moment of 450 in.-kips. The base rests on a concrete footing. Use the AISC allowable stresses in the steel and allow a bearing stress of 500 psi between steel and concrete.

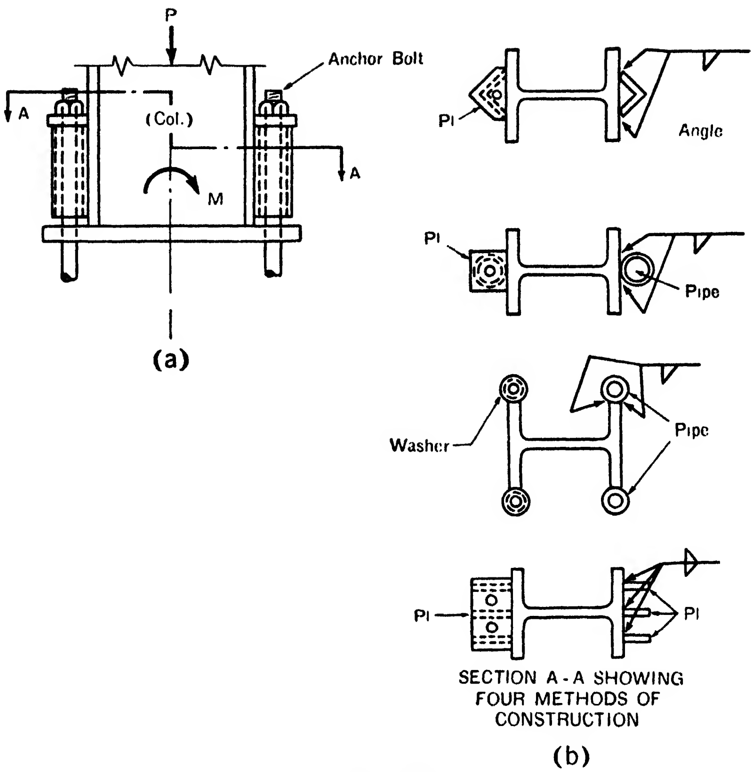


FIG. 8–7.

The moment being small, the design will be attempted without depending on the anchor bolts. The bolts will be located as shown in Fig. 8–6. In this position the bolts are ineffective in resisting moment, but the base detail is compact and easy to fabricate.

A trial calculation is made first, assuming the entire column cross section to be in bearing against the base plate. By this assumption $A = 14.4$ sq in., and $S = 54.6$ in.[3] Then,

$$\frac{P}{A} = \frac{100{,}000}{14.4} = 6{,}950 \text{ psi compression,}$$

$$\frac{M}{S} = \frac{450{,}000}{54.6} = 8{,}240 \text{ psi.}$$

The combined stresses are 1,290 psi tension at one side of the column and 15,190 psi compression at the other. Since the column is not welded to the base plate, the tensile stress does not actually exist. In the areas for which the calculations show a tensile stress, the surfaces separate slightly. The remaining area of actual contact becomes the active area.

For a second trial assume the separation to extend from the edge of the column to a distance of 2.0 in. from the same edge. [See dimension h of Fig. 8–6(b).] The properties of the assumed active section are now computed:

	A	d	Ad	Ad^2	I_o
Full section........	14.40	5.00	72.0	360	273
−Flange...........	−5.58	9.73	−54.2	−528	...
−1.44 × 0.34......	−0.49	8.72	− 4.3	− 37	...
	8.33 in.²		13.5	−205	273
				273	
				68	

$$\bar{d} = \frac{13.5}{8.33} = 1.62 \text{ in.}$$

$$-\bar{d}\Sigma(Ad) = -1.62 \times 13.50 = -22$$

$$I = 46 \text{ in.}^4$$

$$S_{\text{comp.}} = \frac{46}{1.62} = 28.4 \text{ in.}^3$$

$$S_{\text{tens.}} = \frac{46}{8.00 - 1.62} = 7.21 \text{ in.}^3$$

The eccentricity e with respect to the column center is M/P or 4.5 in. With respect to the center of gravity of the assumed active section, $e = 1.12$ in. Thus, M for the acting section is 1.12 × 100,000, or 112,000 in.-lb. Then

$$f_t = \frac{-100{,}000}{8.33} + \frac{112{,}000}{7.21} = 3{,}500 \text{ psi,}$$

$$f_c = \frac{100{,}000}{8.33} + \frac{112{,}000}{28.4} = 15{,}940 \text{ psi.}$$

The presence of a tensile stress indicates that the separation extends beyond the assumed 2.0 in. The values of f_t and f_c obtained can be used as a guide in choosing the next trial section. If the values of f_t and f_c were as computed, the neutral axis would occur at

$$8\left(\frac{15.9}{15.9 + 3.5}\right) = 6.55 \text{ in.}$$

from the compression edge, or at about 3.5 in. from the other face of the column. Apparently an error of 1.5 in. was made in the assumed distance h, but a further trial will show separation slightly beyond the 3.5 in. estimated above. Therefore, h is assumed to be 3.75 in. in the next trial.

	A	d	Ad	Ad^2	I_o
Full section........	14.40	5.00	72.0	360	273
−Flange...........	−5.58	9.73	−54.2	−528	...
−3.19 × 0.34......	−1.08	7.85	− 8.5	− 67	...
	7.74 in.²		9.3	−235	273
				273	
				38	

$$\bar{d} = \frac{9.3}{7.74} = 1.2 \text{ in.}$$

$$-\bar{d}\Sigma(Ad) = -1.2 \times 9.3 = -11$$

$$I = 27 \text{ in.}^4$$

$$S_{\text{comp.}} = \frac{27}{1.2} = 22.5 \text{ in.}^3$$

$$S_{\text{tens.}} = \frac{27}{10 - 3.75 - 1.2} = 5.3 \text{ in.}^3$$

The eccentricity e with respect to the center of gravity of the acting section is 0.7 in. The moment is 0.7 × 100,000, or 70,000 in.-lb. Thus

$$f_t = \frac{-100,000}{7.74} + \frac{70,000}{5.3} = 300 \text{ psi},$$

$$f_c = \frac{100,000}{7.74} + \frac{70,000}{22.5} = 16,010 \text{ psi}.$$

The tensile stress computed is nearly zero. Therefore, it is assumed that the acting section has been determined. Since the bearing stress of 16,010 psi at the extreme fiber does not exceed the allowable of 30,000 psi for milled parts, the column base detail as shown is satisfactory.

A base plate may be designed on the basis of 500 psi allowable bearing pressure on the masonry and a load of 100,000 lb eccentric 4.5 in. from the center of the column. It is assumed that the eccentricity of the load may be in either direction from the center of the column; hence, the plate will be symmetrical.

Referring to Fig. 8–6(c), if the width of the plate parallel to the flanges of the column is B, then, on the assumption that the pressure diagram is triangular, the maximum masonry pressure will be

$$500 \text{ psi} = \frac{100,000 \times 2}{B \times 3c}.$$

When B is 20 in.,

$$c = \frac{100,000 \times 2}{20 \times 500 \times 3} = 6.67 \text{ in.}$$

The location of the applied force P is known to be 4.5 in. from the centerline. The edge of the base plate must then be at least 4.5 + 6.67 = 11.17 in. from the centerline. Hence, a plate 20 x 22.5 is assumed. The distance from the resultant force to the edge of the plate is 11.25 − 4.5 = 6.75 in. The width of plate in contact with the masonry will be 3 × 6.75 = 20.25 in. For symmetry, a plate width of 2 × 11.25 = 22.5 in. shall be used, even though 2.25 in. of width near the left edge will be ineffective under this particular loading of the column. The maximum pressure

$$p = \frac{100,000 \times 2}{20 \times 20.25} = 493 \text{ psi}.$$

The thickness of the plate will be controlled by the bending moment along line 1–1 [Fig. 8–2(c)]. The distance m is 6.50 in., and the pressure diagram on the cantilevered strip is trapezoidal. The moment of the upward pressure on the strip about line 1–1

$$334 \times \frac{6.50}{2} \times \frac{6.50}{3} = 2,350$$

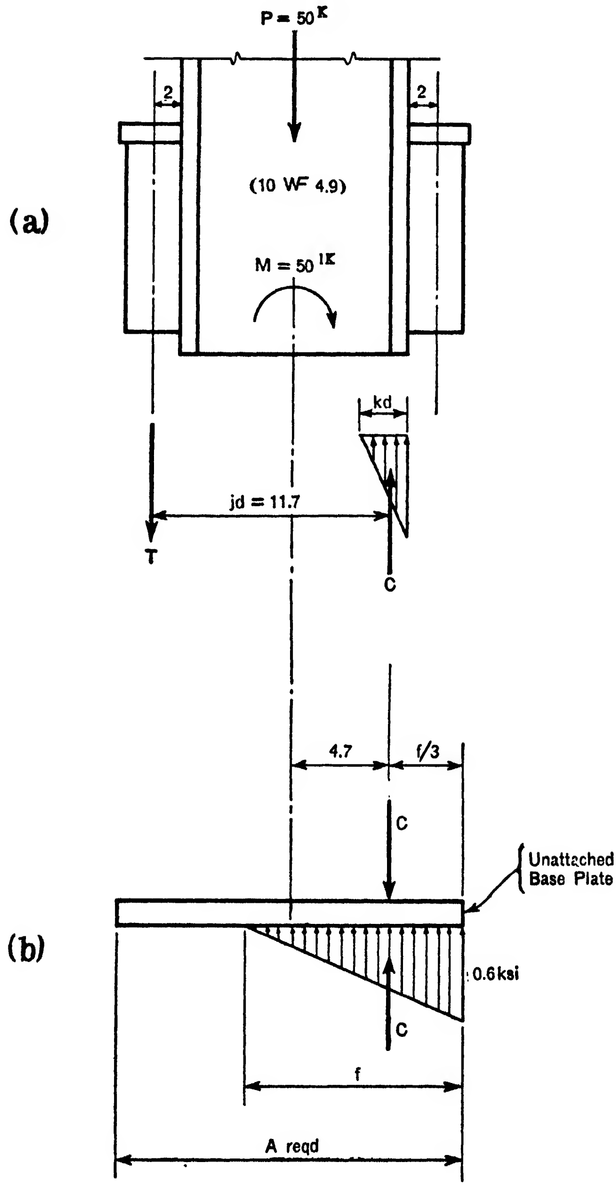
$$493 \times \frac{6.50}{2} \times \frac{2 \times 6.50}{3} = \underline{6,950}$$

$$9,300 \text{ in.-lb (per inch width of plate).}$$

The required section modulus is then 9,300/20,000, or 0.465 in^3., and the required thickness,

$$t = \sqrt{6 \times 0.465} = 1.67 \text{ in.}$$

A base plate measuring 20 x 1¾ x 1–10½ will be used.



P = 50 K
2
2
(10 WF 4.9)
(a)
M = 50 1K
kd
jd = 11.7
T
C
4.7
f/3
C
Unattached
Base Plate
(b)
0.6ksi
C
f
A reqd

FIG. 8-8.

EXAMPLE 8–7. Redesign the base for the column of Ex. 8–5, using an unattached base plate.

Figure 8–7(a) shows the type of base that will be used. The anchor bolts are inserted through brackets welded to the column. Typical methods of constructing the brackets are shown by Fig. 8–7(b).

Assume that the anchor bolts will be located 2 in. from the column face. The forces holding the column in equilibrium will then be as shown by Fig. 8–8(a). Initial tension is assumed to be neglegible, so that force T must be sufficient only to resist tipping of the column about its right edge. The resultant upward force C is of unknown location because kd is not known. However, the majority of the effective area of contact between the column and the plate is the area of the flange, so that it is reasonable to assume that force C acts as the center of the flange thickness. Thus, the lever arm of force T is

$$jd = 10 + 2 - \frac{0.558}{2} \doteq 11.7 \text{ in.}$$

Equating moments about C to zero gives

$$11.7T - (4.7 \times 50) + (12 \times 50) = 0, \text{ and}$$

$$T = \frac{365}{11.7} = 31.2 \text{ kips.}$$

Force C must oppose all downward forces; then

$$C = P + T = 81.2 \text{ kips.}$$

An exact analysis of the intensity of bearing pressure between the column and the base plate is very complicated. It is commonly regarded as sufficiently accurate to determine whether the right flange has enough area to develop force C. The allowable bearing stress for milled surfaces is 30,000 psi. At this stress, the flange capacity is

$$30 \times 10 \times 0.558 = 167 \text{ kips,}$$

which is adequate. (The bearing stress is, of course, nonuniform. Should overstress occur at the extreme outer edge, slight yielding will occur, reducing the nonuniformity of pressure. Failure would not follow until the entire flange area had yielded. This cannot occur until force C is much in excess of 167 kips.)

The bolts are selected next.

$$\text{Net area required} = \frac{31.2}{20} = 1.56 \text{ sq in.}$$

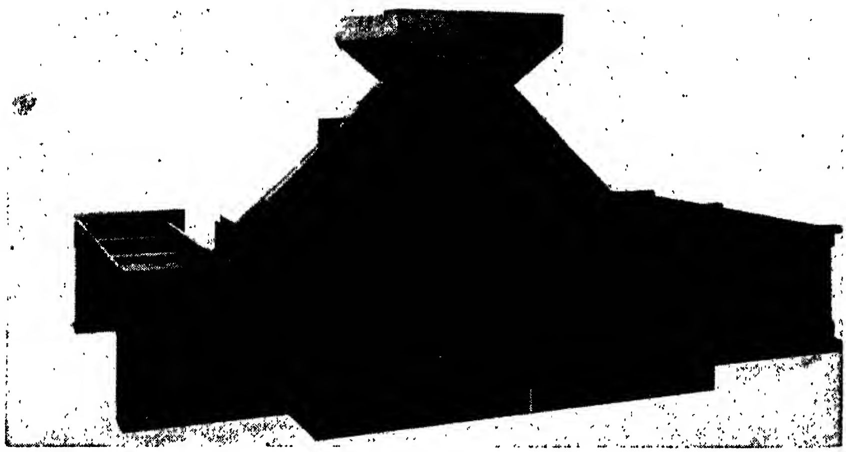
This can be provided by one 1¾-in. bolt or by two 1¼-in. bolts on each side of the column. A glance at tables of nut dimensions shows that either choice is satisfactory as to clearances. The larger size may cause difficulty in the foundation design, however, as its imbedded length must develop the tensile strength of the bolt through bond. Four 1¼-in. bolts, two on each flange, should therefore be used here.

The plate is designed next. Figure 8–8(b) shows a free-body diagram for the plate. In any case, the resultant of the upward pressure is equal to and collinear with the downward force C from the column above. Allowing the maximum concrete stress to be 0.6 ksi, and calling the width of the plate B,

$$C = fB\left(\frac{0.6}{2}\right) = 81.2 \text{ kips.}$$

If B is chosen as 16 in., $f = 16.9$ in., and the required dimension A is $2[4.7 + (f/3)] = 20.7$ in. A 16 x 21-in. plate would be satisfactory. The thickness would be computed in the same manner as for Ex. 8–6.

8–6. Fixed Bearings for Girders or Trusses. Flat plate end bearings of the type illustrated by Fig. 8–1(a) are not suitable for long spans in which live load produces a large part of the reaction. Deflection of the span causes bearing pressures to become excessive and frequently results in serious damage to the masonry. As a result, bearings that provide for the possibility of deflection of the span structure without damage to the bearing are a necessity for many structures. Some type of pin or "line bearing" detail is usually employed in these bearings.



American Bridge Co.

CAST AND FORGED STEEL FIXED BEARING SHOE FOR DELAWARE RIVER TURNPIKE BRIDGE.

The bearings are of two types: fixed and expansion. Several examples of fixed bearings are shown in Fig. 8–9. The term *fixed,* as used in the discussion of the "fixed bearing," means only that the position of the bearing is fixed and not that the slope to the elastic curve is fixed, as defined in the study of statically indeterminate structures.

The design of a fixed bearing, such as shown by Fig. 8–9(a), may be considered in steps, as follows:

1. Selection of a pin size with respect to the strength of the pin in shear and bending and the strength of the vertical plates in bearing against the pin.
2. Determination of the thickness of the vertical plates. This step is usually

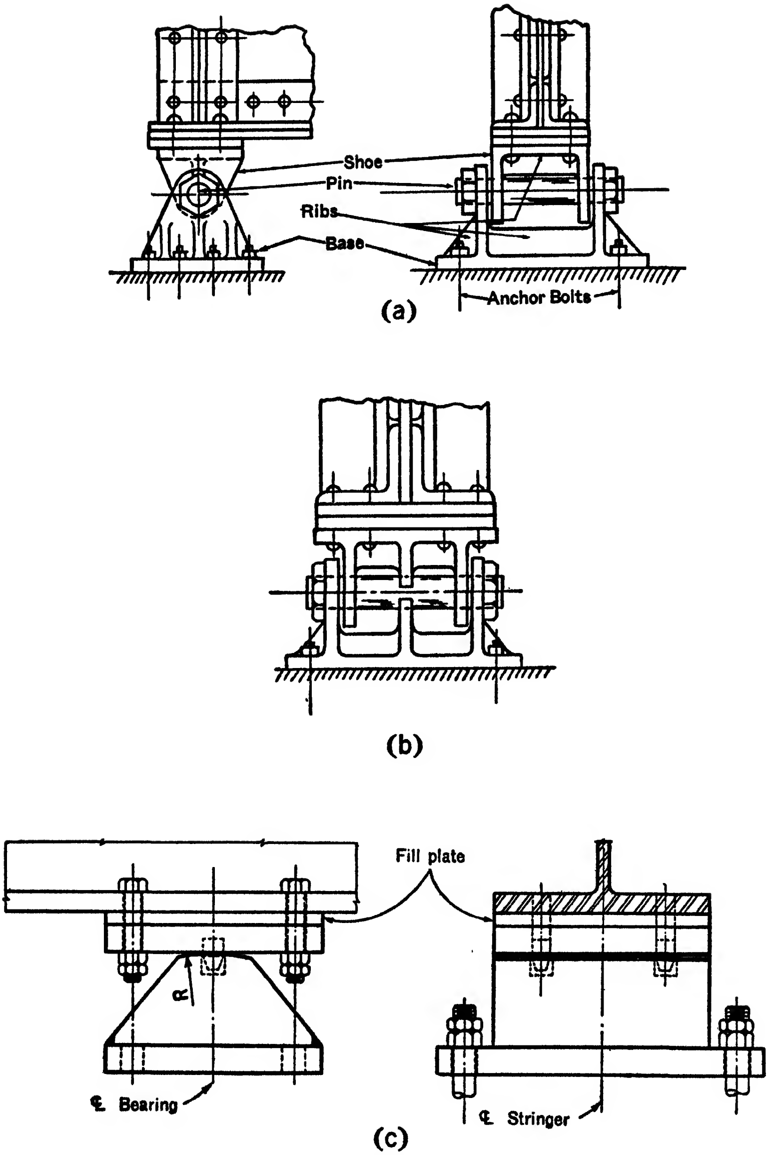

Shoe
Pin
Ribs
Base
Anchor Bolts
(a)
(b)
Fill plate
R
℄ Bearing
℄ Stringer
(c)

FIG. 8-9.

carried out simultaneously with the pin selection, since the plate thickness affects the bending moment on the pin.

3. Determination of the base size required to prevent excessive bearing stress on the supporting masonry.
4. Selection of a base plate of sufficient thickness to resist bending. The procedure is similar to that used for beam and column base plates previously discussed.
5. Selection of anchor bolts. These should hold the base firmly against the foundation so that the friction developed will prevent shifting under lateral or longitudinal load. They should also resist any expected uplift or negative reaction.
6. Location of ribs and diaphragms to reinforce the vertical plates and the base plate. The selection of a base plate in step four may be affected by the location of such diaphragms.

Usually, the steps just given are first performed considering the vertical load only. The bearings may also be subjected to lateral and longitudinal loads, and a complete design should include consideration of these loads as well as the vertical. Most specifications, however, permit an increase of 25 to 33 per cent in all allowable unit stresses when vertical load is combined with lateral or longitudinal loads. Since the vertical load on the bearing is usually many times the other loads, a bearing designed for vertical loads alone is usually satisfactory for combined loadings. A suggested procedure is to design for vertical loads at the given allowable unit stresses; then check the bearing so designed for the effect of simultaneously applied lateral and longitudinal loads, using the increased allowable unit stresses.

The shoe and pedestal may be of cast steel or of welded steel plate. The choice between the two depends on economy and on whether the applicable specification permits welding.

The bearings in parts (*a*) and (*b*) of Fig. 8–9 have vertical plates which completely surround the pin so that resistance to uplift is provided. The center plate of Fig. 8–9(*b*), however, does not completely encircle the pin but contacts it on a surface which is slightly less than a half-cylinder. For a positive reaction, both the side plates and the center plates are effective; for a negative reaction, only the side plates are effective. The plates of Fig. 8–9(*b*) may be assumed to be loaded in proportion to their bearing areas on the pin.

Some specifications require that resistance to uplift be provided even though the design computations show that no uplift is expected. For such cases, details such as in Fig. 8–9(*a*) or (*b*) will be needed. If there is no requirement for resistance to uplift, a detail such as shown in Fig. 8–9(*c*) may be used, in which the bearing is on a curved surface. The radius R for the bearing surface is dependent on the load and the length of the bearing. Pintles are used to keep the bearing in proper alignment.

Example 8–8. Design a fixed bearing similar to the type shown in Fig. 8–9(a) for the support of a two-track, through-truss railway bridge of 160-ft span. The bridge rests on concrete abutments.

Use the AREA specification. The maximum reactions on one bearing are: vertical, 1,200 kips; longitudinal, 119 kips; and lateral, 10 kips due to nosing plus 36 kips due to wind.

The end gusset plates and attached reinforcing plates are used in place of an upper shoe. The gusset plate thickness is 1 in. per plate, and the plates are spaced 18 in. center-to-center.

In Fig. 8–10(a) are shown the vertical loads acting on the pin. The pin is assumed to act as a simple beam supported at its ends by the vertical plates of the shoe and loaded between by the gusset plates and any attached reinforcing plates. The span is unknown until the plate thickness t is determined; therefore, selection of the pin size will be by successive approximation.

The allowable unit stresses for the pin are as follows: shear, 13,500 psi; bearing on structural steel or cast steel, 24,000 psi; flexural stress, 27,000 psi.

The required area for shear resistance is

$$\frac{600}{13.5}, \text{ or } 44.5 \text{ sq in.}$$

A pin with a 7¾-in. or more diam has sufficient cross-sectional area. Assuming a 7¾-in. pin,

$$t \text{ required for bearing} = \frac{600}{24 \times 7.75} = 3.23 \text{ in.,} \qquad \textit{Use } 3\tfrac{1}{4} \text{ in.}$$

$$M = 600\ (3\tfrac{1}{4} + \tfrac{1}{8}) = 2{,}025 \text{ in.-kips.}$$

The resisting moment of a solid pin,

$$M_r = \frac{fI}{r} = \frac{f\pi d^3}{32}.$$

For M of 2,025 in.-kips,

$$d \text{ required} = \sqrt[3]{\frac{32 \times 2{,}025}{27\pi}} = 9.15 \text{ in.}$$

Therefore, a 7¾-in. pin is not satisfactory.

If the pin size is increased, t and m decrease. Therefore, the next pin size investigated will be slightly less than 9.15 in. in diam. For a pin with a 9-in. diam, by computations like those made for the first assumed size,

$$t \text{ required} = 2.78 \text{ in.,} \qquad \textit{Use } 2\tfrac{7}{8} \text{ in.}$$

$$m = 1{,}800 \text{ in.-kips,}$$

$$d \text{ required for flexure} = 8.79 \text{ in.}$$

A 9-in. pin is satisfactory. Reinforcing plates will be added, as shown by Fig. 8–10(c), to make a total thickness of 2⅞ in. in bearing at each gusset plate.

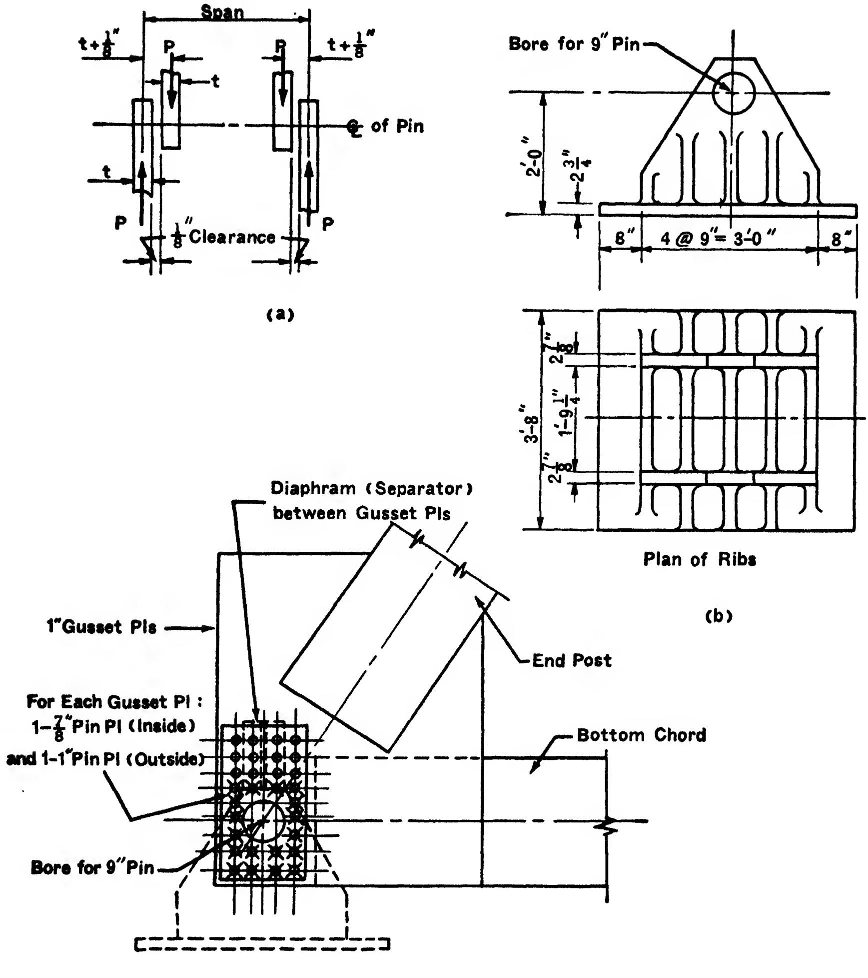


FIG. 8-10.

The bearing area required to prevent crushing of the masonry will be determined for vertical loads alone and checked for the effect of simultaneous vertical and horizontal loads. The allowable bearing pressure of steel on concrete is given by the specification as 600 psi.

The area required = 1,200/0.6 = 2,000 sq in.

Sufficient area can be provided by a 45-in. square base.

The height of the pin above the base depends on many detail requirements peculiar to a particular structure and not necessarily controlled by strength. Assuming that a practical height for this case is 24 in., the maximum masonry bearing pressure may be computed. If the entire base is in compression,

$$f_{max} = \frac{P}{A} + \frac{Mc}{I} + \frac{M'c'}{I'},$$

in which M, moment of the longitudinal load about the base, $= 24 \times 119 = 2{,}856$ in.-kips; M', moment of the lateral load about the base, $= 24 \times 46 = 1.104$ in.-kips; and I/c and I'/c' are values of section modulus of the base about the transverse and the longitudinal centerlines, respectively.

Solving for the maximum bearing pressure,

$$f_{max} = \frac{1{,}200}{2{,}025} + \frac{2{,}856}{15{,}200} + \frac{1{,}104}{15{,}200}$$

$$= 0.592 + 0.188 + 0.073 = 0.853 \text{ kips per sq in.}$$

$$f_{min} = 0.592 - 0.188 - 0.073 = 0.331 \text{ kips per sq in.}$$

Compression exists over entire area.

The AREA specification allows stresses caused by combinations of load, including longitudinal and lateral loads due to nosing or wind, to be 25 per cent greater than those permitted for other combinations of loading. The allowable bearing pressure for the loads being investigated is therefore 1.25×600, or 750 psi. Since the maximum pressure computed above exceeds this allowable, a different base size must be used.

Similar computations for a base 52 in. long by 44 in. wide show the maximum bearing stress to be 733 psi and the minimum 313 psi. A 52 x 44 in. base size is satisfactory.

The base thickness must be sufficient to resist bending. Transverse ribs are located so as to divide the base into narrow strips having small bending moments. For the rib arrangement shown in Fig. 8–10(*b*), the largest bending moment in the base occurs in the cantilevered edge strip. The critical section may reasonably be assumed to be at the face of the rib. (For other arrangements of ribs, an inside section of the base might be more highly stressed in bending than the cantilevered edge strip.)

Since the pressure resulting from combined vertical, lateral, and longitudinal loads exceeds that resulting from vertical loads alone by more than 25 per cent, it will be used in determining the required base thickness. The allowable unit stress for cast steel in flexure is three-quarters of that for rolled structural steel, and it is increased by 25 per cent when the loads include lateral and longitudinal loads.

A conservatively high value of the bending moment at the critical section can be computed by assuming a uniform pressure of 733 psi over the entire cantilevered strip. (If greater accuracy is desired, account of the pressure variation may be taken, as was done in Ex. 8–4.) Thus,

$$M = \frac{733 \times 8^2}{2} = 23{,}500 \text{ in.-lb per strip of 1-in. width,}$$

$$t \text{ required} = \sqrt{\frac{6 \times 23{,}500}{0.75 \times 18{,}000 \times 1.25}} = 2.89 \text{ in.}$$

The pressure used is conservatively high; therefore, a thickness of $2\frac{3}{4}$ in. may be considered satisfactory.

Anchor bolts are required by the specification to be at least $1\frac{1}{4}$ in. in diam.

Since no uplift load is to be considered in this bearing, the selection of size and number of anchor bolts is based on the specification requirements and detail dimension requirements of the base. The lateral and longitudinal loads do not occur without simultaneous vertical loads; thus, friction may be depended on to transfer horizontal load to the footing. The anchor bolts, through their initial tension, aid in increasing the frictional resistance.

By using arrangements of ribs other than that used in this example, it is possible to design similar bases, some of which may be more economical than the one illustrated. A complete design should include a comparison, for economy, of several designs.

8-7. Expansion Bearings. Expansion bearings serve four purposes, as follows:

1. To permit free thermal expansion of the structure.
2. To permit free movement of the end of the structure as a result of length change caused by live load.
3. To permit shifting of the abutment without causing additional stress in the structure.
4. To prevent the transfer of longitudinal loads to certain parts of the substructure.

Several types of expansion bearing used for bridges are illustrated in Fig. 8-11. For short spans, the simple sliding bearing shown in Fig. 8-11(*a*) is used. The sole plate is connected to the beam by welds or bolts. During movement the sole plate slides against the base detail. The radius of curvature for the bearing surface is computed for the load and the length of bearing. For spans of greater length than about 60 or 80 ft, as limited by the specifications, one of the other types of bearing illustrated is used.

Figure 8-11(*b*) shows a rocker type of bearing. The parts of this bearing are usually made of cast steel or machined from a structural steel billet. Longitudinal motion is permitted as the bearing rolls on the curved surfaces of the rocker.

Figure 8-11(*c*) shows a segmental roller bearing. The pintles connecting the rollers to the base and cap plates ensure a constant and predetermined spacing between them. Smaller sizes and single rollers are sometimes complete circles; if rollers of large diameter are used, their sides may be trimmed, as shown in Fig. 8-11(*d*); the remaining portions (shaded) being placed closer together to give a more economical bearing. This segmental roller bearing has the advantage of higher allowable bearing stress resulting from the use of large diameter rollers and the advantage of more compactness than is possible with cylindrical rollers.

Resistance to lateral forces, and creeping of the roller nest may be provided by grooving the rollers and by providing one roller with teeth engaging the upper and lower bearing details as shown in Fig. 8-11(*e*). Two side

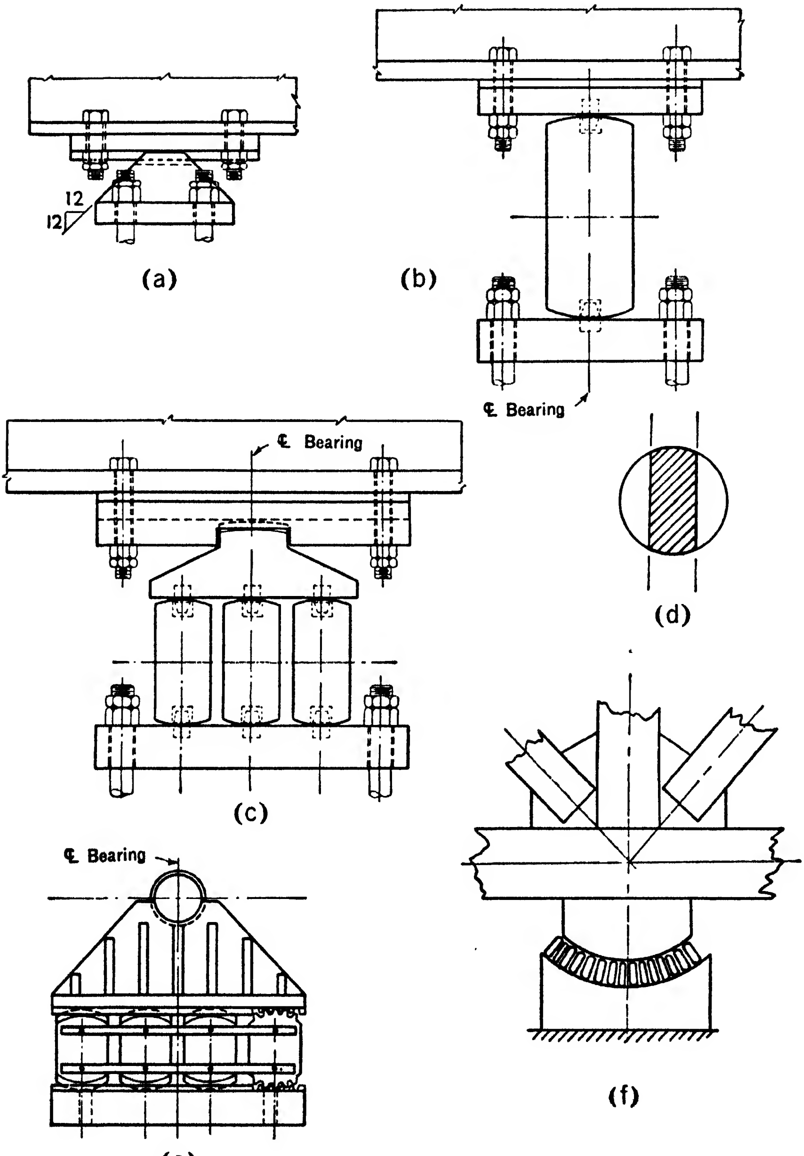


FIG. 8-11.

bars are required at each end of the segmental rollers in this detail to keep them parallel to each other and properly spaced.

The segmental roller may be used for a fixed bearing by making the base curved, as shown by Fig. 8–11(*f*). Such a detail might be useful where the pin sizes or plate thickness of details, such as those of Fig. 8–9, are impractical because of large reactions. The well-known cantilever railway bridge over the Ohio River at Beaver, Pennsylvania, has this type of bearing at the tower bases.

8–8. Design of Expansion Bearings. The first consideration in the design of any expansion bearing is the amount of movement required. For short spans, the movement required for elastic deformation is negligible. For larger trussed structures it may be determined by computation of the change of length of the bottom chord due to stress.

The allowance for thermal expansion is often limited by the specification. For example, the AREA specification requires provisions for a change of span length, resulting from temperature change, of 1 in. per 100 ft of span.

The 1946 *Specifications for Design of Highway Structures*, of the Ohio State Highway Department, controls in greater detail the provision for thermal expansion, as follows:

86. . . . . For steel beam structures (of not more than 150 ft over-all length) with timber substructures, no expansion devices need be provided; for bridges of this type with over-all length greater than 150 ft, provision shall be made for a change of length (in each direction) of ⅛ in. for each 30 ft of length.

87. For other superstructures which are free to become longer or shorter with changes of temperature, such as the usual statically determinate slab, beam, girder, or truss which is partially supported on rockers, rollers, sliding plates, flexible columns, or flexible piers, of either single or multiple span, provision shall be made for expansion and contraction due to the following temperature variations, in degrees Fahrenheit:

	Rise	*Fall*
Reinforced concrete superstructure	40	75
Structural steel with concrete floor	45	85
Structural steel with metal floor	50	95

Elsewhere in the Ohio specification is the requirement that

. . . to allow for slight tilting or sliding of masonry abutments (except on rock or shale) after the backfill has been placed, there shall be provided, between superstructure and abutment, an opening (in addition to that required for elongation of superstructure) of at least $\frac{1}{32}$ in. and preferably $\frac{1}{16}$ in. for each foot of height (measured from bottom of footing to roadway surface) for each abutment.

Before deformation of the bed or base plate and of the roller or rocker, there is only a line of contact between the plate and the rocker. The allowable stress in bearing between the roller and plate is given in pounds per linear inch of contact.

Experiments at the University of Illinois show that the allowable bearing stress is affected by the roller diameter and by the tensile yield points of the roller and plate materials. The AREA specification considers both of these variables in its formulas. Most other specifications show the diameter as the only variable, although the constant of any one formula includes the effect of the yield point for the materials covered by that specification.

Using the allowable bearing values, satisfactory combinations of diameter and length of rollers may be determined. From these combinations, a choice between rollers and rockers may be made.

In a rocker or segmental-roller design, the next step is to determine the dimensions necessary to allow the required movement. For a rocker of known radius, the degrees of arc for the upper and lower surfaces to permit both full expansion and full contraction must be computed. For a segmental roller, the width of the segments must provide enough arc length on the upper and lower surfaces to permit both full expansion and full contraction. The spacing between the segments must be enough so that clearance between the segments exists even in the extreme expanded or contracted positions. Since no factor of safety exists in the specified amounts of movement to be provided for, it is the practice to provide a capacity for some movement beyond the specified amount. For example, a segmental roller is not designed so that under the full specified movement the segment is in bearing along a line at the edge of the curved surface.

EXAMPLE 8-9. Select a possible size and length of segmental rollers in an expansion bearing for the truss of Ex. 8-8.

The bearing stress allowed by the AREA specification is given, for rollers up to 25-in. diam, by the expression

$$\frac{p - 13{,}000}{20{,}000} \times 600d \text{ (pounds per linear inch),}$$

in which d = roller diameter in inches, and p = tensile yield point of the steel in the roller or in the base, whichever is smaller.

Elsewhere in the same specification is the requirement that forged steel be used for the rollers and that it must have a yield point of not less than 33,000 psi. Substituting 33,000 for p, the expression reduces to

$$f_{\text{brg}} \text{ allowable} = 600d,$$

which is the equation given by the AISC and other specifications.

In the selection of a roller size and length, a practical diameter is assumed, the allowable bearing stress per inch computed, and the required length for that size bearing computed. The process is repeated until a satisfactory or economical combination of diameter and length is determined.

Assuming a 6-in. roller diam,

$$f_{brg} = 600 \times 6 = 3{,}600 \text{ lb per in.},$$

$$L \text{ required} = 1{,}200 \div 3.6 = 333 \text{ in.}$$

Ten rollers, 6 in. in diam by 33 in. long, would provide the necessary bearing strength. Similarly computed, ten rollers of 8-in. diam and 25-in. length, or eight rollers of 10-in. diam and 25-in. length, would satisfy the strength requirements.

Design of the shoe and base castings or assemblies is similar to that of Ex. 8–8, although it is not affected by longitudinal loads.

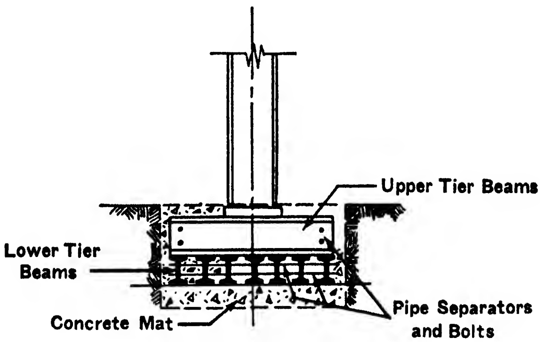


FIG. 8–12.

8–9. Grillage Footings. Figure 8–12 shows a typical grillage footing. The grillage consists of two or more tiers of beams arranged so as to spread a concentrated load over a large area, thus preventing excessive bearing pressures against the supporting soil. A permanent grillage is usually filled with and encased in concrete. At one time grillage footings were extensively used. Now, however, footings of reinforced concrete are made, which are equally satisfactory and less expensive. Although many grillage footings still exist, new ones are seldom constructed except for under pinnings or for temporary footings.

In the design of a grillage footing, four types of possible failure are considered. They are the failure of the soil by bearing pressure, and shear, crippling, and flexural failures of the beams. Vertical buckling of the web is not considered, since the web is laterally supported by the encasing concrete and by the separators and bolts which space the beams. Because the beams are of short length, shear often controls the selection of the section.

In design, the soil pressure is considered to be distributed uniformly over the base. The effective base area is assumed to extend beyond the edges of the beams for a distance equal to about one-half of the flange width, provided the concrete mat extends that far. Thus, in the average case, the entire area of the concrete mat may be used as the effective base area. In computing the bearing pressure on the soil, the entire load (column load plus the footing weight) is considered. In computing the shear and bending moment in the beams, it is assumed that the pressure against the beam is distributed uniformly along the length of the beam. The weight of the encasing concrete does not load the beams; therefore, the column load only is used in designing the beams. (For greater accuracy, the weight of the upper tier might be added to the load used in designing the lower tier beams.)

Detail requirements include a spacing between the beam flanges sufficient to permit the placing and tamping or vibrating of the concrete. A 2-in. space is usually sufficient, although this varies with the size of aggregate to be used. To allow bond of the concrete to the steel, the beams are left unpainted. On beams of not more than 8 in. in depth, a single row of pipe spacers and bolts is used. On deeper beams two rows are used. The spacers are placed at intervals of 5 ft or less.

EXAMPLE 8–10. Design a grillage footing to support a column load of 400 kips on a soil having an allowable bearing pressure of 4,000 lb per sq ft. The column has a base plate measuring 24 in. square. Use the AISC specification.

The weight of the footing will be assumed at 30 kips, making the total load 430 kips. The soil bearing area required = 430/4 = 107.5 sq ft.

Try a footing size of 8–0 x 13–6. A footing having these dimensions is shown by Fig. 8–13. Assuming an encasement of concrete of 4-in. thickness, the lengths of the beams are 7 ft 4 in. for the top tier and 12 ft 10 in. for the lower tier. The upper tier beams will be selected first.

The loads on the upper tier are shown in Fig. 8–13(*b*). From these loads the shear and moment values are computed. Fig. 8–13(*c*) and (*d*) show, respectively, the shear and bending moment diagrams.

S required for the entire tier = $(268 \times 12{,}000)/20{,}000 = 161$ in.^{3}

The flexural strength needed can be provided by three beams, 16 WF 36 ($S = 3 \times 56.3$). Checking these beams for shear and web crippling,

$$v = \frac{146{,}000}{3 \times 15.85 \times 0.299} = 10{,}270 \text{ psi},$$

$$f_{\text{brg}} = \frac{146{,}000}{3 \times 0.299\ (24 + 1.88)} = 6{,}270 \text{ psi}.$$

Since neither of the above stresses exceeds the allowable values, the use of three 16 WF 36 sections is satisfactory. As the flange width of one of these beams is 7 in., it will be necessary to extend the flange of the outer beams ½ in. beyond the edge of the column base plate in order to maintain a clear spacing of 2 in. between the flanges.

By similar computations, the values obtained for the lower tier are

$$V = 169 \text{ kips};\quad M = 542 \text{ ft-kips};\quad \text{required } S = 325 \text{ in.}^3$$

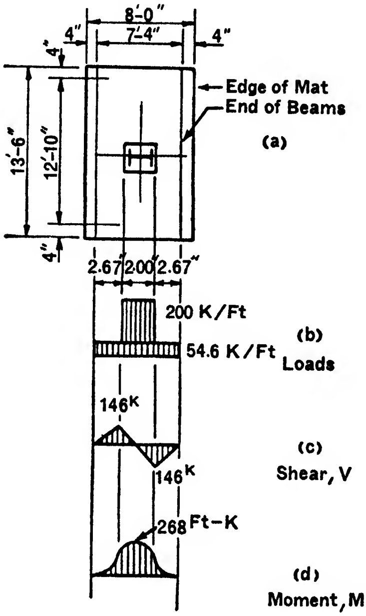


FIG. 8-13.

Ten 12 WF 27 sections will satisfy the flexural requirements. For these sections,

$$v = \frac{169{,}000}{10 \times 11.95 \times 0.24} = 5{,}900 \text{ psi},$$

which is less than the allowable value.

Web crippling is checked by using the width of the flange of an upper tier beam as the length in bearing against the lower tier beams. Since the flange of the upper beam is not rigid, advantage is not taken of the k distance when checking the lower sections. Thus computed,

$$f_{\text{brg}} = \frac{400{,}000}{6.99 \times 30 \times 0.24} = 7{,}950 \text{ psi},$$

which is less than the allowable of 24,000 psi. (The number 30 is the number of intersections of upper and lower beams.)

The footing will consist of an upper tier of three 16 WF 36 beams and a lower tier of ten 12 WF 27 beams. The entire footing will rest on a concrete mat from 6 to 9 in. thick, according to the specification requirements. If used for a permanent footing, the grillage will be encased in concrete.

PROBLEMS

8-1. Design bearing plates for beams as follows:

(a) For a 16 WF 36 with an end reaction of 25 kips bearing on a brick wall. Use the AISC specification.

(b) For a 30 WF 116 with an end reaction of 100 kips bearing on a concrete abutment. Use the AREA specification.

8–2. Select base plates for columns as follows:

(a) For an 8 WF 31 having an axial load of 92 kips and supported by a concrete footing. Use the AISC specification.

(b) Same as above, but for a 14 WF 87 having an axial load of 400 kips.

8–3. Design column base details as follows:

(a) For a 14 WF 87 supported by a concrete footing and having an axial load of 130 kips and an end moment of 400 in.-kips. Use the AISC specification.

(b) Same, but having an axial load of 130 kips and an end moment of 750 in.-kips.

(c) Same, but having an axial load of 130 kips and an end moment of 1,400 in.-kips.

8–4. Design a bearing of the type shown by Fig. 8–9(*a*) for a railway plate girder having an end reaction of 327 kips. Assume the bearing to be made of cast steel and resting on a concrete abutment. Use the AREA specification.

8–5. Same as Prob. 8–4, but of the type shown by Fig. 8–9(*c*).

8–6. Design an expansion bearing of the type shown by Fig. 8–11(*b*) for the plate girder of Prob. 8–4.

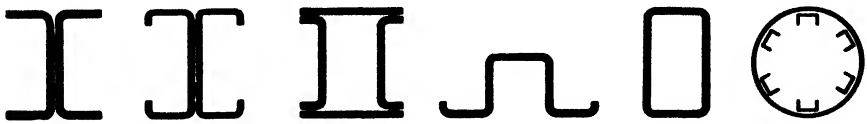
8–7. Select segmental rollers for an expansion bearing of the type shown by Fig. 8–11(*e*) for the end support of a 220-ft railway bridge truss having an end reaction of 1,182 kips. Determine also the required spacing between the segments. Use the AREA specification.

8–8. Design completely an expansion bearing of suitable type for the highway bridge truss designed in Chap. 7. (See Arts. 7–6 to 7–9.) Use an allowable bearing stress of 600 psi on the concrete.

CHAPTER 9

LIGHT GAGE STRUCTURAL MEMBERS

9–1. Use of Light Gage Metal for Structural Members. The use of light gage members dates back at least to 1855, during which year the building for the Bank of the State of New York was constructed. That building contained members which were cold formed from $\frac{1}{16}$ in. and $\frac{1}{8}$ in. thick steel sheets. Since that time the use of light gage metal for particular



(a) (b) (c) (d) (e) (f)

FIG. 9–1.

parts of structures has become commonplace. For example, steel roof and floor decks and formed steel roof and floor joists have been extensively used. Corrugated light gage metal has long been used for culvert construction. During and since World War II, light gage construction has been used for various types of buildings, from large hangars to small Quonset huts.

Much use of light gage members is made in aircraft. The need for greater strength with less weight (even though at slightly higher cost) has promoted their use.

Some typical cross sections of light gage members are shown in Fig. 9–1. Members (*a*), (*b*), (*c*), and (*d*) are made by rolling or braking flat strip steel to the desired cross section. In members such as (*a*), (*b*), and (*c*) the parts are assembled by riveting or spot welding to form a single member. Member (*e*) can be drawn or can be folded and welded. The section of part (*f*), which is common in aircraft, consists of a curved sheet or "skin" attached to longitudinal stiffeners or longerons, which in turn are formed of light gage metal or of rolled or extruded shapes.

Although light gage members have been used for many years, it was not until 1946 that a general specification for their design became available. At that time the first edition of the *Specification for the Design of Light Gage Steel Structural Members* was published by the American Iron and Steel Institute. This specification (hereafter called the "AISI specification") was based mainly on the results of experimental work directed by Dr. George Winter at Cornell University. Excerpts from a revised edition of the specification are given in Appendix F.

What distinguishes the light gage member from the more usual type? Because light gage members tend to fail in local buckling, it would seem correct to classify a member according to its probability of local buckling. The probability could be measured by the ratio of transverse width to thickness for each flat element of the member. Before the member could be classified and the applicable specification determined, it would be necessary in many cases to check the section for over-all buckling or flexure, and then for local buckling to determine which controlled. Because of the difficulty that would result, some specifications classify members strictly according to the thickness of the metal. For example, the AISI specification defines light gage members for buildings as those "cold formed to shape from sheet or strip steel less than $\frac{3}{16}$ inch thick and used for load carrying purposes . . . . "

The AISC specification is not intended to cover light gage members. This is shown by a statement in its preface, which reads:

SCOPE

As used throughout this Specification, the term "structural steel" refers exclusively to those items which are assigned to Class A in the "Code of Standard Practice for Steel Buildings and Bridges" of the American Institute of Steel Construction, and nothing herein contained shall be interpreted as a recommended practice for steel joists, members formed of flat rolled sheet or strip, light-gage steel construction, skylights, marquises (except structural frame), fire excapes, or other items not specifically assigned to Class A in that Code.

In aircraft, the design of light gage members is covered by specifications such as ANC-5, supplemented by research data and specifications of the manufacturers.

The design examples of this chapter are based on the AISI specification. For other specifications the principles are the same, although the exact form of the equations and the constants used are different. The various formulas of this chapter are taken from the AISI specification. The space available is not sufficient to permit the development or the justification of the formulas. This chapter attempts only to explain the principles in general and to show how the AISI specification is applied. It should be noted that much of the notation has a different meaning from that used elsewhere in this book.

9–2. Stiffened and Unstiffened Elements. The AISI specification classifies the elements of members as stiffened or unstiffened, according to the following rules:

1. **If an element is bounded on *both* edges, parallel to the direction of stress, by attached stiffeners or by formed lips or flanges offering a specified resistance to lateral deformation, the element is regarded as *stiffened*.**

2. An element having either one or both edges, parallel to the direction of stress, free from lateral restraint is regarded as *unstiffened.*

By this classification, the horizontal flanges of the member of Fig. 9–1(*a*) are unstiffened, while the vertical webs are stiffened by the flanges. The horizontal flanges of the sections in parts (*b*) or (*d*) are stiffened along one edge by the stiffening lips and along the other edge by the vertical webs. All elements in the member of part (*e*) are stiffened. The skin of (*f*) is stiffened; but the flanges of the stiffener channels, having one free edge, are regarded as unstiffened.

The reason for classifying elements as stiffened or unstiffened is that a difference exists in their manner of failure. An unstiffened element has a definite critical load at which sudden local buckling occurs. The critical unit stress may be less than the yield-point stress. The stiffened element does not buckle suddenly but, with increasing load, gradually develops buckling waves which reduce the effective section and cause a redistribution of stress. Article 9–3 will describe this failure in greater detail.

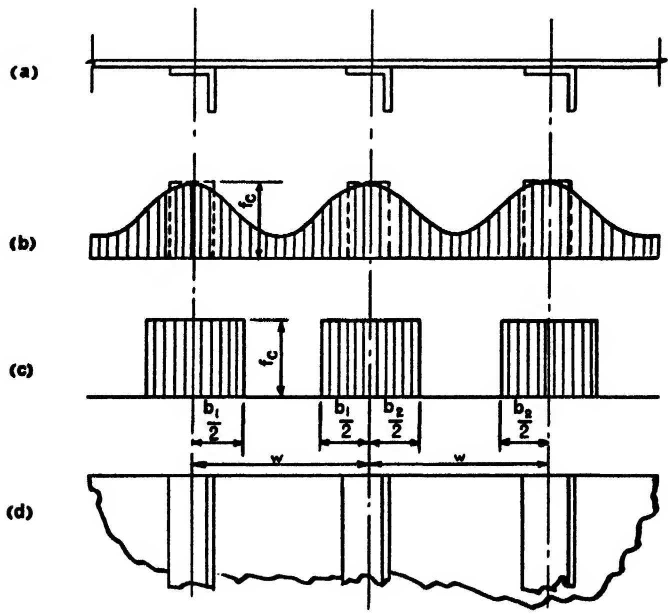


FIG. 9–2.

9–3. Stress Distribution in a Stiffened Sheet. A typical stiffened sheet is illustrated by Fig. 9–2. If a tensile load is applied, parallel to the stiffeners, the entire gross cross-sectional area, stiffeners and plate, will be uniformly stressed. No matter how thin the sheet or what the spacing of the stiffeners, uneven stress distribution or buckling of the sheet would not occur, except near points of concentrated load application.

Under small compressive loads, the entire section is effective and is stressed uniformly. As the compressive load is increased, portions of the sheet having low lateral stability buckle locally. As the applied load increases further, the first buckled portions remain stressed only to the intensity which caused them to buckle (assuming that the load is not great enough to cause ultimate collapse of the entire panel). The increase of load causes the stress to rise in the unbuckled portions and causes more material to buckle, thus increasing the width of the buckled portion.

The resulting distribution of stress on the cross section is similar to that shown by Fig. 9–2(*b*). The stress is least at the portions which buckled under the lowest total load, that is, at the center of the spaces between stiffeners or possibly at the free edges of the sheet. Closer to the stiffeners, where the bracing effect of the stiffeners is greater, a higher unit stress exists. The maximum stress occurs in the unbuckled portions or in the portions which buckled at the highest load. If longitudinal buckling of the sheet, between the rivets or welds connecting the sheet to the stiffener, does not occur, this maximum stress will exist in both the stiffener and the immediately adjacent portion of sheet.

For design purposes it is convenient to treat the flat element between stiffeners as having an effective width b, one-half adjacent to each stiffener. The total load on the member is assumed to be divided uniformly over the combined area of the stiffeners and the effective widths of the sheet. The balance of the sheet is assumed to be ineffective and unstressed. The assumed distribution on the stiffeners and the effective widths of sheet is shown by Fig. 9–2(*c*).

From the above description it should be apparent that the effective width varies with the stress. Formulas from the AISI and ANC-5 specifications apply this principle. In each the effective width is shown as a function of the unit compressive stress and of the flat-width ratio w/t. The analysis of a stiffened section is, then, performed by successive approximation. A stress on the effective section is assumed, then an effective section is computed for that stress, and finally a new unit stress is determined for the effective section. The new unit stress is used as the start of the second trial. Enough trials are made to give nearly constant results.

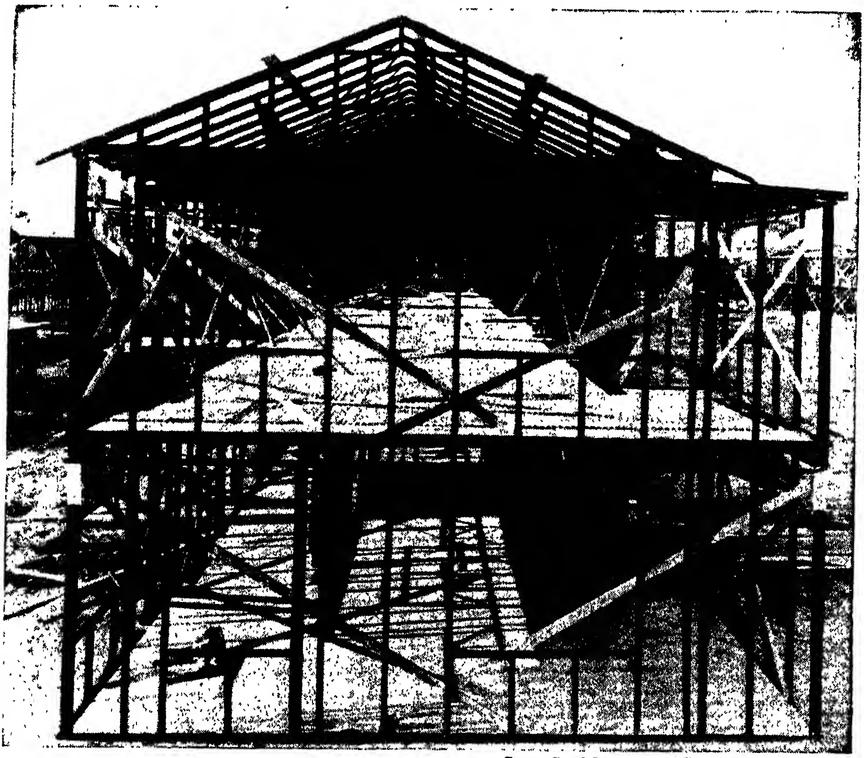
Since the analysis of some stiffened members is quite complex and the desired factor of safety is sometimes low (as in aircraft), full-size tests are frequently used to verify the computed strengths.

9–4. Unstiffened Light Gage Flexural Members. In the design of unstiffened flexural members, four considerations must be made, as follows:

1. The computed flexural stress, tensile and compressive, based on the properties of the entire section, must not exceed a basic allowable stress given by the

specification. In the AISI specification, basic stresses for three grades of steel are given, providing a factor of safety of 1.85 with respect to yield.

2. The allowable compressive flexural stress is limited so as to prevent lateral buckling of the entire compression flange. The purpose of this limitation and the method of determining it are like that given in Chap. 4 for heavier beams. (The exact formula for the reduced allowable stress is different from that of Chap. 4.)
3. Local buckling of the compression flange is prevented by limiting the allowable compressive stress according to the ratio of the flat width of the element to its thickness (flat-width ratio). The allowable unit stress is given by a formula



Stran Steel Division of Great Lakes Steel Corp.

LIGHT GAGE STEEL FRAMEWORK FOR A TWO-STORY BUILDING.

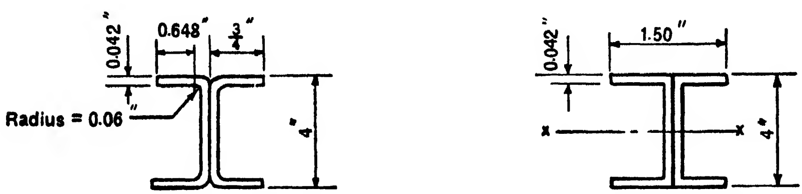
relating it to the flat-width ratio w/t or by a table in Section 3.2 of the AISI specification. This method of considering local buckling is somewhat like that given by Chap. 4 for structural aluminum sections.

4. If the span is short and subject to concentrated loads, a nonuniform stress distribution, and perhaps buckling diagonally to the longitudinal shear, may occur on the tension flange. Because of this, the entire section is not fully effective and some parts may be overstressed. To compensate, it may be necessary to consider only a portion of the tension flange in computing the sectional

properties. The portions to be considered are given by Section 2.3.5 and Table 2.3.5 of the AISI specification.

When the properties of the entire section [except as modified by consideration (4)] have been computed and the allowable unit stress determined, the flexure formula is applied, just as for the beams of Chap. 4.

EXAMPLE 9–1. Compute the allowable uniform load per foot for the beam section shown by Fig. 9–3(a). The simple span is 6 ft, and the distance between lateral supports is 3 ft. The material is the Grade B steel of the AISI specification.



Actual Section (a) Assumed Section (b)

FIG. 9–3.

For computation of the properties, the section is assumed to have sharp corners, as shown by Fig. 9–3(b). The error introduced by this assumption is slight when the curve radii are small as compared with the over-all section. Thus,

$$A = (4 \times 0.084) + (4 \times 0.708 \times 0.042) = 0.455 \text{ in.}^2,$$

$$I_x = \left(\frac{1}{12} \times 0.084 \times 4^3\right) + (4 \times 0.042 \times 0.708 \times 1.98^2) = 0.914 \text{ in.}^4,$$

$$I_y = \frac{1}{12} \times 0.084 \times 1.50^3 = 0.0236 \text{ in.}^4,$$

$$r_y = \sqrt{\frac{0.0236}{0.455}} = 0.228 \text{ in.}$$

The basic allowable stress for Grade B steel is given by Section 3.1 of the AISI specification as 16,500 psi. Note that this stress provides a factor of safety of about 1.85 with respect to the yield point of the material.

The allowable compressive stress, as limited by consideration of lateral buckling by the compression flange, is

$$\frac{250{,}000{,}000}{(L/r_y)^2} = \frac{250{,}000{,}000}{(36/0.228)^2} = 10{,}000 \text{ psi.}$$

The flat width ratio w/t for the flange is

$$\frac{0.648}{0.042} = 15.4.$$

The allowable compressive stress, as limited by consideration of local buckling, is given by the following formula when w/t is between 12 and 30;

$$f_c = (1.67f_b - 5{,}430) - \left(\frac{1}{18}\right)(f_b - 8{,}150)w/t.$$

Substituting the basic 16,500 for f_b and 15.4 for w/t (or using the table of solutions to this equation given by the specification), the allowable compressive stress, as limited by local buckling, is about 15,000 psi.

Comparing the three allowable compressive stresses, it is seen that the controlling value in compression is 10,000 psi. The allowable tensile stress is the basic 16,500 psi. Using $f = 10{,}000$ psi, the allowable bending moment and load per foot are computed, as follows:

$$M = \frac{fI}{c} = \frac{10{,}000 \times 0.914}{2} = 4{,}570 \text{ in.-lb, or } 380 \text{ ft-lb.}$$

The allowable unit load is

$$\frac{8M}{L^2} = \frac{8 \times 380}{36} = 84.5 \text{ lb per ft.}$$

The analysis just completed is for flexure only. The complete check should also include analyses for web shear, web compression, and the connection of the channels to each other so as to form a single member. These analyses will be discussed in later articles of this chapter.

9–5. Stiffened Light Gage Flexural Members. In analyzing a stiffened element, the first step is to determine the effective design width by successive approximation, as was shown by Art. 9–3. Flexural stresses are computed, using the flexure formula and the properties of the effective section. The computed stresses should then be compared with the allowables.

The allowable tensile flexural stress is the basic allowable for the material, as given by Section 3.1 of the AISI specification. For compression the allowable is either the basic stress or a stress reduced because of lateral buckling of the section, whichever is smaller. No reduction of allowable stress is made for local buckling, since the local buckling waves are already considered by using an effective width.

Tension flanges on short spans that carry concentrated loads are subject to the same treatment mentioned in the fourth consideration of Art. 9–4 for unstiffened beams, and the effective tension flange may be less than the full section.

Before elements may be definitely classified as "stiffened," the properties of the stiffeners must be checked to ensure that they are rigid enough to stiffen the element. The required properties for stiffeners are given by Section 2.3.2 of the AISI specification.

Examples 9–2 and 9–3 illustrate the AISI methods of analysis of stiffened flexural members.

EXAMPLE 9–2. The section shown by Fig. 9–4 is similar to that of Fig. 9–3, but it has lips which serve as stiffeners for the flanges. Assuming a 12-ft span, lateral support at 6-ft intervals, and Grade B steel, compute the allowable uniform load per foot of beam.

The web is considered stiffened, since it is supported along both edges by the flanges. The lips, while they serve as stiffeners, must themselves be considered as unstiffened elements because they have one free edge. The horizontal elements of flange may be considered stiffened if the strength of the lips satisfies the specification requirements.

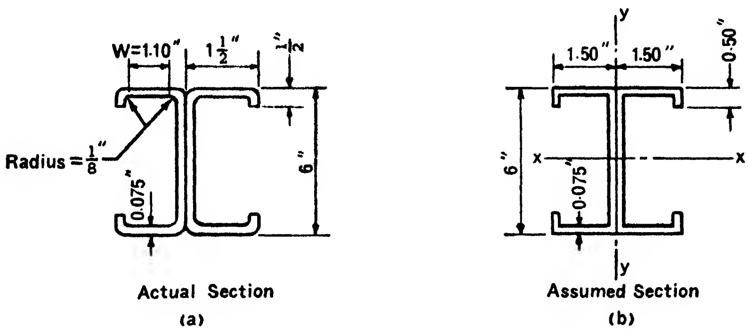


FIG. 9–4.

By Section 2.3.2 of the AISI specification, the moment of inertia of the stiffener itself must be at least

$$1.83t^4\sqrt{(w/t)^2 - 144}.$$

Substituting 1.10 for w and 0.075 for t, the required I for the lip is 0.000488 in.^{4} The I provided is

$$\frac{1}{12} \times 0.075 \times \overline{0.50}^3 = 0.00078 \text{ in.}^4,$$

which is sufficient.

Therefore the horizontal elements of the flanges may be considered stiffened. (The lip might be checked instead by use of an approximate formula for the required depth of a single lip. This formula is also given by Section 2.3.2 of the AISI specification.)

The flat-width ratio of the lips does not exceed 12. According to Section 3.2 of the specification, the allowable stress on an unstiffened element having w/t of not more than 12 may be as high as the basic allowable. Thus, in computing the effective section of the beam, the entire lip may be considered.

The flat-width ratio of the horizontal flange elements is 1.10/0.075, or 14.7. According to Section 2.3.1.1 of the AISI specification, the effective width b is equal to the flat width w for ratios up to $w/t = 3{,}790/\sqrt{f}$. The computed flexural stress will certainly not exceed the basic allowable stress of 16,500 psi. Thus, the limiting ratio will be at least as large as $w/t = 29.5$. The actual ratio, 14.7, is below this limit; therefore the entire flange width is effective.

$$A = (6 \times 0.150) + (4 \times 1.35 \times 0.075) + (4 \times 0.50 \times 0.075) = 1.455 \text{ in.}^2,$$

$$I_x = \left(\frac{1}{12} \times 0.15 \times \overline{6}^3\right) + (4 \times 1.35 \times 0.075 \times \overline{2.96}^2)$$

$$+ \left(\frac{4}{12} \times 0.075 \times \overline{0.50}^3\right) + (4 \times 0.50 \times 0.075 \times \overline{2.75}^2) = 7.39 \text{ in.}^4,$$

$$I_y = \left(\frac{2}{12} \times 0.075 \times \overline{3}^3\right) + (4 \times 0.425 \times 0.075 \times \overline{1.46}^2) = 0.610 \text{ in.}^4,$$

$$r_y = \sqrt{\frac{0.610}{1.455}} = 0.648 \text{ in.}$$ The basic allowable stress is 16,500 psi.

The allowable compressive stress for the flange, based on lateral buckling, is

$$\frac{250{,}000{,}000}{(L/r_y)^2} = \frac{250{,}000{,}000}{(72/0.648)^2} = 20{,}250 \text{ psi.}$$

The allowable compressive flexural stress for the web is given by Section 3.4.2 of the AISI specification as $f_w = 520{,}000{,}000/(h/t)^2$. Using the flat width of the web for h gives $f_w = 93{,}300$ psi. This allowable is limited to the basic stress.

Therefore, the basic allowable stress of 16,500 psi in the flange will control.

The resisting moment is

$$M = \frac{fI}{c} = \frac{16{,}500 \times 7.39}{3} = 40{,}600 \text{ in.-lb, or } 3{,}380 \text{ ft-lb.}$$

The allowable load is $\dfrac{8M}{L^2} = \dfrac{8 \times 3{,}380}{144} = 188$ lb per ft.

Considerable strength is added by the lips. A similar section, made without stiffening lips, would have only about 69 per cent of the capacity of the section of this example.

EXAMPLE 9–3. Determine the allowable uniform load per foot on the beam section shown by Fig. 9–5(a). The beam has a 10-ft simple span with lateral supports at intervals of 5 ft. The material is Grade A steel.

The allowable tensile stress is the basic value of 13,500 psi. The allowable compressive stress for the flange is either the basic value or the value given by the equation

$$f_c' = \frac{250{,}000{,}000}{(L/r_y)^2},$$

whichever is smaller. In this formula, the radius of gyration of the *entire* section is used. The computation of r_y and f_c' follows:

$$I_y = \left(\frac{1}{12} \times 0.06 \times \overline{4}^3\right) + (2 \times 0.06 \times 2.88 \times \overline{1.97}^2)$$

$$+ \left(\frac{2}{12} \times 0.06 \times \overline{0.5}^3\right) + (2 \times 0.06 \times 0.5 \times \overline{1.75}^2) = 1.845 \text{ in.}^4,$$

$$A = 0.06(4 + \overline{2 \times 2.88} + \overline{2 \times 0.5}) = 0.646 \text{ in.}^2,$$

$$r_y = \sqrt{\frac{1.845}{0.646}} = 1.69 \text{ in.,}$$

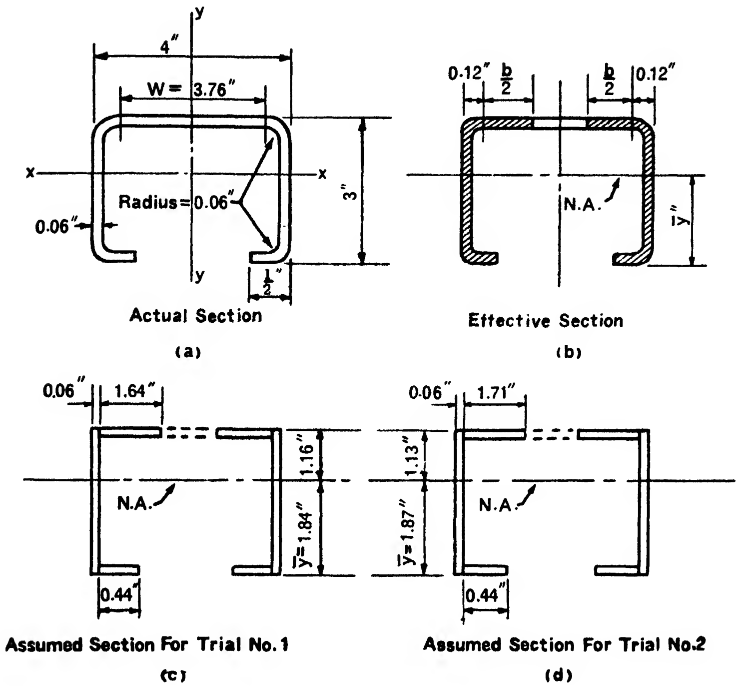


FIG. 9–5.

$$f_c' = \frac{250{,}000{,}000}{(60/1.69)^2} = 198{,}000 \text{ psi.}$$

Obviously, the basic value of 13,500 will control. Unless the location of the neutral axis is known, however, it cannot be told which extreme fibers, top or bottom, reach this stress intensity first. It will be assumed in this case that the neutral axis is closer to the top than to the bottom. When the bottom fibers are stressed to the tensile allowable stress of 13,500 psi, the upper fibers will be stressed in compression to a lower intensity.

For the first trial, assume that the compressive stress is 10,000 psi. The effective design width for the horizontal compression flange is determined, using the formulas of Section 2.3.1 of the AISI specification, as follows:

Limiting w/t (for $b = w$) $= 3{,}790/\sqrt{f} = 37.9$. The actual ratio $w/t = 3.76/0.06 = 62.7$. This exceeds 37.9 so that the following equation is used to compute the effective width b of the top flange.

$$\frac{b}{t} = \frac{7{,}590}{\sqrt{f}}\left[1 - \frac{1{,}900}{(w/t)\sqrt{f}}\right].$$

Substituting 10,000 for f in this equation, $b/t = 52.8$.

To permit an easier solution of the equations for effective width, Charts 3A and 3B of the AISI manual are shown in Appendix F.

The effective width b for the first trial is $52.8t$, or 3.17 in. The effective width is assumed to be located as shown by Fig. 9–5(*b*), one-half adjacent to each web. The location of the center of gravity is computed for an assumed section having square corners and the dimensions shown by Fig. 9–5(*c*):

Item	A	y	Ay
2 x 3 x 0.06	0.360	1.50	0.540
2 x 0.44 x 0.06	0.053	0.03	0.002
2 x 1.64 x 0.06	0.197	2.97	0.585
Σ	0.610		1.127

Thus,

$$\bar{y} = \frac{\Sigma(Ay)}{\Sigma A} = 1.84 \text{ in.}$$

When f on the tension side is 13,500 psi, f on the compression side is 13,500 × 1.16/1.84, or 8,510 psi.

Notice that for this type of problem, the moment of inertia is not needed except after the final approximation. If the load were given and the problem were to determine the actual stresses, however, a complete computation of the properties would be needed at each trial.

For the second approximation, the stress resulting from the first trial is used. The limiting ratio $w/t = 3{,}790/\sqrt{8{,}510} = 41.1$. The actual ratio w/t is larger, so

$$\frac{b}{t} = \frac{7{,}590}{\sqrt{8{,}510}}\left[1 - \frac{1{,}900}{62.7\sqrt{8{,}510}}\right] = 55.2.$$

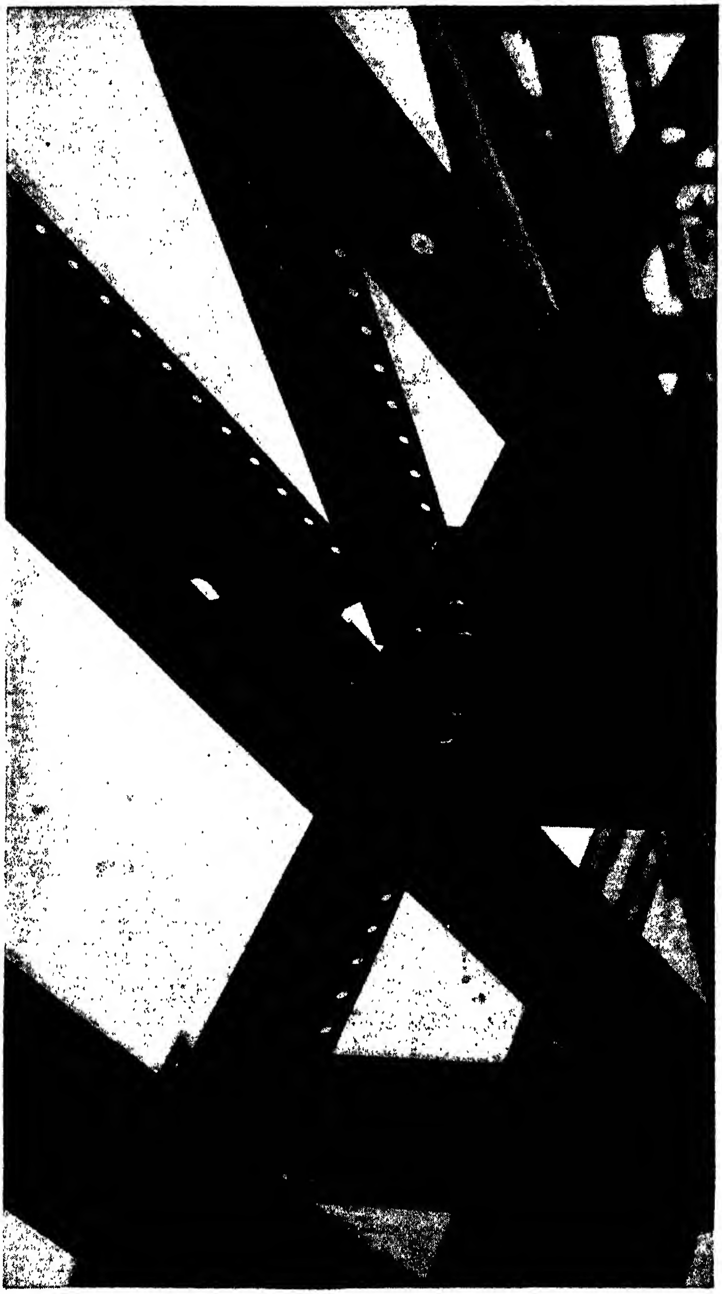
The effective width b is $55.2t$, or 3.31 in. The section properties are recomputed using this value and the assumed section of Fig. 9–5(*d*):

Item	A	y	Ay	Ay^2	I_o
2 x 3 x 0.06	0.360	1.50	0.540	0.810	0.270
2 x 0.44 x 0.06	0.053	0.03	0.002	0.0	0.0
2 x 1.71 x 0.06	0.205	2.97	0.610	1.811	0.0
Σ	0.618		1.152	2.621	0.270

Thus

$$\bar{y} = \frac{\Sigma(Ay)}{\Sigma A} = 1.87 \text{ in.}$$

The distances from the neutral axis to the tension and compression extreme fibers are, respectively, 1.87 in. and 1.13 in. When the tensile stress is 13,500 psi, the compressive stress will be (13,500 × 1.13)/1.87, or 8,150 psi. Using this value of stress and Chart 3A of the AISI manual, it is seen that further trials will give the same value of b/t as was used in this second approximation. Therefore, the moment of inertia of the section of this approximation is



Stran Steel Division of Great Lakes Steel Corp.

TYPICAL FRAMING DETAILS OF LIGHT GAGE MEMBERS IN BUILDING CONSTRUCTION.

$$I = \Sigma(Ay^2) + \Sigma I_o - \bar{y}\Sigma(Ay)$$
$$= 2.621 + 0.270 - (1.87 \times 1.152) = 0.737 \text{ in.}^4$$

The resisting moment is

$$\frac{fI}{c} = \frac{13{,}500 \times 0.737}{1.87} = 5{,}320 \text{ in.-lb. or } 443 \text{ ft-lb.}$$

The allowable load is

$$\frac{8M}{L^2} = \frac{8 \times 443}{100} = 35.4 \text{ lb per ft.}$$

9–6. Deflection of Stiffened Beams. In the analysis of a stiffened beam for strength, the effective width specified is that for a theoretical stress equal to the factor of safety times the stress computed for design loading. To obtain accurate values of deflection, however, the width effective under design load itself should be used. Therefore, in computing the deflection of a member, a different value of effective width is used than that in the strength computations.

The equations for effective design widths in deflection computations also are given by Section 2.3.1.1 of the AISI specification. Charts 3A and 3B give b/t values for strength and Charts 3C and 3D for deflection purposes.

Other than for the effective widths used, there is no difference between the methods used in computing the properties for strength and those for deflections.

9–7. Shear on Beam Webs. In computing the shear strength of light gage beams, the *average* web shear is used, just as specifications permit for structural I-beams. This average shear stress is compared with an allowable value, given by the specification. The allowable shear is limited by the AISI specification to either two-thirds of the basic working stress for bending or the quantity $64{,}000{,}000/(h/t)^2$, whichever is smaller. The term h is the clear distance between flanges, and t is the thickness of the web material. (See AISI Section 3.4. See also Section 3.4.3 for combined bending and shear.) The quantity $64{,}000{,}000/(h/t)^2$ is the shearing stress at which diagonal buckling will occur, divided by the factor of safety. It is the same quantity that is given by the AISC specification for the allowable shear on an unstiffened beam web. It is merely Euler's equation with appropriate constants and with the radius of gyration expressed as a function of the thickness t.

9–8. Web Crippling of Beams. When the web thickness and bearing length are known, the allowable reaction or concentrated load may be determined by the equations of Section 3.5 of the AISI specification. Other equations of that section permit the solution for bearing length when the load or reaction is given. Unlike the web crippling formulas used for structural steel, those of the AISI specification include the effect of buckling of the web at the location of the concentrated load or reaction. Their effect and purpose is similar to that of the equations given in Chap. 4 for vertical buckling of the webs of aluminum beams. The use of one of the AISI equations for web crippling is demonstrated by Ex. 9–4.

EXAMPLE 9–4. Check the beam of Ex. 9–3 to determine its shear value, and then determine whether or not a 1-in. length of bearing is sufficient to prevent web crippling at the end reaction.

For the section shown by Fig. 9–5(a), h is 3.00 − 0.12, or 2.88 in. Then

$$\frac{64{,}000{,}000}{(h/t)^2} = \frac{64{,}000{,}000}{(2.88/0.06)^2} = 27{,}800 \text{ psi.}$$

Two-thirds of the basic 13,500 is 9,000 psi.

The lower value of 9,000 psi is the controlling allowable average unit shearing stress. Two webs, each 2.88 in. deep by 0.06 in. thick, resist the applied shear. Thus

$$V \text{ (max allowable)} = 2 \times 2.88 \times 0.06 \times 9{,}000 = 3{,}110 \text{ lb.}$$

When the beam is loaded to its flexural capacity of 35.4 lb per ft, the actual shear is only 5 × 35.4, or 177 lb. Since the actual shear is less than the allowable, the section is satisfactory in shear.

Web crippling is checked by equation (1) of Section 3.5 of the AISI specification, as follows:

$$P_{max} = 100t^2[980 + 42(B/t) - 0.22(B/t)(h/t) - 0.11(h/t)]$$

Substitution of $t = 0.06$, $B = 1.0$, and $h = 2.88$ gives

$$P_{max} = 540 \text{ lb per web.}$$

The allowable reaction, as controlled by web crippling, is 2 × 540 = 1,080 lb. The 1-in. bearing length is satisfactory.

9–9. Light Gage Columns. Light gage columns might be made of any of the shapes shown by Fig. 9–1 and of numerous other shapes not shown. Column sections are classified according to whether they are composed entirely of stiffened elements, entirely of unstiffened elements, or of combinations of both types of elements.

The action of a stiffened sheet in compression was described in Art. 9–3. The stress in that member under very low loads is uniform, or $P = fA$. As the load is increased, some parts buckle. A buckled portion maintains the stress at which it buckled, while the stress on the unbuckled portions continues to rise with increase of load. Calling the stress on the first buckled part f_1, the area of that part a_1, and the stress on the other parts f, the load at which the first buckling occurs is

$$P = f_1a_1 + f(A - a_1).$$

Stran Steel Division of Great Lakes Steel Corp.

TYPICAL SECTIONS OF LIGHT GAGE STEEL.

If the load is increased until all the elements have buckled or reached the yield stress,

$$P \text{ (ultimate)} = f_1a_1 + f_2a_2 + f_3a_3 + \cdots f_na_n = \Sigma(fa).$$

The ultimate column load P divided by the highest unit stress, that which caused failure of the last element to occur, is called the *effective area* of the section. Unless the stiffeners themselves buckle, the part of the element nearest to the stiffener is stressed to the yield point by the ultimate load. Thus the effective area is P/f_y.

If the quantity $\Sigma(fa)$ is computed for the flat sheet alone,

$$\frac{\Sigma(fa)}{f_yA} = \frac{\text{effective area}}{\text{actual area}} = \frac{\text{effective width } b}{\text{flat width } w} = q.$$

The ratio b/w may be called the form factor q for the sheet. A form factor may be computed in a similar manner for the complete cross section. For example, consider a short column of a section similar to that of Fig. 9–1(e). The ultimate load for that section would cause all elements to either buckle or reach the yield-point stress. Expressed as an equation,

$$P = \Sigma(fa) \text{ for the wide webs } + \Sigma(fa) \text{ for the narrow webs } + \Sigma(fa) \text{ for the curved portions.}$$

Each of the terms $\Sigma(fa)$ into which the equation is divided could be substituted by $fyqA$, giving

$$P = f_y\Sigma(qA) = f_y\Sigma \text{ (effective areas).}$$

The form factor Q for the entire section is the ratio of its effective area to its actual area, or

$$Q = \frac{\Sigma(qA)}{\Sigma A} = \frac{P/f_y}{\Sigma A}.$$

The AISI equations for allowable stress on columns which fail by buckling of the entire column and without local buckling are a form of Euler's equation. For those whose failure is preceded by local buckling, the formulas include the factor Q, computed as above but based on a working stress f_b instead of the yield stress f_y. The slenderness ratio L/r of the columns, above which value the column fails without local buckling, is also given as a function of the factor Q.

For columns consisting of unstiffened elements or of combinations of both stiffened and unstiffened elements, the same AISI formulas are used but with different definitions for the factor Q.

The allowable stress given by the AISI formulas includes the effect of local buckling by inclusion of the term Q and therefore is applied, using the area and other properties of the entire cross section without reduction.

The slenderness of light gage columns is limited to specified maximum values, just as it is for columns of structural steel.

Members subject to combined axial load and bending are analyzed by a method similar to that of the AISC for members of structural steel. The member is considered satisfactory when $f_a/F_a + f_b'/F_b$ does not exceed one.

The difference between the AISC and AISI equations is only in the definition of terms. The terms of the AISI equation are defined as follows:

f_a = the computed average unit stress P/A, based on the entire section,

F_a = the allowable P/A, based on the entire section,

f_b' = the computed flexural stress, based on the entire section if unstiffened or on the effective design section if stiffened, and

F_b = the controlling allowable value of the stress f_b'.

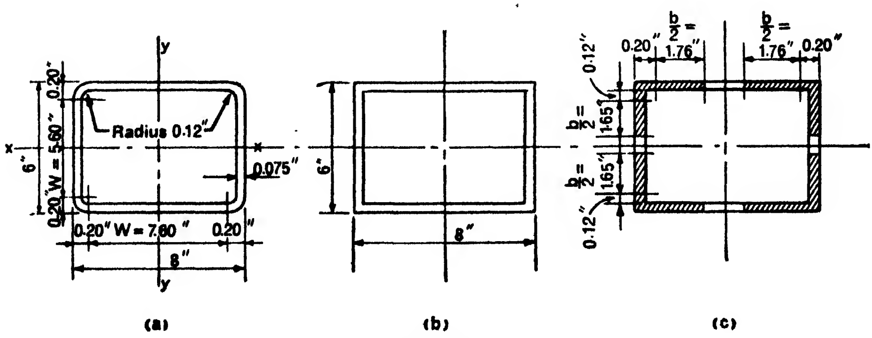


FIG. 9-6.

EXAMPLE 9–5. Find the allowable axial load for the column section shown by Fig. 9–6(*a*). The material is 14 gage (U.S. Standard), Grade C steel. The column is 12 ft long and laterally supported at the ends only.

By the definitions of Art. 9–2, the elements of this section are stiffened.

The allowable stress P/A, based on the entire sectional area A, is given for Grade C steel by one of the following equations:

$$\text{For } \frac{L}{r} \text{ not exceeding } \frac{132}{\sqrt{Q}}: \frac{P}{A} = 15{,}300Q - 0.437Q^2\left(\frac{L}{r}\right)^2;$$

$$\text{For } \frac{L}{r} \text{ exceeding } \frac{132}{\sqrt{Q}}: \frac{P}{A} = \frac{134{,}000{,}000}{(L/r)^2}.$$

In the above equations A and r are based on the properties of the entire area of the section. The term Q is, for stiffened sections, the ratio of the effective area (at the basic allowable stress value) to the entire area.

The properties of the entire area are computed first, using the assumed square-cornered section of Fig. 9–6(*b*):

$$A = 0.075(\overline{2 \times 8} + \overline{2 \times 5.85}) = 2.08 \text{ in.}^2,$$

$$I_{\min} = I_x = \left(\frac{2}{12} \times \overline{6}^3 \times 0.075\right) + (2 \times 7.85 \times 0.075 \times \overline{2.96^2}) = 13.02 \text{ in.}^4,$$

$$r_x = \sqrt{\frac{13.02}{2.08}} = 2.50 \text{ in.},$$

$$\frac{L}{r} = \frac{144}{2.5} = 57.6.$$

The basic allowable stress for Grade C material is 18,000 psi. Assuming f_a to be 18,000 psi and using Chart 3A of the AISI specification, the effective width of each element is determined:

$$\text{For the 7.6-in. flat width: } \frac{w}{t} = 101.3,\ \frac{b}{t} = 47, \text{ and } b = 3.53 \text{ in.};$$

$$\text{For the 5.6-in. flat width: } \frac{w}{t} = 74.7,\ \frac{b}{t} = 44, \text{ and } b = 3.30 \text{ in.}$$

The assumed square-cornered section, using the effective width b, is shown by Fig. 9–6(*c*). The effective area is

$$0.075 \times 4(1.76 + 0.20 + 0.12 + 1.65) = 1.12 \text{ in.}^2$$

Then

$$Q = \frac{1.12}{2.08} = 0.538,$$

$$\frac{132}{\sqrt{Q}} = 180.$$

Since L/r does not exceed $132/\sqrt{Q}$, the allowable average stress is

$$\frac{P}{A} \text{ (allowable)} = 15{,}300 \times 0.538 - 0.437\,(0.538 \times 57.6)^2 = 7{,}820 \text{ psi.}$$

The allowable axial load P is

$$2.08 \times 7{,}820 = 16{,}300 \text{ lb.}$$

9–10. Connections. The AISI specification gives definite requirements for connecting together the elements of such members as shown by Fig. 9–1(*a*), (*b*), or (*c*).

A single laterally unsupported channel used as a beam will twist unless the load is applied on a line passing through the "shear center" of the section. (See texts on strength of materials.) If the load does not pass through the shear center, the eccentricity m of the load from the shear center causes a torque. If the load per inch of channel length is q, the torque per inch is mq, as shown by Fig. 9–7(*a*).

If two channels are connected back-to-back, as in Fig. 9–7(*b*), a torque of $mq/2$ per inch occurs on each channel. The connections between the channels must furnish an equal internal resisting moment. It is assumed in Fig. 9–7(*c*) that the resisting moment is that of tensile and compressive forces in the connecting spot welds. The tension in the welds per inch of beam is

$$T = \frac{mq/2}{c}.$$

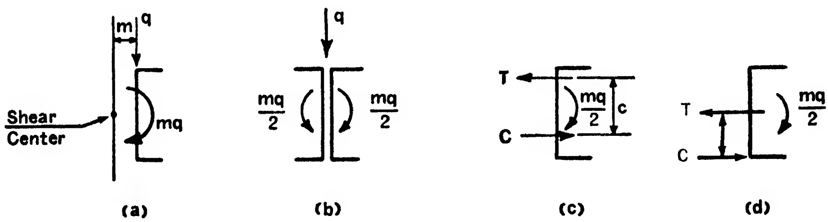


Fig. 9–7.

If the welds are placed a distance s apart (longitudinally), the load per weld is

$$S = \frac{mqs}{2c}.$$

This last equation is given by Section 4.3 of the AISI specification. Methods of obtaining the distance m are given by the specification for channels with stiffening lips and for channels without stiffening lips.

The term q is defined by the specification as *three times* the actual uniform load per inch, so as to include the effect of uneven loading of the channels caused by allowable errors of fabrication and by misalignment of the flanges. For concentrated loads or reactions, q is the load divided by the length in bearing or by s, whichever is larger.

Connections subject to longitudinal shear only are designed by the same methods used for the connections of reinforced beams or plate girders.

The welds joining the cover plates to the channels of Fig. 9–1(*c*) are subject to longitudinal shear and to transverse shear resulting from the tendency of each channel to twist when loaded in the plane of the web. The weld spacing is determined so that the resultant of the transverse and longitudinal shears per weld does not exceed the allowable shear per spot weld. In computing the transverse load per weld, the lever arm c of the internal resisting couple is obviously the depth of the channel.

For other types of connection patterns, similar procedures are followed. For example, the solution for a two-channel member joined by a single row of spot welds is like that for the member of Fig. 9–7(*c*) except that the lever arm c of the internal couple is the distance from the weld to the flange of the channel. In that case the internal couple is provided by tension T in the weld and compression C of one channel against the other, as shown by Fig. 9–7(*d*).

The design strength of spot welds is given by Section 4.2.2 of the AISI specification.

PROBLEMS

9–1. Assume that the section of Fig. 9–8 is used for a beam of 6-ft span with full lateral support of the compression flange. The material is Grade B steel as described by the AISI specification. Compute the allowable bending moment for the beam.

9–2. Compute the allowable bending moment for the beam of Prob. 9–1 when lateral support of the compression flange is supplied at intervals of 6 ft.

9–3. Compute the allowable bending moment for a laterally supported beam having a section like that of Fig. 9–8 but with the flange width equal to ⅝ in. instead of 1 in.

9–4. What would be the allowable bending moment for the beam section of Fig. 9–8 if it were used to support concentrated loads on a span of 2 ft? Assume Grade B steel. Notice Section 2.3.5 of the AISI specification.

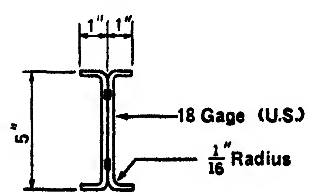


FIG. 9–8.

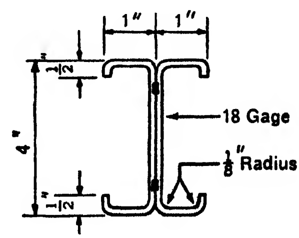


FIG. 9–9.

9–5. Assume that the section shown by Fig. 9–9 is made of Grade B steel and is laterally supported on a 6-ft span. Compute the allowable bending moment.

9–6. Same as Prob. 9–5, but for a section made of 24 gage material.

9–7. Compute the allowable uniform load per square foot for the stiffened panel of Fig. 9–10. Assume a 10-ft span parallel to the stiffeners. The flat sheet is on the compression side. Check for both flexure and shear.

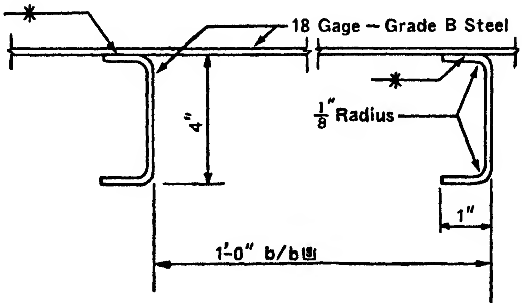


Fig. 9–10.

9–8. Determine the deflection of the beam of Prob. 9–7 when acting under the full allowable load as determined in that problem.

9–9. Compute the allowable shear for the section of Fig. 9–8. Assume Grade B material and two rows of spot welds, each row being 1 in. from the outside of the beam.

9–10. Compute the allowable shear for the beam of Prob. 9–5. Assume a single row of spot welds on the centerline of the section.

9–11. Compute the length of bearing required at the end of the simple span of Prob. 9–7 in order to prevent web crippling.

9–12. Compute the allowable axial load for the section of Fig. 9–9 when the material is Grade C steel and the laterally unsupported length is 5 ft.

CHAPTER 10

FATIGUE OF STRUCTURAL MEMBERS

10–1. Fatigue Failures. Failure of metals under repeated applications of load, commonly called "fatigue failure," is best described as failure by progressive fracture.

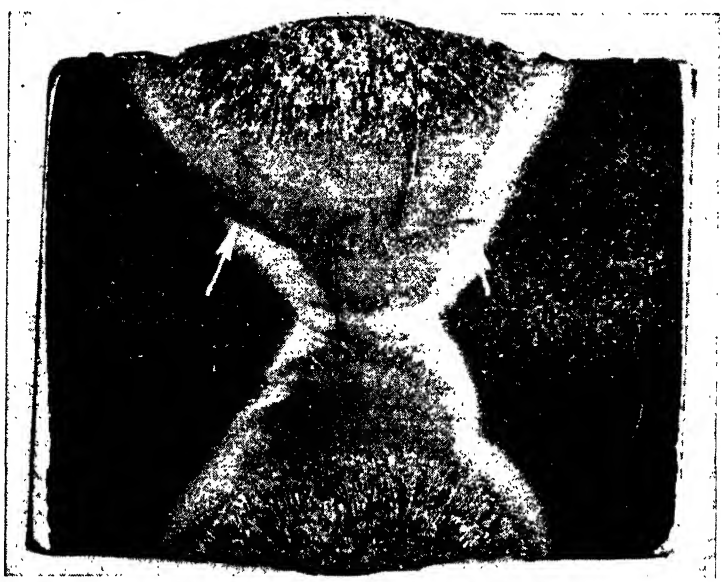


FIG. 10–1. Micrograph of a butt-welded joint after repeated loading to failure. ×1.5. [Source: Wilson, Bruckner, McCrackin, and Beede, "Fatigue Tests of Commercial Butt Welds in Structural Steel Plates," *University of Illinois Engineering Experiment Station Bulletin*, No. 344 (October 1943).]

A single application of load producing stress at the yield point will seldom cause a member to fail, but a large number of repetitions of load producing stress considerably less than the yield point may result in a sudden failure.

Figure 10–1 is a copy of a micrograph of a butt-welded joint tested in direct tension, consisting of applications of load producing 30,000 psi followed by release of the tension to zero stress.* Failure of this specimen

* W. M. Wilson, W. H. Bruckner, T. H. McCrackin, and H. C. Beede, "Fatigue Tests of Commercial Butt Welds in Structural Steel Plates," *University of Illinois Engineering Experiment Station Bulletin*, No. 344 (October 1943).

occurred after 220,700 cycles of this loading. Improper fusion or slag inclusions probably served as stress raisers within the weld, causing cracks to start and to progress slowly on repeated applications of the load. This type of failure is illustrative of fatigue failures which start as minute cracks and progress to cause ultimate failure.

There are many examples of fatigue failures which have occurred in bridge members after 20 to 30 years of service. Many such failures cannot be traced to a recent unusual loading but come about after a large number of repetitions of a load, which cause unit stress above the endurance limit of the material.

The fact that materials may fail under repeated loading while being adequate for static loading becomes an important factor in the design of certain types of structures. The anticipated number of repetitions of load producing relatively high unit stress is the factor which determines whether fatigue failure should be a consideration. A few thousand repetitions of loading producing stresses at the usual design values seldom need involve fatigue considerations. Very few structural members in a building will be subjected to sufficient repetitions of high load, followed by unloading, to require consideration of the endurance limit. Chord members of truss bridges receive relatively few repetitions of high stress as compared to floor members. Many types of industrial equipment have certain members receiving large numbers of repetitions of loading. Railroad car axles, rotating in the journals and receiving complete reversals of stress, are an example of members subjected to the possibility of progressive failure. Many members of an aircraft structure are subjected to large numbers of repetitions of loading, at rather high values.

The anticipated number of repetitions of stress of a given magnitude will vary with the structure and the conditions of its use. For example, 50 trains per day may pass over a single-track railway bridge. If all trains are pulled by the heaviest locomotives and all cars are fully loaded, the chords will probably be loaded 50 times per day, or 18,250 times each year and 912,500 times in 50 years. A member of the floor system, such as a stringer or floorbeam, in the bridge may receive a high stress as each pair of car trucks passes over it. Such a member may receive more than 40 million repetitions of relatively high stress during the 50 years. On the other hand, the maximum stress may not occur except when the drivers of the locomotive are on the member, and the number of repetitions may not be greater for the floor member than for the chords.

A crane may pass along a crane girder in a busy shop a hundred times in a day. The several spans of this girder may have repeated applications of loads as many as 30,000 times in a year, or over a million times in 35 years.

Maximum stress is not likely to occur so frequently in most members of a bridge when the bridge carries multiple tracks of railway or multiple lanes of highway. Each condition must be reviewed before an estimate can be made of the probable number of high-stress repetitions during its life.

10–2. Endurance Limit. The endurance limit for a given material is defined as the unit stress which can be repeated an indefinite number of

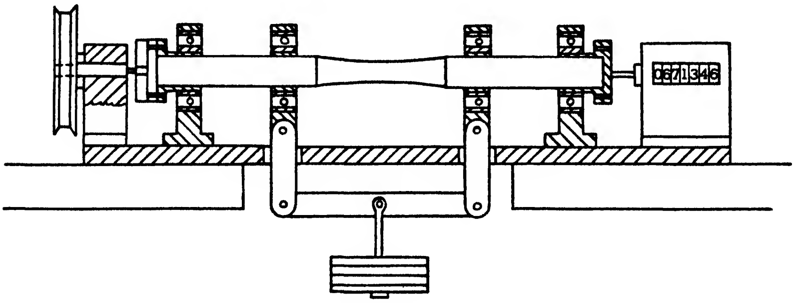


FIG. 10–2. The R. R. Moore fatigue testing machine.

times without failure by progressive fracture. Some of the factors that influence the number of times a given load may be applied are:

1. The ratio of the maximum unit stress to the yield point of the material;
2. The ratio of the maximum unit stress to the mean or average stress;
3. Stress concentrations;
4. The nature of the stress, that is, tension, compression, or shear; and
5. Ductility of the material.

Values given for the endurance limits of materials are usually determined by the rotating-beam method. The specimen is machined to standard dimensions and polished to remove all surface irregularities. The equipment for testing such specimens is known as the R. R. Moore fatigue testing machine. Figure 10–2 illustrates this type of machine diagrammatically. The specimen is rotated by means of an electric motor, and the number of rotations is recorded mechanically. The specimen is a short beam with circular cross section loaded with two symmetrically placed concentrated loads. The loading produces bending moment of uniform amount between the loads. Each 180° rotation of the beam causes unit stress at any point to change from tension to compression, or from compression to tension; each completed rotation causes a given point to receive one application of maximum tension and one application of maximum compression; the mean unit stress is zero, since the maximum stresses are equal and opposite.

The general method of determining the endurance limit is to test a series of specimens, decreasing the magnitude of load with each successive specimen until a loading is reached which does not cause failure with a large

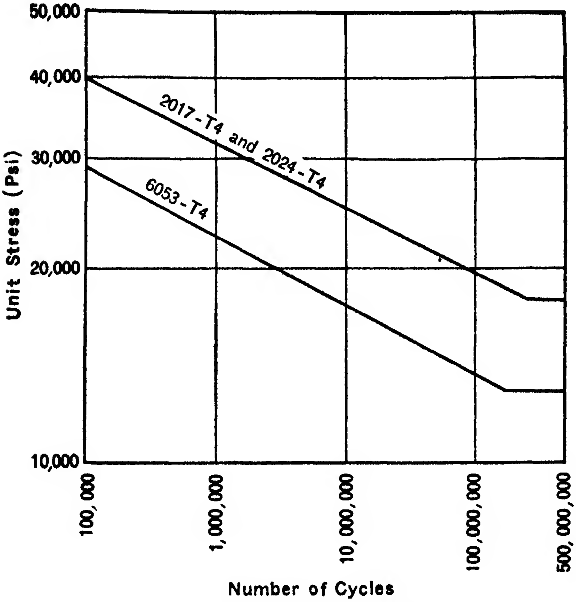


FIG. 10–3. Graph of fatigue values with rotating beam specimens.

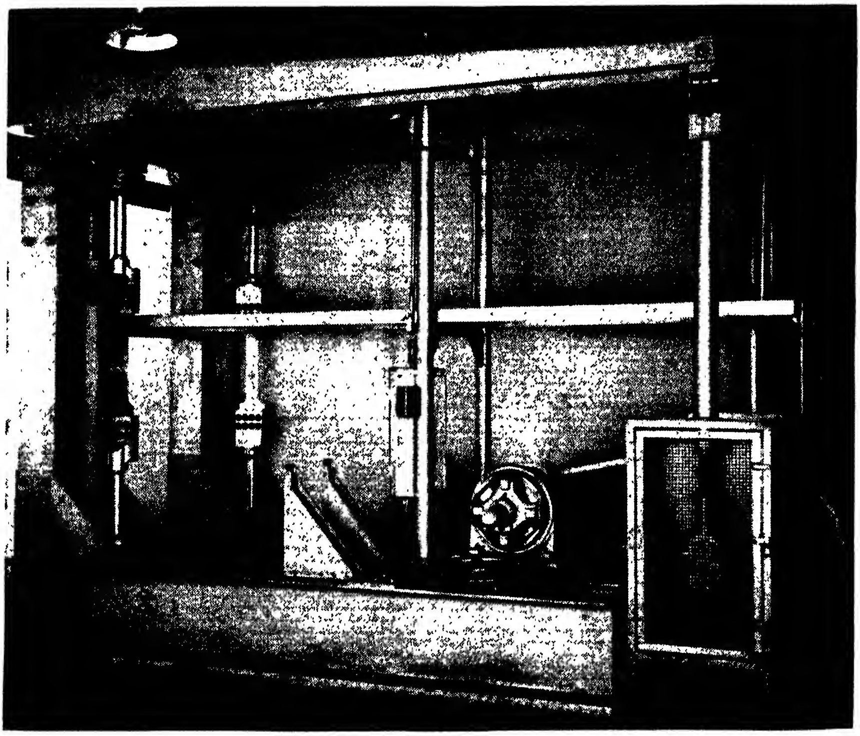
number of repetitions. The computed unit stress may be plotted against the number of repetitions to form the graph known as the S–N curve (often called the "Wöhler curve"). Typical S–N curves are shown in Fig. 10–3, plotted to log-log coordinate scales.

While the endurance limit is defined as the stress which may be repeated an indefinite number of times, it has been common practice to refer to a stress which can be repeated a given number of times as a fatigue limit. For instance, a material may have an endurance limit of 15,000 psi and may have a fatigue limit for 50,000 cycles of 30,000 psi.

The number of repetitions necessary to establish the endurance limit varies with different materials. In the case of structural grade steel (ASTM, A7), the S–N curve becomes practically horizontal at about 10,000,000 cycles, but some lighter metals, such as 2017-T4 aluminum alloy, may require from 100 to 500 million cycles before the lowest limit of stress has been established.

Endurance limits established by polished rotating-beam specimens do not provide the final values against which structural members may be designed because they are based on one type of stress variation and are without surface imperfections. The actual design conditions may be alternately high and low values of stress in the same direction, or alternate tension and compression of unequal amounts. Furthermore, the structure as built is not composed of flawless polished material but has rivet holes, sudden changes of section, mill scale, or other types of stress raisers. The endurance limit as established by polished specimens provides a fairly definite characteristic of the material and has valuable use in determining the effects of other factors.

A notch or nick in the surface of a fatigue specimen causes unit stress locally in excess of the average unit stress. Static tests will not establish the magnitude of such stress raisers because of the redistribution of stress at the elastic limit. However, fatigue tests which stress the material below



Aluminum Company of America

A 50,000-LB TEMPLIN STRUCTURAL FATIGUE TESTING MACHINE.

the yield point are not influenced by redistribution of stress and serve as a means of evaluating the effects of a stress raiser. For example, if the endurance limit has been established for a given material at 15,000 psi with polished specimens and is found to be 12,000 psi for specimens with a particular type of stress raiser, this stress raiser would be considered as having produced a local unit stress of 3,000 psi.

10-3. The Effect of Stress Range. One type of stress variation has been described for which an endurance limit may be determined experimentally—the rotating beam, which produces alternately tension and compression of equal amounts.

Other types of stress variation have been the subjects of many investigations, such as repeated torsional shear, direct tension varying from zero to maximum, tension varying from a large value to a lesser value, compression varying from zero to maximum, and alternate direct tension and direct compression of unequal magnitudes.

A majority of bridge or aircraft members are subjected to a steady load produced by the weight of the structure and to a superimposed load due to live load and impact. The live load may produce an increase in stress in the same direction as the dead-load stress, may reverse the direction of the stress, or may produce tension and compression alternately during the passage of the live load on the structure.

The endurance limit for a material varies with each type of stress variation. As an example, 2017-T4 aluminum alloy has an endurance limit of 15,000 psi, as established by 500 million cycles with the rotating beam, but has a value of 27,000 psi for 500 million cycles of direct tension, when the stress varies from zero as the minimum. Figure 10–4 shows several S–N curves between 100,000 and 500 million cycles for 2024-T4.* It will be noted that, as the minimum stress of the cycle increases, the fatigue limit also increases.

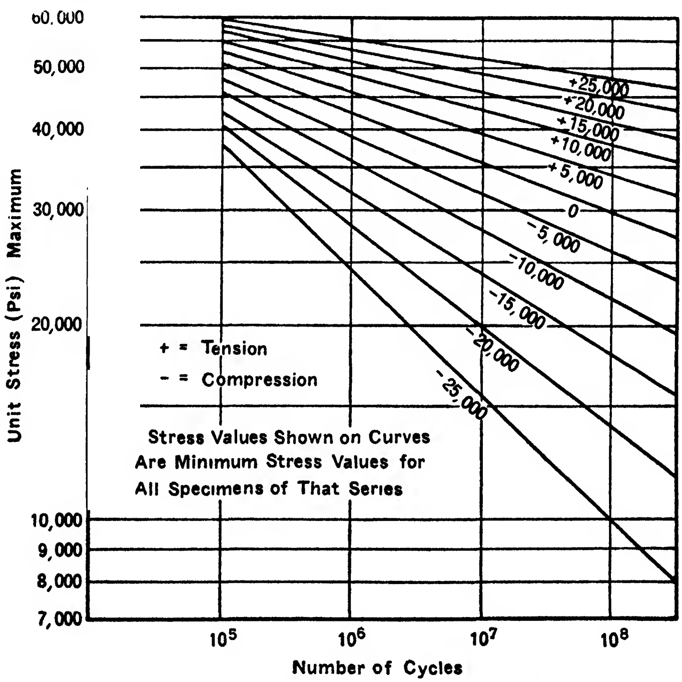


FIG. 10–4. The effect of stress range on the fatigue limit of 2024-T4 aluminum alloy.

* Aluminum Company of America, *Structural Aluminum Handbook* (Pittsburgh: 1948), pp. 25–26.

The Goodman diagram* (Fig. 10–5) is a graphical representation of the approximate values of endurance limits for any value of the minimum stress in the cycle. This diagram is based on a series of tests by Wöhler. Line OA is drawn at 45° with the coordinate axes; point A is a point having an ordinate equal to the static tensile value of the material; line OA is extended to a point C having an ordinate equal to one-third of the ultimate tensile limit; point B is plotted an equal distance above the zero line; and DA is drawn from A through the intersection of BC and the zero line. When the minimum stress of a cycle is represented by the ordinate to line AC (s_m), the intercept of an ordinate between AC and AB represents the

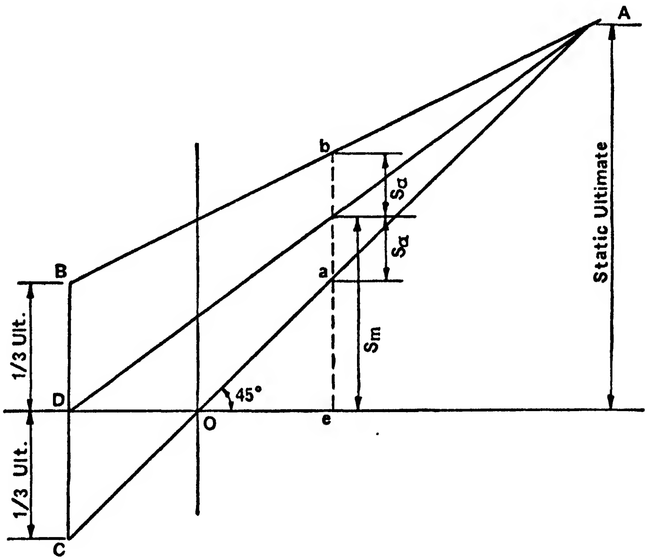


FIG. 10–5. Goodman diagram.

anticipated applied stress in the cycle which could be applied an indefinite number of times. Line AD cuts the applied stress into two equal parts (s_a), and hence represents the mean of the oscillating stress.

Many recent investigations indicate that a modified Goodman diagram† represents the anticipated fatigue limits somewhat closer than the original Goodman diagram. This modified diagram is developed by extending the line AC to the point established by complete reversal of stress, rather than to a value of one-third of the ultimate tension. Point B is then the tensile endurance limit for complete reversal. Fig. 10–6 shows the modified

* J. Goodman, *Mechanics Applied to Engineering* (New York: Longmans, Green & Co.), p. 635.

† James O. Smith, "The Effect of Range of Stress on the Fatigue Strength of Metals," *University of Illinois Engineering Experiment Station Bulletin*, No. 334 (February 1942).

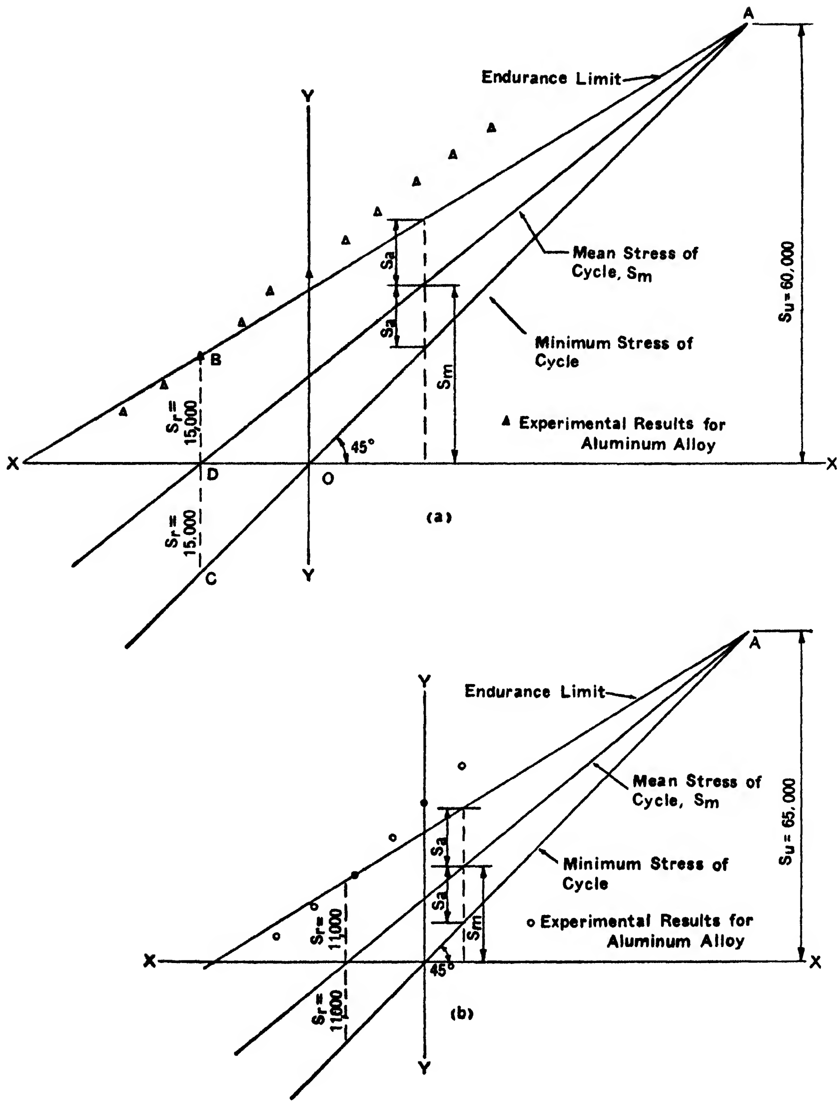


FIG. 10-6. Modified Goodman diagram.
Symbols: Δ, 2017-T4 Aluminum Alloy; o, 6061-T4 Aluminum Alloy.

Goodman diagram with the values of endurance limits for direct stress plotted for two aluminum alloys.* Although the line AB does not coincide with the values of the endurance limits found by test, it is probably a fair representation of the values which might be expected. It will be noted

* *Structural Aluminum Handbook*, pp. 25–26.

that, as the value of the minimum stress in the cycle increases, the amount of the superimposed load decreases.

A cycle in which the applied unit stress varies from zero to 20,000 psi tension may be described as a cycle of ± 10,000 psi alternating stress superimposed on a steady stress of 10,000 psi tension. Similarly, a stress variation of 15,000 psi compression to 5,000 psi tension may be described as an alternating stress of ± 10,000 psi from a steady stress of 5,000 psi compression.

Figure 10–7 is typical of the diagrams known as "steady stress-alternating stress" diagrams.* When an abscissa represents the ratio of the steady stress, s_m, to the ultimate static tensile value of the material, s_u, the corresponding ordinate represents the anticipated alternating stress s_a which may be superimposed on the steady stress in terms of a percentage of the complete reversal endurance limit s_r. For example, if a material has a complete reversal endurance limit s_r of ± 20,000 psi, when such a stress cycle is applied the steady stress s_m would be zero and the ordinate OA would represent the alternating stress s_a. If the same material has a

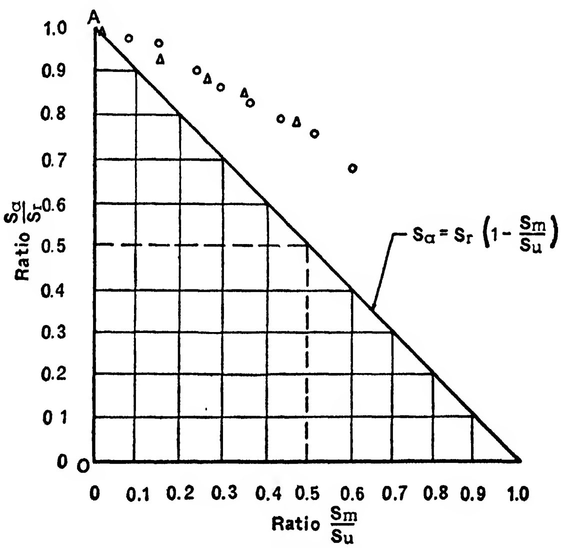


FIG. 10–7. Soderberg diagram.
Alloy symbols: ○, 2017-T4; △, 6053-T4.

static tensile ultimate s_u of 60,000 psi, and a stress cycle having a steady or mean value of 30,000 psi is applied, it is anticipated that an alternating stress s_a of 10,000 psi (0.5 of 20,000) may be applied an indefinite number of times without failure by progressive cracking. The abscissa corre-

* C. R. Soderberg, "Working Stresses," *Trans. ASME*, Applied Mechanics Division, Vol. 1, No. 3 (July-September 1935).

sponding to $s_m = 30{,}000$ would be $0.5\ s_u$ and the corresponding ordinate would be $0.5\ s_r$. Based on known values for the static ultimate stress s_u and an endurance limit for complete reversal s_r, it is anticipated that the material will withstand a stress cycle varying from 20,000 psi to 40,000 psi in tension as an endurance limit.

In general, fatigue values in which the maximum tension stress, $s_m + s_a$, is greater than the yield point of the material have little significance, since general yielding will occur.

The diagram of Fig. 10–7, frequently called the "Soderberg diagram," is similar to the modified Goodman diagram of Fig. 10–6, except in scale. In the case of most ductile materials having tension as the steady stress (mean stress), these diagrams are conservative in that a majority of test values are higher than anticipated by the use of the diagrams.

From the results of a large number of tests collected from various sources, J. O. Smith* has made studies of both ductile and brittle materials for cycles in which the steady stress was tension, torsion, or compression. Both polished and notched specimens were compared. The following conclusions can be drawn from the graphs for the materials represented, when all tests were made with notch-free specimens:

1. For ductile materials, the predicted alternating stress endurance limit is conservative when the steady stress is tension; that is, practically all test values fall above the line $s_a = s_r(1 - s_m/s_u)$.
2. When the steady stress is compressive, each ductile material has a fairly constant endurance limit for alternating stress for various values of steady stress. This alternating stress endurance limit is equal to or greater than the endurance limit s_r, for complete reversal. There were four test results for which endurance limits less than s_r were found. The range in steady stress included in the tests was from 0.3 to 1.2 times the compressive yield stress and included 13 ductile materials.
3. For 27 ductile materials, torsional fatigue tests indicated that the endurance limit for alternating stress is a constant for any material and for all values of the steady stress, so long as the maximum stress does not exceed 80 per cent of the torsional yield stress. The value of the alternating stress s_a which can be superimposed on a steady stress s_m is equal to the torsional endurance limit for that material under complete reversal, that is, with the mean stress s_m equal to zero.
4. With brittle metals the alternating tension, compression, or torsional stress which can be superimposed on a steady stress is less than would be indicated by the Soderberg diagram and can be fairly well predicted from the curve,

$$s_a = s_r \frac{1 - \dfrac{s_m}{s_u}}{1 + \dfrac{s_m}{s_u}}.$$

* Smith, *op. cit.*

H. F. Moore* has developed the expression $(FL)' = 3(FL)/(2 - r)$ from the results of fatigue tests of polished specimens. The value (FL) is the endurance limit for complete reversal of stress; the ratio of minimum to maximum stress is called r; the value $(FL)'$ is the anticipated endurance limit corresponding with r. Values from fatigue tests† on riveted joints indicate that the same expression may be applicable to other than polished specimens.

10–4. Stress Concentrations. Sudden changes in cross section, such as those caused by rivet holes, screw threads, mill scale, and welds, result in concentrations of stress beyond the values indicated by straight-line distribution. Many independent investigators have established concentration factors for various types of stress raisers. Such studies are usually experimental, and use of the results is limited to conditions similar to those of the tests. The application of test information to conditions differing from the conditions of the tests may give widely erroneous results.

An important characteristic of ductile metals is their ability to absorb high localized stress by redistribution after the yield point has been exceeded. Static tests have proved that high localized stresses do not reduce the value of ductile metals in resisting a single application of tension.

Frocht and Hill‡ reported stress-concentration factors varying from 2 to 13.7, as a result of photoelastic studies of the effects of holes in flat plates. The stress-concentration factor is the ratio of the measured maximum stress intensity to that computed by the straight-line distribution methods; that is, when K is the stress-concentration factor, the maximum unit stress is considered to be KP/A, or KMc/I. The Frocht and Hill tests included varying ratios of hole size to plate width, varying edge distances in line of loading, and varying fits of the loading pin.

Figure 10–8 shows a typical photograph of a polarized light study of the effect of a hole on the stress fringe pattern in a plate subjected to tension.

Bulletin No. 302 of the University of Illinois Engineering Experiment Station reported static tests on a series of riveted joints in which the net section through rivet holes developed equivalent strength to control specimens cut from the same plate, confirming the results of similar tests by others. Comparing the detail of the plates used in the Illinois tests with the Frocht and Hill tests, a stress-concentration factor of about 3.5 would be expected. It is quite probable that such a concentration factor may have been effective with the first application of load and until the maxi-

* H. F. Moore, *Materials of Engineering* (7th ed.; New York: McGraw-Hill Book Co., Inc., 1947) p. 60.

† W. M. Wilson and Frank P. Thomas, "Fatigue Tests of Riveted Joints," *University of Illinois Engineering Experiment Station Bulletin*, No. 302 (May 1938).

‡ M. M. Frocht and H. N. Hill, "Stress Concentration Factors around a Central Circular Hole in a Plate Loaded through a Pin in the Hole," *Journal of Applied Mechanics*, March 1940.

mum intensity of stress reached the yield point of the steel, after which the yielding of the highly stressed portions caused redistribution of the stress.

The effect of stress-concentration factors on members subjected to repeated loads producing unit stresses above the yield point is a subject of primary importance and is an area of study in which research is in its

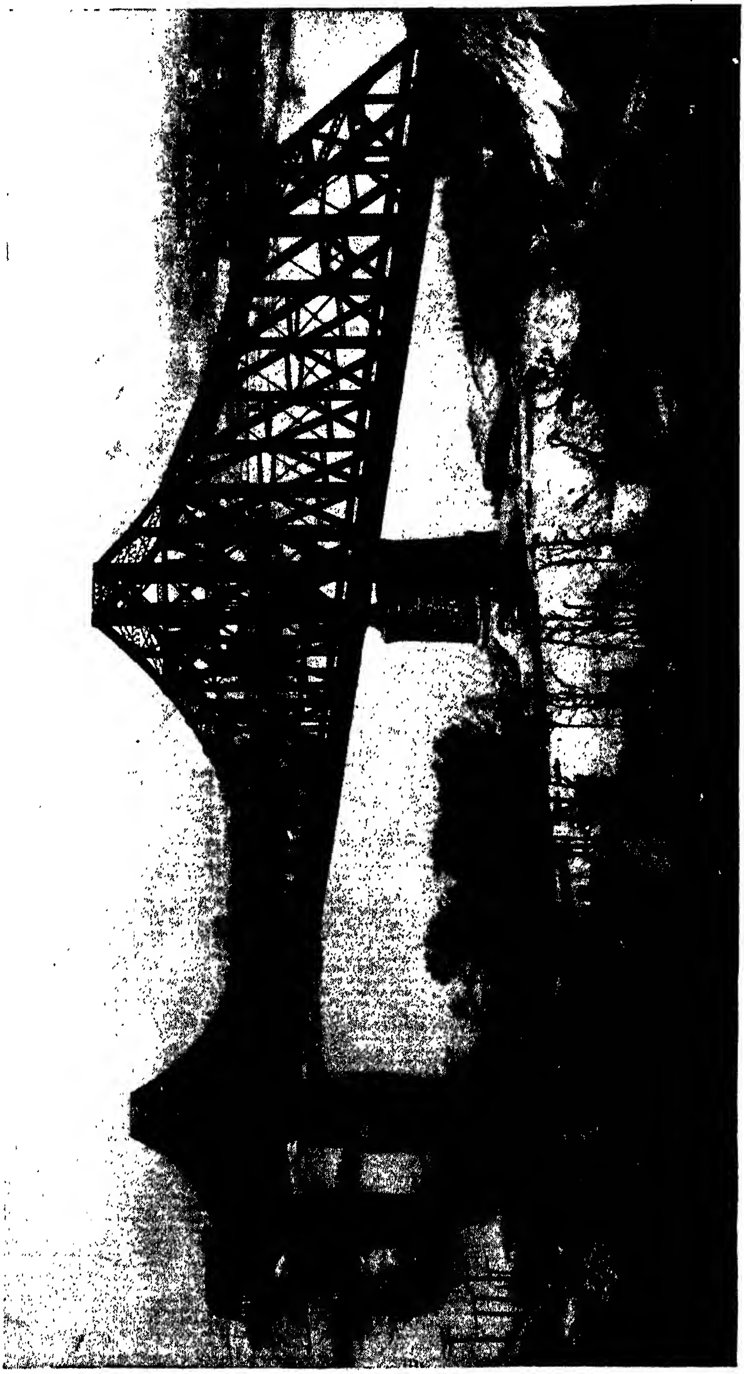
FIG. 10–8.

infancy. Stress cycles may produce unit stress which exceeds the yield point slightly but which is not sufficient to cause general yielding and redistribution of stress; each stress range, producing different amounts of redistribution, becomes a special problem.

E. C. Hartmann* has suggested one approach to the problem of dealing with known stress-concentration factors. His proposal is to reduce the effect of a stress-concentration factor when the maximum unit stress produced by it is greater than the plastic yielding value of the material. He uses an example in which a stress-concentration factor of 2.5 is considered to be constant until the value $2.5P/A$ (axial loading) is equal to 34,000 psi on aluminum alloy 2017-T4; then he reduces the factor from 2.5 to 1.0 as the unit stress approaches the ultimate tensile value of the material. Figure 10–9 is taken from his paper on the subject.

A broad field for research in determining stress-concentration factors is

* E. C. Hartmann, "Fatigue Test Results, Their Use in Design Calculations," *Product Engineering*, February 1941.



Bethlehem Steel Co.

P. & L. E. Railroad Bridge over Ohio River, Beaver, Pa.

open. The various details which produce stress concentrations and the effect of stress range on the factors provide a problem which has tremendous possibilities for test and study.

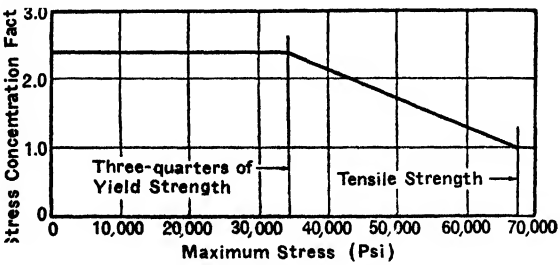


FIG. 10–9. Proposed method of modifying stress-concentration factors to take plastic deformations into account in 2017-T4 aluminum alloy. (Source: *Product Engineering,* February 1941.)

10–5. Riveted Members. Little work has been done in fatigue testing of riveted members. The cost of specimens and equipment and the time required for the application of a few million cycles of loading are factors which slow the progress of tests in this area. There are so many combinations of structural shapes used in design and so many possible rivet combinations that an almost infinite number of problems exist. Methods of approach have been to make tests on basic elements, such as riveted plates, with the hope of establishing relations which may be interpreted into more complex combinations.

While determination of the endurance limits of structural steel usually requires from 5 to 10 million stress cycles, it has been common to determine fatigue values for structural elements at 2 million cycles in the belief that this value will be ample for service conditions. One of the common characteristics of fatigue S–N curves which assists in reducing the number of specimens to be tested is that the curves between the values of N = 100,000 and N = 2,000,000 are usually straight, when plotted to logarithmic scales. Tests at the University of Illinois* indicate that this line has the equation $S = B/N^k$, in which S is the maximum stress in the stress cycle, N is the number of cycles for failure, and the values of B and k are experimental constants.

Since fabricated specimens cannot be identical in either material or detail, considerable variation in test results is expected. It is desirable that any established fatigue value be the average of several tests, with specimens as nearly alike as is practicable. Presumably, when the number of cycles corresponding to a given fatigue value and the constant k have been determined, a fatigue limit foı a different number of cycles might be

* H. F. Moore and J. B. Kommers, "An Investigation of the Fatigue of Metals," *University of Illinois Engineering Experiment Station Bulletin,* No. 124 (October 1921), p. 92.

predicted from the developed equation $F = SN^k/n^k$, in which S and n correspond with the test results, and N is the number of cycles for which fatigue value F is desired. When the values of k and N are dependent on curves established by two fatigue values, these curves can be made more dependable by the use of a rather wide range of N (number of cycles).

At the present time, available test information pertaining to fatigue limits for riveted members is of a preliminary or pilot nature. The University of Illinois published in Engineering Experiment Station Bulletin No. 302* the results of a series of tests on riveted joints. These tests were made on a fatigue testing machine capable of applying axial loads varying from 200,000 lb tension to an equal compression. The applied load may be adjusted to any desired range of maximum and minimum values within the limits of the machine. The speed of the machine is such as to apply 180 cycles of stress per minute; this speed requires approximately 8 days of continuous operation for 2,000,000 cycles of stress on one specimen.

The Illinois report cautions against the use of developed data as conclusive and suggests several items in which research would be desirable. However, the test data for more than 180 specimens suggest a few probable conclusions. Tests covered plate specimens with 1-in. diam rivets, with some specimens designed to fail in the rivets and others designed to fail in the plates. The fatigue strength was defined in the report as the unit stress which would withstand 2,000,000 cycles. Unit stress was computed by dividing the load by the area of the rivets in shear or by the net cross section of the plates, thus representing average rather than maximum unit stresses. A study was made of the effects of rivet grip, transverse spacing of rivets, relation between unit shear, bearing, and tension, combinations of carbon steel, silicon steel, and nickel steel plates with carbon steel and manganese steel rivets, and the methods of making rivet holes. Static tests were made on specimens of each type to provide comparative data. A majority of tests were made with a stress cycle varying from zero to tension. Metallurgical studies were made of the various specimens and data were taken pertaining to the slip of the joints. In general, the test results indicated the following:

1. The fatigue strength of rivets subjected to repeated shear from zero to maximum is in the order of 30,000 psi.
2. When slip occurs between the plates, the fatigue strength of the rivets is reduced.
3. Reversal of direction of stress has more effect on the fatigue strength of the rivets than it has on the fatigue strength of the plates.
4. There was some evidence that decreasing the transverse spacing of rivets increased the fatigue strength of carbon steel plates.
5. The fatigue strength of the plates was found to be in the order of 26,000 psi

* Wilson and Thomas, *op. cit.*

for a stress range from zero to tension, with no apparent increase in value for silicon steel and nickel steel over carbon steel.

More recent tests* by the same experimenters included tests on plates without rivet holes and indicated a fatigue value of 22,800 psi at 2,000,000 cycles for carbon steel with a stress range from maximum tension to equal compression. Applying the formula $(FL)' = 3(FL)/(2 - r)$, in Art. 10-3, the anticipated fatigue strength for a range of stress from zero to tension is $(FL)' = (3 \times 22{,}800)/(2 - 0) = 34{,}200$ psi. The fatigue value of 26,000 reported in Bulletin No. 302 would thus indicate that a stress-concentration factor of 1.32 could be applied to the average of the specimens.

University of Illinois Bulletin No. 350† reports a series of tests on carbon steel plates subjected to complete reversal with fatigue strength on the gross section of 11,700 psi for 2,000,000 cycles of 16,100 psi on the net section. Converting this value to a probable value with a stress range from zero to tension, $(FL)' = (3 \times 16{,}100)/(2 - 0) = 24{,}150$ psi, which is somewhat less than that reported in Bulletin 302. It is of interest to note that the steel used in the tests reported in Bulletin 302 had a yield point of the order of 35,000 psi and an ultimate strength of 63,000 psi, while the steel reported in Bulletin 350 had a yield point stress of 45,600 psi and an ultimate of 64,100 psi. There is some evidence that steels with high yield points tend toward low fatigue limits. This is a subject requiring further study.

Bulletin No. 302 suggests several factors for further study, among which are:

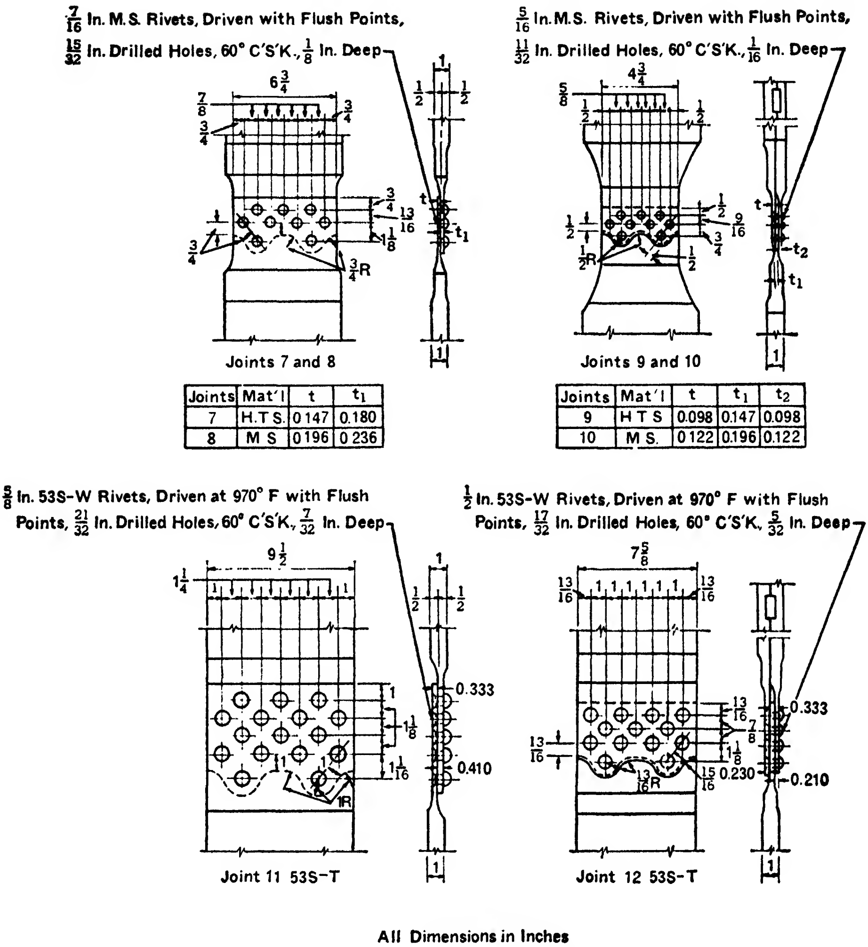
1. The effect of the number of rivets in a line in the direction of stress upon the fatigue strength of the rivets,
2. The effect of tightness of fit of rivets on fatigue strength of the rivets,
3. The effect of frictional resistance between the plates on the fatigue strength of the rivets,
4. The relation between rivet grip and rivet fatigue strength,
5. The effect of fills on the fatigue strength of rivets.

Technical paper No. 5 of the Aluminum Research Laboratories‡ contains a report covering fatigue values for plate splices of both single- and

* W. M. Wilson, W. H. Bruckner, J. V. Coombe, and R. A. Wilde, "Fatigue Tests of Welded Joints in Structural Steel Plates," *University of Illinois Engineering Experiment Station Bulletin*, No. 327 (February 1941).

† W. M. Wilson, W. H. Bruckner, J. E. Dubery, and H. C. Beede, "Fatigue Strength of Fillet Weld and Plug Weld Connections," *University of Illinois Engineering Experiment Station Bulletin*, No. 350 (1944), p. 76.

‡ R. L. Templin and E. C. Hartmann, *Static and Repeated Load Tests of Aluminum Alloy and Steel Riveted Hull Plate Splices* (Pittsburgh: Aluminum Company of America, 1941), Technical Paper No. 5.



Joints	Mat'l	t	t_1
7	H.T.S.	0.147	0.180
8	M.S.	0.196	0.236

Joints	Mat'l	t	t_1	t_2
9	H.T.S.	0.098	0.147	0.098
10	M.S.	0.122	0.196	0.122

FIG. 10–10. Nominal dimensions of small joints for fatigue tests. (Source: Technical Paper No. 5, Aluminum Company of America.)

double-shear types with three kinds of material—mild steel, high-tension steel, and aluminum alloy 6053-T4. Fig. 10–10 shows the detail of the joints tested, and Table 10–1 gives the relations developed from the test results. The lower value of the single-shear types was to be expected because of the bending in the plates. The relatively low fatigue strengths of the high-tensile steel is noteworthy.

10–6. Welded Members. Fatigue tests on welded members bring out very clearly the effects of sudden change in cross section. Failure seldom occurs in the weld, but the fatigue values for welded plates are frequently reduced considerably in the vicinity of the welds when the transition in cross section is sudden.

TABLE 10-1

Summary of Results of Repeated Load Tests

Joint No.	Single or Double Butt Straps	Material		Fatigue Strength of Joint (psi)		Fatigue Strength of Material (psi)		Ratio of Fatigue Strength: Material ÷ Joint		Fatigue Strength of Joint (Lb per In. of Width)		Static Strength of Joint (Lb per In. of Width)	Ratio of Strengths: Fatigue ÷ Static		Maximum Number of Cycles at Nominal Design Load
		Main Plates	Rivets	at 50,000 Cycles	at 1,000,000 Cycles	at 50,000 Cycles	at 1,000,000 Cycles	at 50,000 Cycles	at 1,000,000 Cycles	at 50,000 Cycles	at 1,000,000 Cycles		at 50,000 Cycles	at 1,000,000 Cycles	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
7	Single	H.T.S.	M.S.	18,200	8,100	50,000	37,400	2.74	4.61	2,700	1,180	9,940	0.272	0.119	23,000
8	Single	M.S.	M.S.	14,900	6,400	31.700	26,000	2.13	4.06	3,270	1,410	11,170	0.293	0.126	45,000
9	Double	H.T.S.	M.S.	22,700	11,800	50,000	37,400	2.20	3.17	3,290	1,710	10,080	0.327	0.170	48,000
10	Double	M.S.	M.S.	22,000	17,100	31,700	26,000	1.44	1.52	4,300	3,330	10,920	0.394	0.305	1,000,000
11	Single	53S-T	53S-W	10,000	5,200	23,100	17,300	2.31	3.32	3,350	1,730	10,080	0.332	0.172	52.000
12	Double	53S-T	53S-W	10,800	6,700	23,100	17,300	2.14	2.58	3,630	2,230	10,420	0.348	0.214	96,000

From R. L. Templin and E. C. Hartmann, *Static and Repeated Load Tests of Aluminum Alloy and Steel Riveted Hull Plate Splices*, Aluminum Company of America (1941), Technical Paper No. 5.
Note: Designation 53S-T and 53S-W have been changed to 6053-T4 and 6053-W.

As an example of the effect of a butt weld on the strength of the welded plates, tests* have shown that specimens having the welds machined flush with the welded plates have fatigue strengths from 12 to 30 per cent higher than similar specimens with the welds reinforced, or projecting beyond the surfaces of the plates. The same tests indicated that the specimens with welds machined flush with the plates developed fatigue strengths approximately equal to the fatigue strength of the plate without welds. Removal of mill scale was found to raise the fatigue strength of plates, whether welded or without welds. The results of these tests seem to indicate that the lowering of fatigue strength is largely due to the concentration of stress caused by surface irregularities.

The same series of tests indicated that the fatigue strengths developed by butt-welded plates were comparative to the fatigue strengths of the net cross sections of riveted plates, ranging from 2 to 10 per cent less for the welded plates. Progressive failure due to repeated loads usually appears first at the edge of any deposited weld metal in welded specimens and at rivet holes in riveted members-both at points of change in cross section.

Contrary to usual ideas, these tests did not indicate any increase in fatigue value for butt welds that had been stress relieved, possibly indicating that repeated application of loads tends to neutralize the welding stress.

A later series of tests† of butt-welded specimens prepared by various fabricators indicates the need for careful inspection and for experienced operators in welding operations. Several specimens failed in welds, and failure in each such case was traced to lack of fusion, blow holes, or slag inclusion. The variation in fatigue strengths was greatly amplified by poor workmanship.

A series of fatigue tests reported by the University of Illinois‡ on fillet- and plug-weld connections indicates a considerable reduction in the value of plates connected by fillet welds. Rapid change in section is the chief problem in the design of fillet-welded connections and is a subject for continuing study. Welded plate joints with longitudinal fillet welds along the sides and transverse fillet welds across the ends appeared to be as satisfactory as any type of welded connection tested in the series. The tests on plug-welded connections indicated that the fatigue strengths of the plug welds decrease, in pounds per square inch, as the size of the weld increases. For stress cycles varying from zero to tension in the plates, the shear fatigue strengths of the plug welds were found to be in the order of 20,000 to 25,000 psi for 100,000 cycles, based on the nominal area of the welds.

A report on the results of fatigue tests on welded connections, prepared

* Wilson, Bruckner, Coombe, and Wilde, *op. cit.*

† Wilson, Bruckner, McCrackin, and Beede, *op. cit.*

‡ Wilson, Bruckner, Dubery, and Beede, *op. cit.*

by Dr. Ing. Kommerell, Director of the German State Railways in 1935, appears to confirm the Illinois tests in that the fatigue strengths for butt welds were found to be much higher than for fillet welds. The important conclusions stated in the German report relating to the shape of fillet welds were:

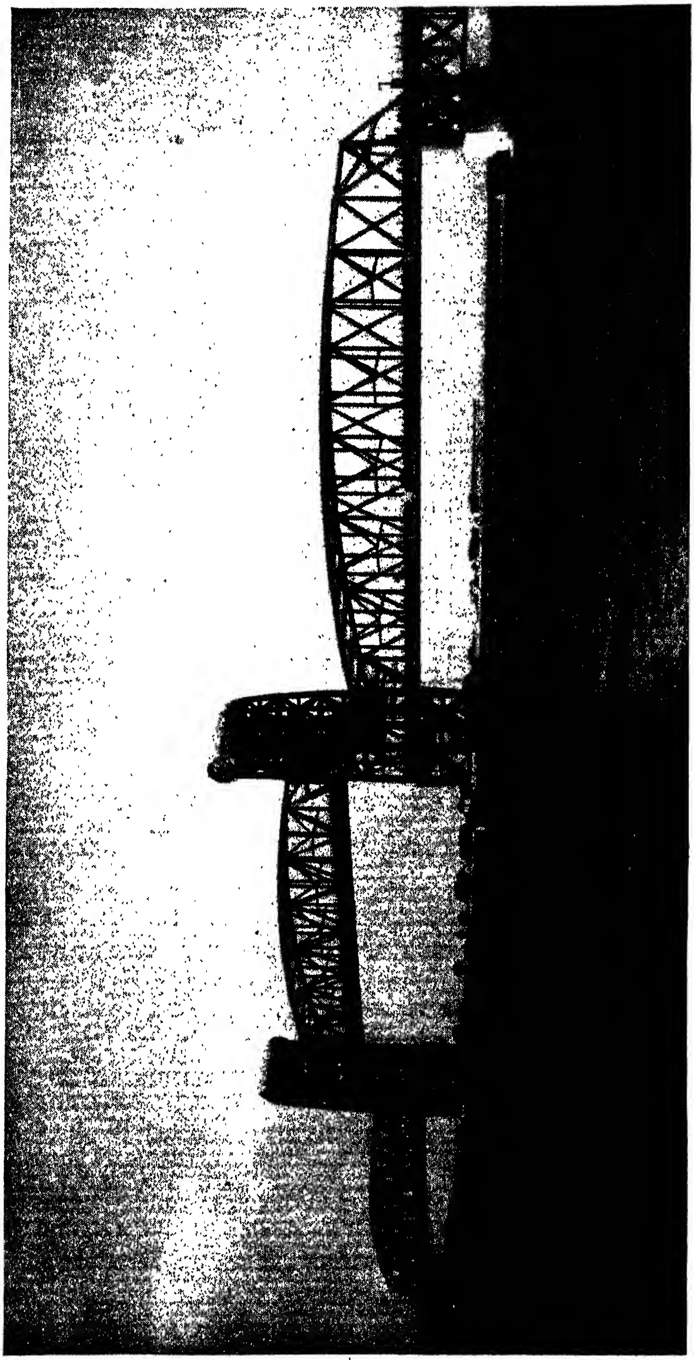
1. In front fillet welds, contrary to previous opinion, light fillet welds with a gradual transition of weld to plate proved better than full fillet welds.
2. If the transition of weld to plate and to the connected member is formed with a smooth curve at places where fillet welds begin and end, a considerably higher minimum non-alternating fatigue strength results.
3. The form of the weld, particularly at the transition between weld and plate, plays a decisive role and is much more important than the selection of the welding material used.*

10–7. Fatigue Considerations in the Design of Structures. Application of fatigue considerations to the design of structures has not developed to the point where it is a satisfactory science. Much information is needed before it can be considered proper to design with a low factor of safety against the fatigue limits of fabricated members. There are, however, several factors which should be considered in determining whether fatigue considerations need be included in the design.

There is a natural tendency to overestimate the number of cycles of high stress likely to occur in the lifetime of a structure. Although a member may receive a large number of repetitions of loading during its life, a considerable percentage of the loads may produce stress well below the design values. Allowances for impact may be excessive for many of the applications. A truss railroad bridge is one of the structures most commonly associated with fatigue considerations; such factors as the number of tracks, the number of trains each day, the speed of the trains, and the relation of that speed to the speed which is critical for impact, and the number of times each member is loaded and unloaded are important to the study. Certain members, such as chords, may be gradually loaded and unloaded on passage of the train, while floor members may receive relatively high load many times during the passage of the train.

Care should be taken in the design of members subjected to repeated loading to reduce eccentricities as much as possible and to include all stresses in the design computations. Design values permitted by specifications are, in general, low enough to provide for many repetitions of high stress. Stresses produced by continuity of members and secondary stresses caused by rigidity of joints are probably as important to the fatigue life of a structure as stresses computed without their consideration.

* Dr. Ing. Kommerell, "Results of Fatigue Tests on Welded Connections," translated from the *Publications of the International Association for Bridge and Structural Engineering*, Vol. 3 (1935), pp. 230–270.



American Bridge Co.

MARINE PARKWAY BRIDGE, NEW YORK.

Having the fatigue characteristics of the material to be used and having stress-concentration factors which are applicable to the case under consideration, a great deal can be done to safeguard the structure against failure by fatigue.

A typical problem in design for fatigue limitations is illustrated by Ex. 10–1.

EXAMPLE 10–1. The member to be designed is to be made of 2024-T4 aluminum alloy, and it is desired to provide a factor of safety of 2 for 1,000,000 cycles of repeated stress; the dead load on the member is found to be 50,000 lb direct tension, and the live load produces alternately 20,000 lb compression and 30,000 lb tension; there is a stress concentration factor of 2.0. It is required to determine the size of the member.

The maximum tensile stress in the member is

$$\frac{KP}{A} = \frac{2.0 \times 80{,}000}{A} = \frac{160{,}000}{A} \text{ psi},$$

and the minimum tensile stress is

$$\frac{2.0 \times 30{,}000}{A} = \frac{60{,}000}{A} \text{ psi}.$$

It is desired to know the maximum unit stress the material will withstand for 1,000,000 cycles, when the ratio of maximum to minimum stress is 160,000/60,000 = 2.67/1. Referring to Fig. 10–4, when the minimum direct stress is 15,000 psi tension, it is found that the material will withstand 1,000,000 cycles, including 51,000 psi as the maximum stress. The ratio of the maximum to the minimum stress in this case is 51,000/15,000 = 3.40. For a minimum unit stress of 20,000 psi, the maximum unit stress in the cycle is found to be 53,000, and the ratio of maximum to minimum is 53,000/20,000 = 2.65. A value for the ratio of 2.67 will fall between 15,000 psi and 20,000 psi; interpolating between these values,

$$\left(\frac{0.02}{0.75} \times 5{,}000\right) + 15{,}000 = 15{,}133 \text{ psi}.$$

Since a factor of safety of 2 is required, the minimum stress should not exceed 7,566 psi, and the maximum 2.67 × 7,566 = 20,200 psi. The maximum is less than 36,000 psi; hence, by a curve like that of Fig. 10–9 it is seen that the working unit stress will not be affected by redistribution of the stress concentration. The area required will be

$$A = \frac{KP}{f} = \frac{2.0 \times 80{,}000}{20{,}200} = 7.92 \text{ sq in.}$$

10–8. Studies in Process. Many tests of full-sized structural details are in progress. These tests include varying rivet patterns in riveted plates, combinations of plates with angles, channels, and beams, end-connection angles in beams under bending action, and various types of welds.

There are other directions for investigation which have been pursued but slightly. Fundamental research is needed in establishing stress-raising factors for various types of cross-sectional changes and combinations of those changes. These studies need to include conditions in which part of the critical section may have stress above the yield point of the material. When stress-raising factors are established for all probable types of cross-sectional changes and stress magnitudes, the transition from material to structure should be much easier.

The effects of varying low-stress values combined with higher-stress values have not been determined. Much fundamental research is necessary to establish endurance limits for the various materials, when 50 per cent of the loads produce 80 per cent of the yield-point stress and the remaining 50 per cent of the loads produce 60 per cent of the yield-point stress; when 60 per cent of the loads produce 70 per cent of the yield-point stress and the remainder, a lower stress, etc.

The missing fundamental data are too important in the evaluation of tests on built-up members to be neglected. Tests on members fabricated from combinations of structural shapes may show results that are quite consistent in any series, but which would change with a slight variation in fabrication.

PROBLEMS

10–1. Steel specimens tested as rotating beams were found to have an average endurance limit of 14,000 psi. Static tests of the same material indicated an ultimate tensile value of 64,000 psi and a yield-point stress of 32,000 psi. A tension member made of this material contains 4.0 sq in. and supports a dead load of 40,000 lb. (a) By use of a modified Goodman diagram, determine the magnitude of live load that may be applied to the member an indefinite number of times without failure. (b) What is the mean stress s_m for the cycle? (c) What is the alternating stress s_a?

10–2. From the data of Prob. 10–1 determine the value of s_m/s_u, and find the value of s_a from the Soderberg diagram.

10–i3. A member, subjected to direct stress without bending, is made of 2024-T4 aluminum alloy. If a tension load of 50,000 lb and a compression load of 50,000 lb are aternately applied, what size member will provide a factor of safety of 2 with respect to the fatigue limit for 10 million cycles of loading? Use the data of Fig. 10–4.

10–4. Direct-stress tests have determined that specimens of a certain steel plate will withstand 2 million cycles of alternate tension and compression of 26,000 psi. (a) Determine the anticipated fatigue limit for 2 million cycles when the compression load is twice the tension load, using the Moore formula of Art. 10–3. Hint: The value of $r = -2$. (b) What is the anticipated fatigue limit for this material for 2 million cycles when the load produces alternately tension and compression half as great?

10–5. A series of tests has established a fatigue limit of 30,000 psi for a given steel having 1 million cycles of loading producing stress varying from zero to tension. A specimen of the same material has a stress raiser with a concentration factor of 2.0.

What is the anticipated average stress at which this specimen will fail under 1 million cycles of load from zero to maximum tension?

10–6. Polished rotating-beam specimens have established that steel of a particular quality has fatigue values of 32,000 psi at 100,000 cycles and 24,000 psi at 2,000,000 cycles. If mill scale and surface irregularities are estimated to produce a concentration factor of $K = 1.2$, what size steel beam should be used for 400,000 in.-lb, applied 1,000,000 times, if a factor of safety of 2 is required? What is the factor of safety with respect to a yield-point stress of 34,000 psi? Hint: Plot the two given fatigue values on log-log paper, and determine the fatigue value for 1,000,000 cycles, which will be 2.4 times the working unit stress.

CHAPTER 11

THE RIGID FRAME

11–1. General Discussion. The term "rigid frame" has been commonly applied to a particular type of structure consisting of vertical and horizontal members rigidly attached at their intersections. While practically all building structures and many others have rigidly attached connections, the term "rigid frame" is more frequently used when referring to such a frame in which the effects of continuity are computed and a balanced design is carried out, providing for bending moments and forces resulting from such continuity.

Rigid frames are used for all types of structures, such as bridges, building frames, crane supports, and other industrial frames. The rigid frame is not restricted to single-span, single-story frames, nor to frames having particular shapes. The early conception of a rigid frame was largely restricted to a bridge having main carrying members with variable section and with single span. The concept has broadened to include such structures as multiple-story buildings with "moment connections" designed to take computed forces from wind loads and earthquake loads. Many frames have been designed to fit desired contours that are unsymmetrical and unusual.

11–2. Analysis of Rigid Frames. It is outside the scope of this text to present methods of analysis of rigid frames. Any textbook on statically indeterminate structures may be used as a reference.*

The frames used as examples in this text may be analyzed by any of the standard procedures, but for simplicity, the calculations for single-story, single-span frames are made by use of the three fundamental static equations, $\Sigma M = 0$, $\Sigma H = 0$, and $\Sigma V = 0$, and as many of the basic elastic equations as are required.

The elastic equations are:

1. $\Sigma(M/EI)\,ds = 0,$
2. $\Sigma(Mx/EI)\,ds = 0,$
3. $\Sigma(My/EI)\,ds = 0.$

Equation (1) is applicable between two points at which the frame is fixed in direction and states that the summation of the change of slope to the elastic curve between the two points is zero. Equation (2) is applicable when end "A" of the frame is fixed in direction and end "B" is supported to prevent vertical movement relative to end "A," when the mo-

* C. D. Williams, *Analysis of Statically Indeterminate Structures* (Scranton, Pa.: International Textbook Co., 1951), Ch. 4; Paul Andersen, *Statically Indeterminate Structures* (New York: The Ronald Press Co., 1953), Ch. 9.

ments of the M/EI area are taken about end "B." Equation (3) is applicable when the two ends of the frame are supported to prevent change in span length. In these equations M/EI is the average value of bending moment divided by E and I for any ds length, x is a horizontal ordinate for the ds length, and y is its vertical ordinate, measured to the line connecting the ends of the frame.

The single-span frame with hinged ends is singly indeterminate and requires but one elastic equation, $\Sigma(My/EI)\,ds = 0$, together with the static equations. When one end of the frame is fixed, the equation $\Sigma(Mx/EI)\,ds = 0$ is also required. In the case of a frame with both ends fixed, Equations (1) and (3) are required; if the loading is unsymmetrical or the frame is unsymmetrical, all three elastic equations must be employed.

In the application of the elastic equations, the value of M is usually considered to be composed of the moments resulting from the several static conditions into which the indeterminate condition may be divided. For example, the single-span frame with hinged ends is indeterminate in the first degree, when change of span length is prevented.

In the solution of this problem one static condition is the simple beam without restraint to change in span length. The effects of the force preventing change in span length are considered separately, with the results expressed in terms of the unknown H force. When the sum of these two values is set equal to zero, a value for H may be determined.

When the moment of inertia varies along the length of any member or when the member is curved, it is usually necessary to divide the member into a convenient number of ds lengths in order to evaluate the moment of the M/EI areas. When the frame is made up from straight members of constant section, the summations are simplified.

Example 11-1. The frame of Fig. 11-1(a) has hinged ends and is loaded with 80 lb per ft of length. The vertical legs are assumed to have a value of $I = 1{,}000$, and the horizontal span an $I = 2{,}000$. Find the value of H required to prevent change of span length, and the bending moments at the corners and center of the span.

The bending moment diagram that would exist when $H = 0$ is shown in Fig. 11-1(b), and the bending moments due to H are shown in (c). The evaluation of the elastic equation, $\Sigma(My/EI)ds = 0$, follows:

$$\Sigma \frac{My}{EI}\,ds = \frac{2 \times 4{,}000 \times 20 \times 12}{3 \times 2{,}000 \times E} = \frac{320}{E},$$

$$\Sigma \frac{(Hy)y}{EI}\,ds = \frac{12H \times 12 \times 8}{1{,}000E} + \frac{12H \times 20 \times 12}{2{,}000E} = \frac{2.592H}{E}.$$

Then,

$$\Sigma \frac{My}{EI}\,ds = \frac{320}{E} + \frac{2.592H}{E} = 0,$$

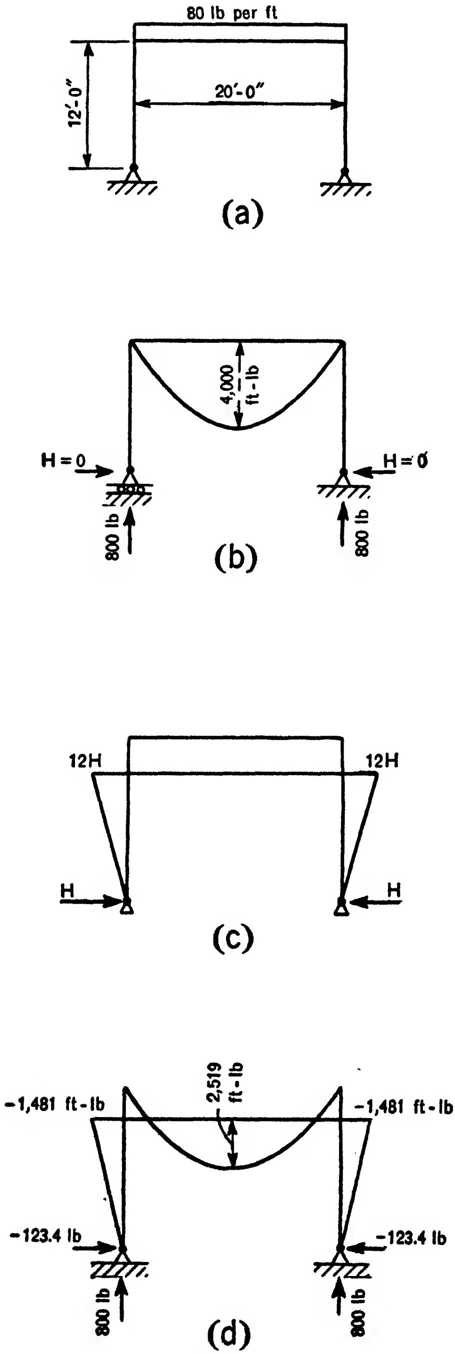
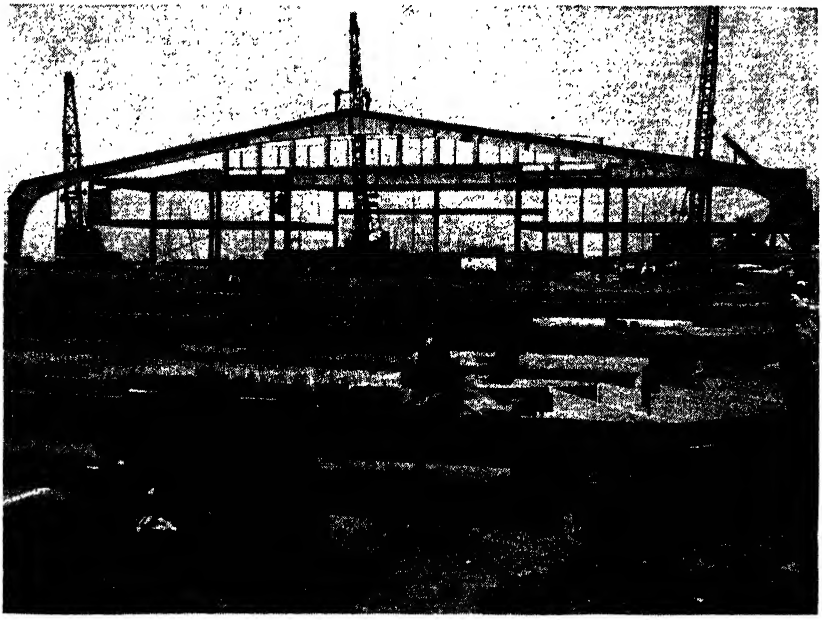

80 lb per ft
12'-0"
20'-0"
(a)
4,000 ft-lb
H = 0
H = 0
800 lb
800 lb
(b)
12H
12H
H
H
(c)
2,519 ft-lb
-1,481 ft-lb
-1,481 ft-lb
-123.4 lb
-123.4 lb
800 lb
800 lb
(d)

FIG. 11-1.



Lincoln Electric Co.

A 200-FT FRAME IN A HARBOR SHED IN LONG BEACH, CALIFORNIA

and $H = -123.4$ lb. The value of H is considered to be minus when it produces tension on the outside of the frame.

The bending moment at the corners of the frame is $12H$, or $12 \times -123.4 = -1{,}481$ ft-lb, and the moment at the center of the span is $4{,}000 - 1{,}481 = 2{,}519$ ft-lb. The resulting moments and reactions are shown in Fig. 11–1(d).

EXAMPLE 11–2. Assume that the ends of the frame of Fig. 11–1 are fixed in direction. Find the value of H, the end moments, the corner moments, and the maximum moment under the load.

Figure 11–2(a) shows the bending moment diagram resulting from application of bending moment at each end of the frame when no other forces are acting. Due to symmetry of loading, the end moments are known to be equal. Hence, only two elastic equations are required for the solution. These are $\Sigma(M/EI)ds = 0$, and $\Sigma(My/EI)\,ds = 0$. These equations are evaluated as follows:

From Example 11–1, $\Sigma \dfrac{M'y}{EI}\,ds = \dfrac{320}{E}$, and $\Sigma \dfrac{(Hy)y}{EI}\,ds = \dfrac{2.592H}{E}$.

From Fig. 11–2(a),

$$\Sigma \frac{M_A y}{EI}\,ds = \frac{M_A \times 12 \times 2 \times 6}{1{,}000E} + \frac{M_A \times 20 \times 12}{2{,}000E} = \frac{0.264M_A}{E}.$$

One equation is $$\Sigma \frac{My}{EI}\,ds = \frac{320}{E} + \frac{2.592H}{E} + \frac{0.264M_A}{E} = 0.$$

From Fig. 11–1(b), $$\Sigma \frac{M'}{EI}\,ds = \frac{2 \times 4{,}000 \times 20}{3 \times 2{,}000 \times E} = \frac{26.67}{E}.$$

From Fig. 11-1(c), $$\Sigma \frac{(Hy)y}{EI} ds = \frac{12H \times 12}{1{,}000E} + \frac{12H \times 20}{2{,}000E} = \frac{0.264H}{E}.$$

From Fig. 11-2(a), $$\Sigma \frac{M_A}{EI} ds = \frac{24M_A}{1{,}000E} + \frac{20M_A}{2{,}000E} = \frac{0.034M_A}{E}.$$

Then,

$$\Sigma \frac{M}{EI} ds = \frac{26.67}{E} + \frac{0.264H}{E} + \frac{0.034M_A}{E} = 0;$$

and from the two equations, $H = -208.4$ lb, $M_A = +833$ ft-lb, the corner moment is $(12 \times -208.4) + 833 = -1{,}667$ ft-lb, and the moment under the load is $4{,}000 - 1{,}667 = 2{,}333$ ft-lb.

Figure 11-2(b) shows the bending moments and reactions.

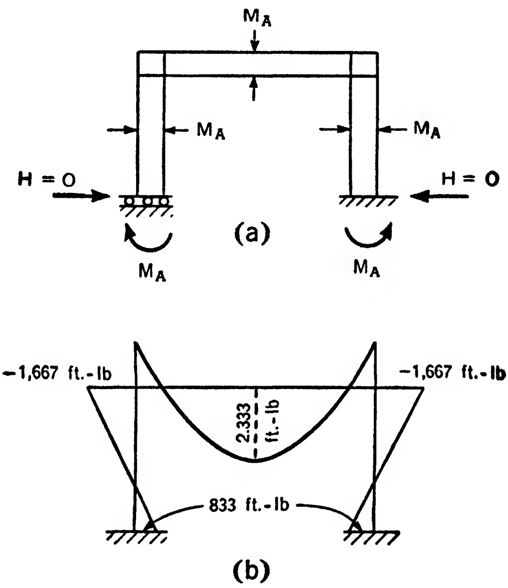


FIG. 11-2.

The same general procedure may be used for single-span frames of any shape or condition of loading. In case of unsymmetrical loading or unsymmetrical construction, the third equation, $\Sigma(Mx/EI)ds = 0$, must also be used.

11-3. Fixed and Hinged Ends. The supports for rigid frames should be designed to be either fixed or hinged. If the attachment of the frame to the footing is not positively hinged, a moment will develop, and the connection and footing should provide for this moment.

Comparison of Exs. 11-1 and 11-2 indicates that the effect of fixing the base was to reduce the maximum positive moment by less than 10 per cent and to increase the corner moment by less than 13 per cent. In some cases the effect of fixing the base is still smaller.

The design of hinged supports is simple and usually results in simplifying the design of the footings. Figure 11-3 shows a type of hinge that is used

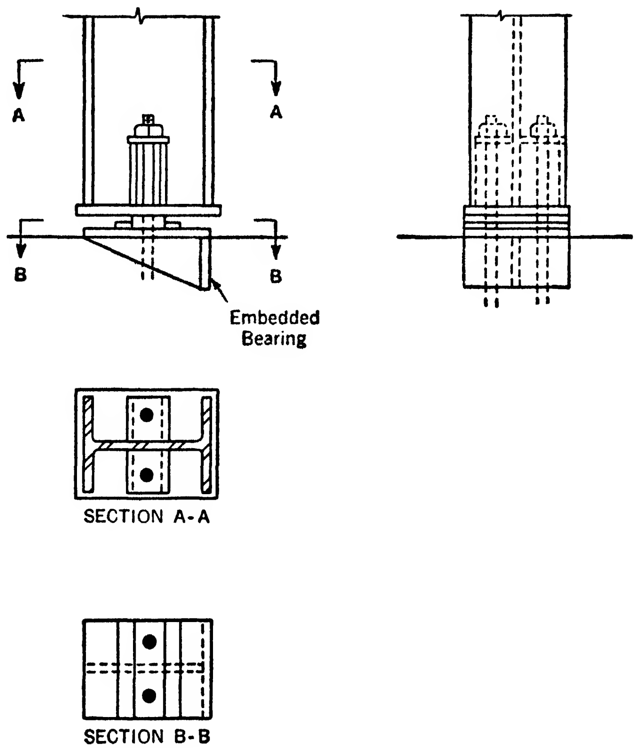
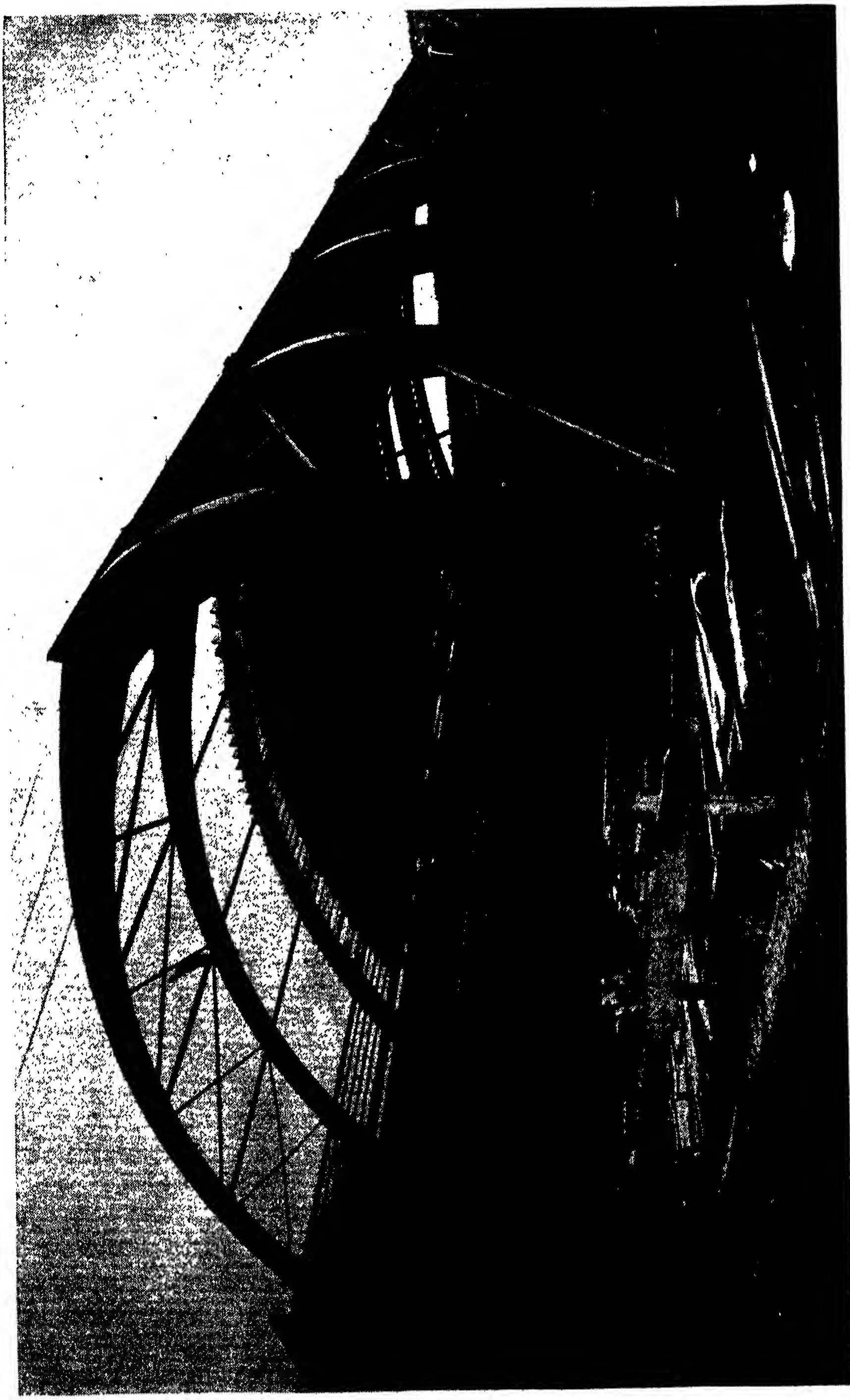


FIG. 11-3.

extensively for the bases of rigid frames. It provides a definite support condition that does not require an assumption of unknown end conditions.

If the footing is designed in a manner that will not permit rotation, and the base of the frame is attached in a manner to develop the induced end moment, there can be no objection to the use of fixed ends—other than, possibly, less economy. The column base may be designed as outlined in Chap. 8. It should be remembered that a flat column base is fixed until the amount of the end moment is sufficient to cause it to tip. Placing anchor bolts in the corners of the base plate will increase the resisting moment up to the limit of the value of the bolts. When the anchor bolts are placed at the center of the base plate, their effective lever arm is simply reduced. Balanced design does not permit any material to be overstressed.

11-4. Procedure in Design. In the design of any statically indeterminate structure it is necessary to make assumptions relative to the probable sections or relation between sizes of sections before an analysis can be started. When an analysis of the trial design has been made, a closer



Lincoln Electric Co

VARIABLE SECTION WELDED RIGID FRAME FOR A CHURCH analysis is possible. The design resulting from the second trial is usually the final design. Knowledge of this procedure will assist the designer to gage the degree of accuracy that must be applied to the computations.

In many cases the trial section can be selected as a frame composed of members of equal size and constant section. This greatly simplifies the first analysis. Sections selected from these results will usually be close to the final sections. In some cases it will be known that certain members will be heavier than others, and this fact can be employed in selection of the trial section. In other cases it may be desirable to select trial sections on the basis of some empirical relationship, knowing that the end moments for horizontal spans will be somewhat less than the fixed-end moments for the same span and loads, and that the center moment will be somewhat larger than for a beam with fixed ends.

It should be remembered in the analysis that it is the relative stiffness of the several members in a frame that affects the distribution of moments, rather than the actual sizes. The moment of inertia of all members can be increased or decreased in the same ratio without affecting the moments and reactions, except for the effect of the weight of the members.

Variable section and built-up members cost more per unit weight than rolled beams. Unless enough weight can be saved to offset the difference in unit cost the variable section may not be economical.

Limiting dimensions for shipments must be observed. Field splices should be located and designed to facilitate ease in erection.

11–5. Design of a Building Frame. The required dimensions for a building frame are shown in Fig. 11–4(a). The roof load is 30 psf live load and 20 psf dead load, including the purlins but not the weight of the frames. The frames are to be spaced at 20 ft 0 in. center-to-center. Wind loads are to be in accordance with the ASCE recommendation (Chap. 1). The frames are to be hinged at the bottom and the footings will provide for the horizontal as well as the vertical reactions.

The dead load of 20 psf will result in a uniform vertical load of 400 lb per lin ft of frame. Figure 4(b) shows the bending moment diagram for this part of the dead load, with the assumption that there is no force preventing change of the span length of the frame. The moments in the vertical legs would be zero under this condition, and the inclined beams would have the simple beam bending moments. The maximum amount of the bending moment at the center of the span would be,

$$\frac{wL^2}{8} = \frac{400 \times 60 \times 60}{8} = 180{,}000 \text{ ft-lb.}$$

It is assumed that the inclined beams will be constant-section rolled beams. The vertical legs will also be rolled beams, but it is assumed that they will need to be larger than the inclined beams. For the trial section

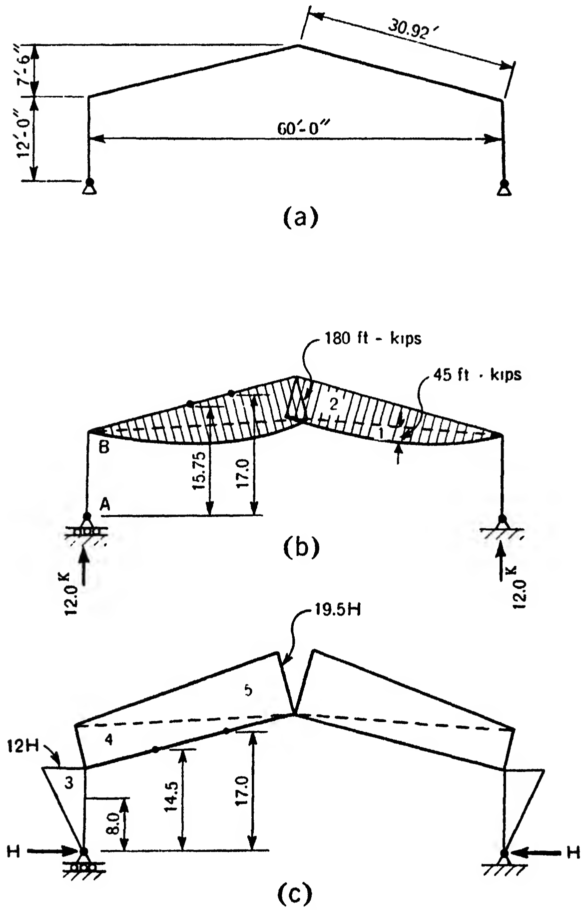


FIG. 11–4a, b, c.

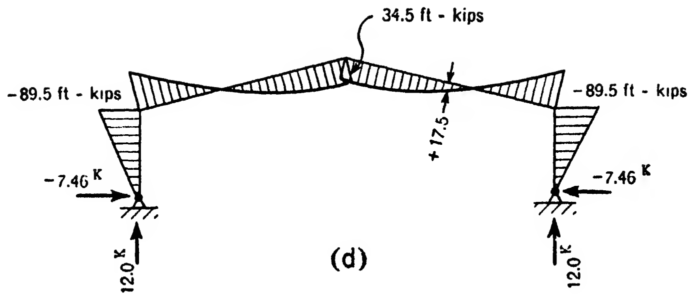
it is assumed that the inclined beams have an $I = 1.0$ and that the vertical legs have an $I = 1.2$.

As shown in Fig. 11–4(*b*) the bending moment diagram is divided into two parts, a triangle and a parabola. The value of $\Sigma(My\ ds/I)$ for this portion of the moment diagram is computed as follows:

No.	*Area*	*I*	$(M\,ds)/I$	*y*	$My\,ds$
1	$\frac{2}{3} \times 45 \times 30.92 = 927.6$	1.0	927.6	15.75	14,609
2	$\frac{1}{2} \times 180 \times 30.92 = 2{,}782.8$	1.0	2,782.8	17.00	47,308
					61,917
					2

$$\Sigma \frac{M'y}{I}\,ds = 123{,}834$$

Figure 11–4(*c*) shows the bending moments that would result from a force H applied horizontally at the reaction points with no other forces



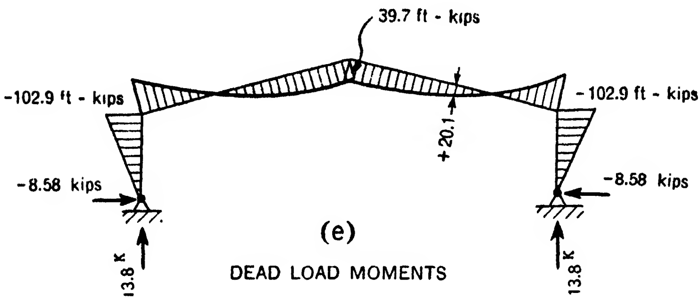


FIG. 11–4d, e.

acting. The bending moment diagram is divided into three triangles, and the required value of $\Sigma(My\,ds)/I$ for this portion of the bending moment is:

No.	*Area*	*I*	*(My ds)/I*	*y*	*(My ds)/I*
3	$6H \times 12 = 72.00H$	1.2	$60.00H$	8.0	$480H$
4	$6H \times 30.92 = 185.52H$	1.0	$185.52H$	14.5	$2{,}690H$
5	$9.75H \times 30.92 = 301.47H$	1.0	$301.47H$	17.0	$5{,}125H$
					$8{,}295H$
					2

$$\Sigma \frac{(Hy)y}{I}\,ds = 16{,}590H$$

The bending moments shown in Fig. 11–4(*b*) cause the frame to increase in span length, while those in (*c*) result in a reduction in span length of the same amount. The conditions of the problem are that the sum of the two changes in span length will result in zero change. Then

$$123{,}834 + 16{,}590H = 0.$$

$$H = -7.46 \text{ kips.}$$

$$12H = 12(-7.46) = -89.5 \text{ ft-kips (at corner)}$$

$$180 + 19.5H = 180 + 19.5(-7.46) = 34.5 \text{ ft-kips (center)}$$

The resulting bending moments and reactions are shown in Fig. 11–4(*d*). The negative sign for H indicates that it produces a bending moment resulting in tension on the outside of the frame, since the known moment was called *positive* when it caused tension on the inside.

The results of the foregoing computations can be used to calculate the moments and reactions for the live load when placed on the entire span. For a vertical load of 30 psf, the moments and reactions will each be 1.5 times the values obtained for the 20-psf dead load. Similarly, the moments and reactions due to the weight of the frame may be computed by a direct ratio.

Figure 11–5(a) shows the live load placed on one half of the span. This condition is investigated to determine whether a larger moment is pro-

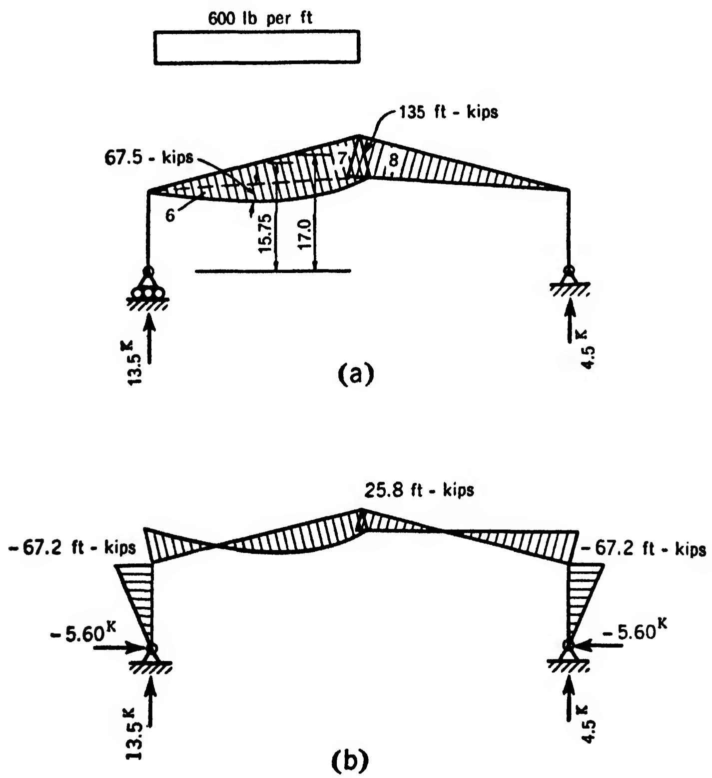


FIG. 11–5. Moments from unsymmetrical live load.

duced at any point than would be produced with load on the entire span. The reactions will be: $(30 \times 0.6) \times \frac{3}{4} = 13.5$, and $18.0 - 13.5 = 4.5$ kips. The bending moment at the center, when $H = 0$, will be $4.5 \times 30 = 135$ ft-kips. On the left side, the point of zero shear is $13.5/0.6 = 22.5$ ft from the support (horizontal). The bending moment at this point is $(13.5 \times 22.5) - (13.5 \times 11.25) = 151.9$ ft-kips. The moment diagram is divided into three figures, and the maximum ordinate to the parabola is $151.9 -$

[(22.5/30) × 135] = 50.6 ft-kips. The center of the parabola is at the midheight of the inclined member. Then the required value of $\Sigma[(My\,ds)/I]$ would be:

No.	*Area*	*I*	*(M ds)/I*	*y*	*(My ds)/I*
6	$\frac{2}{3} \times 67.5 \times 30.92 = 1{,}391$	1.0	1,391	15.75	21,908
7	$\frac{1}{2} \times 135 \times 30.92 = 2{,}087$	1.0	2,087	17.00	35,479
8	$\frac{1}{2} \times 135 \times 30.92 = 2{,}087$	1.0	2,087	17.00	35,479
				$\Sigma \frac{M'y}{I}\,ds =$	92,866

The value of $\Sigma[(Hy)y\,ds/I]$ was found previously. Then

$$92{,}866 + 16{,}590H = 0.$$

$$H = -5.60.$$

$$12H = 12(-5.60) = -67.2 \text{ ft-kips (at corner).}$$

$$135 + 19.5H = 135 + 19.5(-5.60) = 25.8 \text{ ft-kips (at center).}$$

Figure 11–5(*b*) shows the moments and reactions that would result from live load placed on one half of the span.

Assuming that the inclined beams will weigh 60 lb per ft, the dead-load moments and reactions will be 460/400, and the live-load moments and reactions will be 600/400 of the values shown in Fig. 11–4(*d*). The dead-

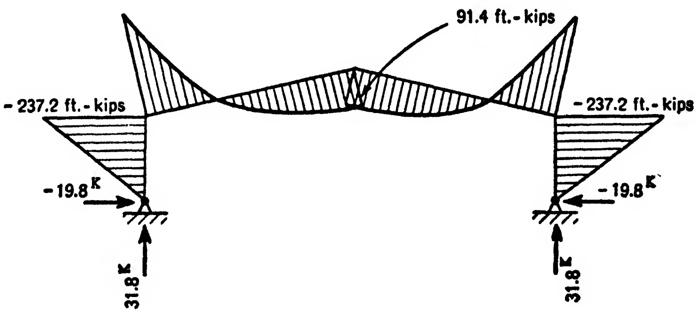


FIG. 11–6. Moments from dead load and symmetrical live load.

plus live-load moments and reactions are shown in Fig. 11–6 as 2.65 times the values in Fig. 11–4(*d*).

The inclined beam must be designed for the direct force component of reactions *H* and *V* and the bending moment at any point along its length. The vertical leg must resist the vertical reaction and bending moment.

Figure 11–7 shows three types of haunch detail that are frequently used. In Fig. 11–7(*a*) the maximum bending moment in the inclined beam occurs

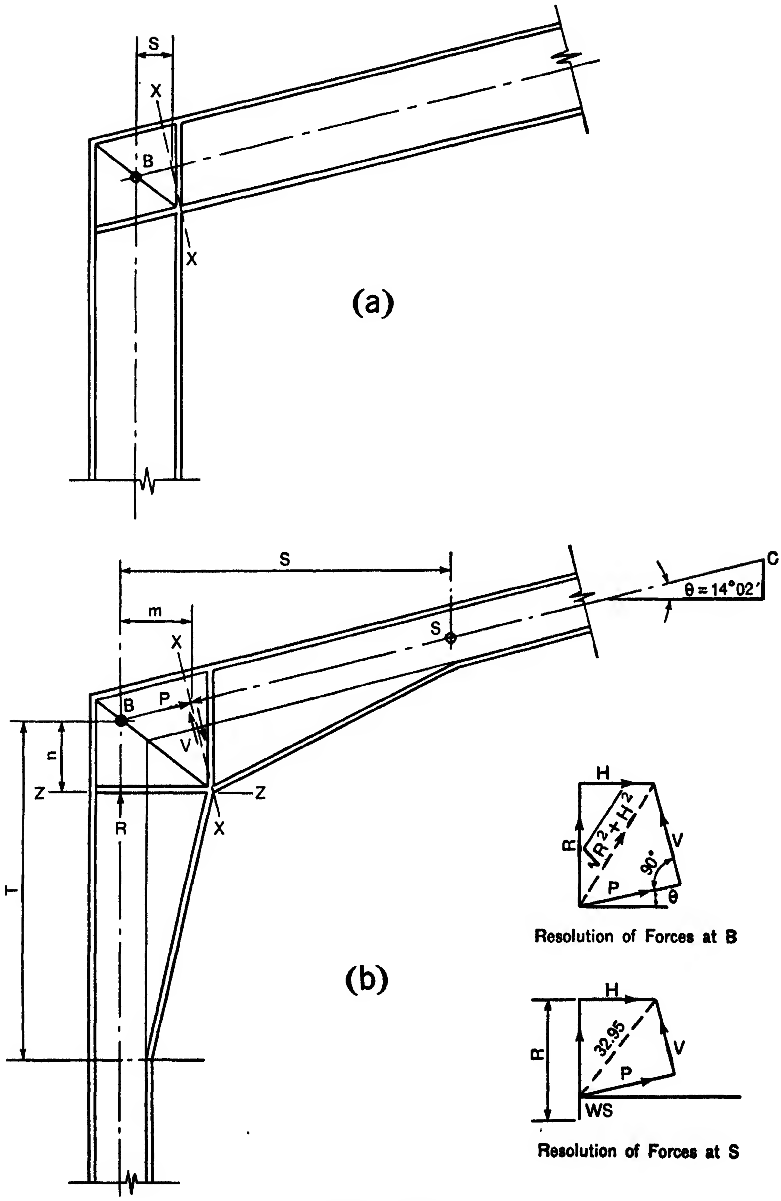

S
X
B
X
(a)
S
m
X
S
θ = 14°02′
C
B
P
V
n
Z
Z
R
X
T
(b)
H
R
$\sqrt{R^2+H^2}$
V
90°
P
θ
Resolution of Forces at B
H
R
32.95
V
P
WS
Resolution of Forces at S

FIG. 11-7a, b.

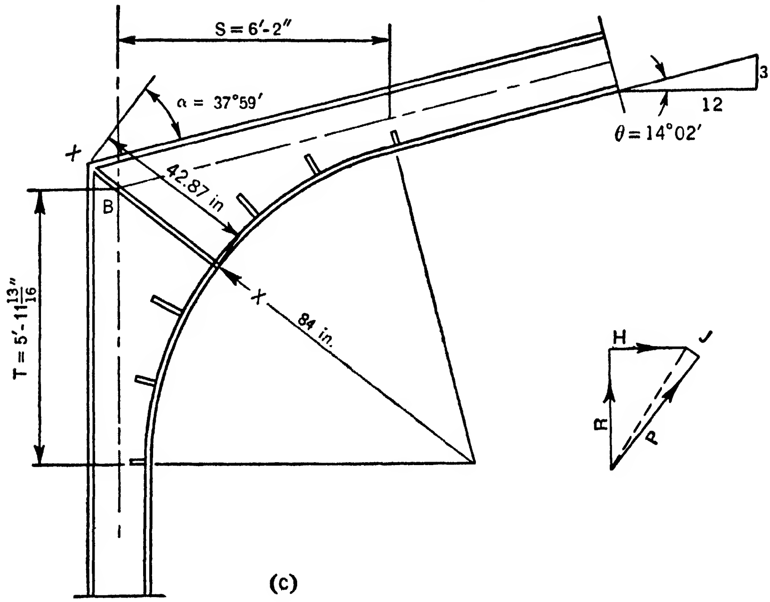


FIG. 11-7c.

at section X–X at distance S from the corner B. In (b) the web of the inclined beam is split, the bottom flange bent, and the space in the web filled with a triangular butt-welded plate. In this case the distance S is selected so that the section at that point will be the same as is required at the center of the span. At section X–X the beam depth is increased to provide a larger section. The forces R and H may be resolved at point B into two components, P and V, with P acting along the line BC and V normal to the line BC. The bending moment at section X–X is then the moment at B reduced by an amount Vm, a force P eccentric with respect to the gravity axis, and a shear V. The bottom flange is sloped and proper correction in the stress must be made. (See Chap. 4.)

With any of the haunch details shown, there should be a strut bracing the compression flange at a point near the corner B. When the strut is provided, the unsupported length of the bottom compression flange is the distance from this brace to the point of contraflexure, or the distance to the base of the frame. The horizontal distance from B to the point of contraflexure can be found by plotting the moment diagram to scale or from the equation of bending moment,

$$M = Hy + \frac{wx}{2}(L - x) = 0.$$

For the frame being analyzed, $y = 12 + (15x/L)$, $w = 1.06$ kips, and $L = 60$ ft.

Then $$M = -19.8\left(12 + \frac{x}{4}\right) + 0.53x(60 - x) = 0,$$

and $x = 11.43$ ft.

Assuming that a 16W 40 may be used for the inclined member, the allowable compression at section X–X or S–S may be determined.

$$\frac{Ld}{bt} = \frac{11.43 \times 12 \times 16}{7 \times 0.503} = 623, \quad \text{and} \quad F_b = \frac{12{,}000{,}000}{623} = 19{,}260 \text{ psi.}$$

Also,

$$F_a = 17{,}000 - 0.485\frac{(11.43 \times 12)^2}{1.5 \times 1.5} = 12{,}940 \text{ psi.}$$

At the center of the span, point C, there is a force of $P = H \cos \theta = 19.8 \times 0.99 = 19.6$ kips and a bending moment of 91.4 ft-kips. The unit stresses are

$$\frac{19{,}600}{11.77} \pm \frac{91{,}400 \times 12}{64.4} = 18{,}700 \text{ psi compression, and } 15{,}360 \text{ psi tension.}$$

The compression flange is assumed to be continuously supported by a rigid roof. The allowable compression will be $f_a = 17{,}000$ and $f_b = 20{,}000$ psi. Investigation of a 16W 36 shows that it is not adequate.

The point where the negative moment is equal to the moment at C is where $M = Hy + [(wx/2)(L - x)] = -91.4$ ft-kips, or

$$-19.8\left(12 + \frac{x}{4}\right) + 0.53\, x\, (60 - x) = -91.4, \quad \text{and} \quad x = 5.16 \text{ ft.}$$

If S in Fig. 11–7(b) is 5.16 ft, the bending moment at this point will be -91.4 ft-kips and the forces P and V may be found by resolution of the forces R, H, and ws ($= 5.16 \times 1.06$) to be $P = 25.6$ and $V = 20.7$ kips. Then,

$$\frac{P}{A} = \frac{25{,}600}{11.77} = 2{,}180 \text{ psi,}$$

$$\frac{M}{S} = \frac{91{,}400 \times 12}{64.4} = 17{,}030 \text{ psi,}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{2{,}180}{12{,}940} + \frac{17{,}030}{19{,}260} = 1.05.$$

Since the section satisfies the allowable stress within 5 per cent, it may be considered satisfactory, but by making the distance $S = 5$ ft 6 in., there will be no doubt.

Assuming that the section X–X is 32 in. deep, the properties of the right section can be computed.

	A	y	$Ay^2 + I_0$
Flanges, $2 \times 7 \times 0.503 =$	7.04 in.^{2}	15.75	1,747 in.^{4}
Web, $31 \times 0.307 =$	9.52	..	762
	16.56 in.^{2}		$I =$ 2,509 in.^{4}

$$\text{Section modulus} = \frac{2{,}509}{16} = 156.8 \text{ in.}^3$$

RIGID FRAME FOR A CHURCH STRUCTURE *Lincoln Electric Co*

The distance $m = 1.5$ ft and the bending moment at this point is

$$M = -19.8\left(12 + \frac{1.5}{4}\right) + (0.53)1.5(60 - 1.5) = -198.5 \text{ ft-kips.}$$

By resolution of the forces R, H, and wm ($= 1.5 \times 1.06$), the values of $P = 26.4$ kips, and $V = 24.4$ kips, are found. Then,

$$\frac{M - Pe}{S} = \frac{(198{,}000 \times 12) - (26{,}400 \times 8)}{156.8} = 13{,}840 \text{ psi,}$$

and

$$\frac{P}{A} = \frac{26{,}400}{16.56} = 1{,}600 \text{ psi.}$$

The stresses in the compression flange are increased because of the slope of the flange.

Thus,
$$f_a = 1.03 \times 1{,}600 = 1{,}650 \text{ psi},$$
$$f_b = 1.03 \times 13{,}840 = 14{,}250 \text{ psi},$$
$$\frac{f_a}{F_a} + \frac{f_b}{F_b} + \frac{1{,}650}{12{,}940} + \frac{14{,}250}{19{,}260} = 0.87 < 1.00.$$

By trial it may be found that a 28-in. depth of section at X–X would be overstressed. It is better to keep the unit stress low at this point; therefore the 32-in. section will be used.

Figure 11–7(*c*) shows an alternate solution for the haunch. The distance S is located in the same manner as before. The center of curvature for the inside flange is located so that the curve will be tangent to the vertical and inclined beams. Usually the radius is given in a convenient even dimension and the tangent points computed to agree. Assuming that the radius of the curve is 7 ft 0 in., the length T will be 5 ft $11\frac{13}{16}$ in., and S will be 6 ft 2 in. The depth of the section X–X will be 42.87 in.

The section X–X is acted upon by the corner moment of -237.2 ft-kips and the normal and direct components of R and H. These forces may be resolved into $V = 12.9$ kips and $P = 35.2$ kips applied at point B.

Since the section X–X is located on a curved portion of the structure, the stress cannot be computed as $(P/A) + (Mc/I)$. Rather, the analysis must be made by consideration of the basic assumptions of stress analysis, i.e., a plane section before bending remains plane after bending, and stress and strain are proportional.

When a section under consideration is normal to a straight gravity axis, the length of any fiber is considered to be unity, and the magnitude of stress due to bending is proportional to the distance from the gravity axis because the deformation is proportional to the distance of the fiber from the gravity axis. If F is the force on any fiber and L is its length, the deformation of the fiber is FL/AE. When the area of the fiber is dA and when its deformation is directly proportional to y and inversely proportional to E, then $FL/dA\,E \propto y/E$, and $F \propto y\,dA/L$. When L is unity, $F \propto y\,dA$. The moment of the force in the fiber is $Fy \propto y^2\,dA$, when y is measured from the gravity axis, and the moment of all the forces $\doteq \Sigma y^2\,dA$, which is called the *moment of inertia*. The unit stress on any fiber is then $f = My/I$. A direct force applied at the gravity axis deforms all fibers the same amount so that the stress on any fiber is $(P/A) \pm (My/I)$.

A section subjected to pure bending must have the sum of compressive forces equal to the sum of the tensile forces. Then, for a section of a straight beam, the $\Sigma y\,dA$ above the neutral axis must equal the $\Sigma y\,dA$ below the neutral axis, or the neutral axis will be at the gravity axis.

In the case of a curved beam, the lengths of fibers on the cross section are not equal and the force on any fiber is $F \propto y\,dA/L$. In the case of pure bending, the sum of compressive and tensile forces must be equal; then the $\Sigma(y\,dA/L)$ above the neutral axis must equal the $\Sigma(y\,dA/L)$

below the neutral axis. The neutral axis for bending is then at the centroid of the $\Sigma(dA/L)$ or $\bar{y} = [\Sigma(y\,dA/L)]/\Sigma(dA/L)$. The moment of all forces about the neutral axis is $Fy \propto \Sigma(y^2\,dA/L)$, when y is measured from the neutral axis. The force on any fiber at distance y, being directly proportional to y and inversely proportional to the length of the fiber L, becomes

$$f_b = \frac{My}{L\Sigma\dfrac{y^2\,dA}{L}}.$$

A direct force P, applied at the centroid of the $\Sigma(dA/L)$, will then deform any fiber in direct proportion to its length. The unit stress at any point due to direct force will then be inversely proportional to L and will be

$$f_a = \frac{P}{L\Sigma\dfrac{dA}{L}}.$$

It will be noted that the direct force must be applied at this centroid to deform all fibers in proportion to their lengths.

The most convenient method for computing the properties of a section on a curved beam, corresponding to area and moment of inertia, is to divide the cross section into a convenient number of divisions. Figure 11–8(a) shows the effective cross section at X–X of the curved haunch. Figure 11–8(b) shows the relative lengths of the fibers. When the length of the fiber at the bottom of the section is called ΔL, the length of a fiber

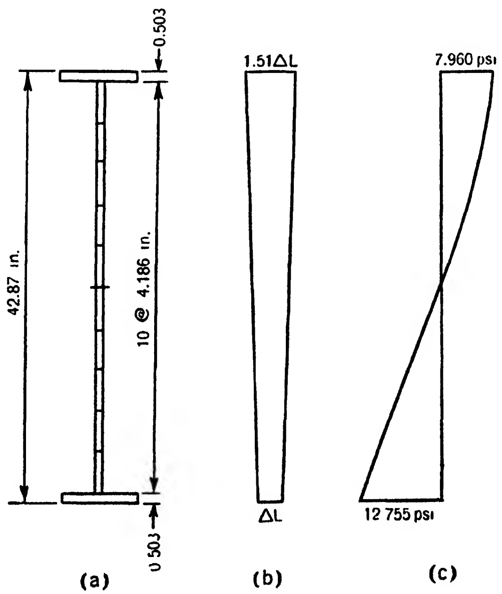


FIG. 11–8.

at the outside flange will be

$$\frac{126.87\Delta L}{84} = 1.51\Delta L.$$

The properties that are required are computed in the following:

Δy	ΔA	y	L	$(\Delta A)/L$	$(y\,\Delta A)/L$	$(y^2\,\Delta A)/L$
0.503	3.521	0.25	1.003	3.510	0.878	
4.186	1.285	2.60	1.031	1.246	3.240	8
4.186	1.285	6.78	1.081	1.188	8.055	55
4.186	1.285	10.97	1.131	1.136	12.462	137
4.186	1.285	15.15	1.180	1.089	16.498	250
4.186	1.285	19.34	1.230	1.045	20.210	391
4.186	1.285	23.53	1.280	1.004	23.624	556
4.186	1.285	27.72	1.330	0.966	26.778	742
4.186	1.285	31.90	1.379	0.932	29.731	948
4.186	1.285	36.09	1.429	0.899	32.445	1,171
4.186	1.285	40.27	1.479	0.869	34.995	1,409
0.503	3.521	42.62	1.507	2.336	99.560	4,243
				16.220	308.476	9,910
					19.02 × 308.476 =	5,867
						4,043 in.⁴

$$\bar{y} = \frac{308.476}{16.22} = 19.02 \text{ in.}$$

Resolution of forces R and H at point B gives values of $P = 35.2$ kips and $V = 12.9$ kips. The bending moment at B is 237.2 ft-kips and the force P is eccentric 12.54 in. The stresses in the top fiber based on the right section will be

$$f_a = \frac{35{,}200}{1.51 \times 16.22} = 1{,}440 \text{ psi},$$

$$f_b = \frac{[(237{,}200 \times 12) - (35{,}200 \times 12.54)]\,23.85}{1.51 \times 4{,}043} = 9{,}400 \text{ psi}.$$

The total tension on the right section is $9{,}400 - 1{,}440 = 7{,}960$ psi, and the tension on the extreme fiber because of the slope of the flange with respect to the right section is $7{,}960 \sec 37°59' = 7{,}960 \times 1.27 = 10{,}100$ psi.

At the bottom fiber the bending stress will be

$$f_b = \frac{[(237{,}200 \times 12) - (35{,}200 \times 12.54)]\,19.02}{4{,}043} = 11{,}315 \text{ psi}.$$

The allowable axial compression is based on r, which varies along the length of the haunch. The least value is

$$r = \sqrt{\frac{26.5}{19.89}} = 1.16 \text{ and } F_a = 17{,}000 - 0.485\,\frac{(11.43 \times 12)^2}{1.16 \times 1.16} = 10{,}220 \text{ psi}.$$

F_b has not been reduced because of stiffeners and strut provided at the haunch.

Then,
$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{1{,}440}{10{,}220} + \frac{11{,}315}{19{,}260} = 0.72 < 1.00.$$

Figure 11–8(c) shows the distribution of stress on the right section at X–X.

In the haunches of Fig. 11–7(a) and (b), stiffeners should be provided to transfer the compression flange force into the web. The area of each pair of stiffeners should provide bearing area enough and should be attached with sufficient weld to transfer the full compression. For example, the bottom flange on section X–X of (b) was found to have a compressive stress of $14{,}250 + 1{,}650 = 15{,}900$ psi at the extreme fiber. The horizontal component of the force in the flange is then a little less than $(7.00 \times 0.503)(15{,}900) \cos \theta$, or $3.521 \times 15{,}900 \times 0.97 = 54{,}300$ lb. The bearing area required will be $54{,}300/27{,}000 = 2.00$. Two 3 x ½-in. stiffeners will provide a bearing area on the flange of $2(3.00 - 0.50)\,0.50 = 2.50$ sq in. The average shear on the column web is $19{,}800/(31 \times 0.307) = 2{,}080$ psi, leaving $13{,}000 - 2{,}080 = 10{,}920$ psi shear value that may be utilized in the welded connection. (See Chap. 5.) The maximum effective size of each weld on the web will then be $(0.307 \times 10{,}920)/(0.707 \times 13{,}600) = 0.35$ in. Six pairs of 5⁄16-welds 1½ in. long on each bar will provide a value of $4 \times 6 \times 1.5 \times 3{,}000 = 108{,}000$ lb, or more than sufficient to transfer the load.

The compression flange of the curved haunch of Fig. 11–7(c) should be stiffened to prevent local buckling of the flange. It has been found that stiffeners spaced at a maximum distance equal to about three times the width of the flange will be effective. These stiffeners need extend only about one-third of the depth of the web and may be made triangular. The top tension flange should have a stiffener at the bend point adequate to transfer the diagonal component of the flange stress into the web. The compression in this stiffener will be equal to

$$2(3.521 \times 10{,}100) \sin 37°59' = 71{,}120 \times 0.615 = 43{,}740 \text{ lb.}$$

Two 3 x ⅜-in. stiffeners will provide a bearing value of

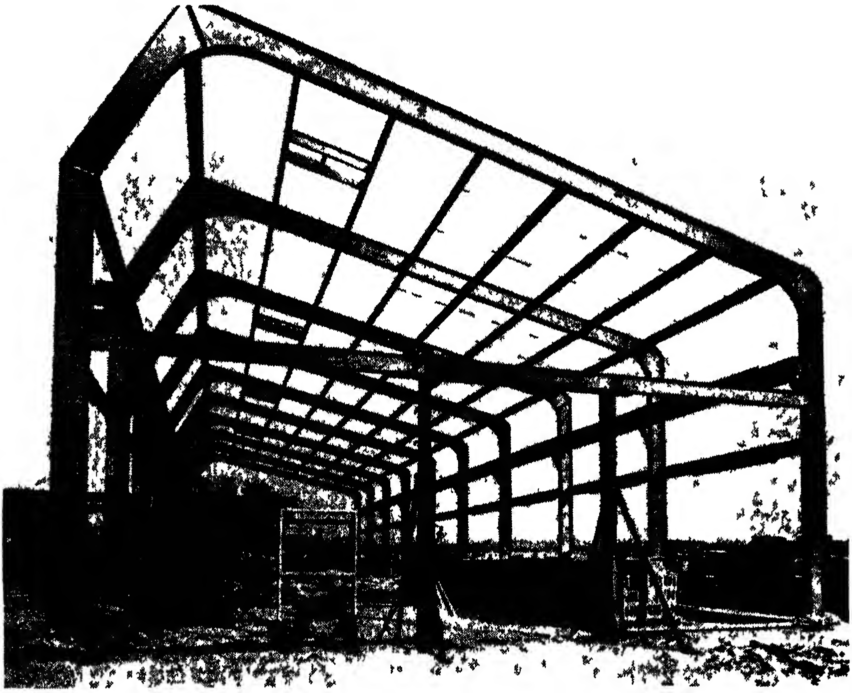
$$2(3.00 - 0.50) \times \tfrac{3}{8} \times 24{,}000 = 45{,}000 \text{ lb.}$$

Intermittent 5⁄16-in. welds, spaced at 6 in., will be adequate for the attachment of the stiffeners.

The vertical leg of the frame will have its maximum stress at the bottom of the haunch. In the case of the haunch of Fig. 11–7(c) the bending moment at this point will be $-19.8(6.01) = -119.0$ ft-kips and the direct force will be 31.8 kips. Assuming that the vertical member is a 16WF 50, the allowable axial stress is $17{,}000 - 0.485[(144/1.54)^2] = 12{,}750$ psi. The allowable stress in the compression flange due to bending is 24,000 psi, by special provision in the AISC specification Section 15(a)(3).

A 16WF 50 has an area of 14.70 and a section modulus of 80.7. Then $f_a = 31{,}800/14.70 = 2{,}160$ psi, $f_b = (119{,}000 \times 12)/80.7 = 17{,}705$ psi, and the value, $f_a/F_a + f_b/F_b = (2{,}160/12{,}750) + (17{,}705/24{,}000) = 0.91 < 1.00$.

The value of I for the 16WF 50 is 655 4, and the I for the inclined 16WF 40 is 515 5 The ratio of the I values is 1 27, which is near enough to the ratio of 1 20 used in the computation of bending moments so that a new calculation of moments and shears is not required



Lincoln Electric Co

WELDED RIGID FRAME FOR CROWN IRON WORKS IN MINNEAPOLIS

The Code permits increases in the allowable design stresses when wind forces are considered in combination with other forces. Computation of the wind bending moments on this frame shows that the increase in moment is less than would be permitted by the increase in unit stresses.

11–6. The Rigid Frame Railroad Bridge. In the example that follows, an application of the 1946 AREA specifications is made to a single-span hinged-end rigid frame. The structure is designed to carry a ballasted single track and Cooper's E-72 loading. Such "unit type" bridges have been constructed frequently with plate girders for the main carrying elements supported on abutments. Railroad tracks are usually spaced 13 ft center-to-center, and the clearance diagram would not permit any part of the structure to be much above rail height without the necessity of spreading the tracks farther than 13 ft.

Unit-type bridges are designed so that any number may be placed side by side to provide for multiple tracks. The details of ballast floors used by the various railroads vary greatly. The details used in this example are similar to those used by one of the railroads on a large number of bridges. Such bridges are usually used to carry the railroad over a street or highway. In such cases it is usually desirable to make certain that the floor is watertight. In the design that is followed in this example, the concrete floor is waterproofed on the top side, and a nonstructural floor plate is also provided under the slab.

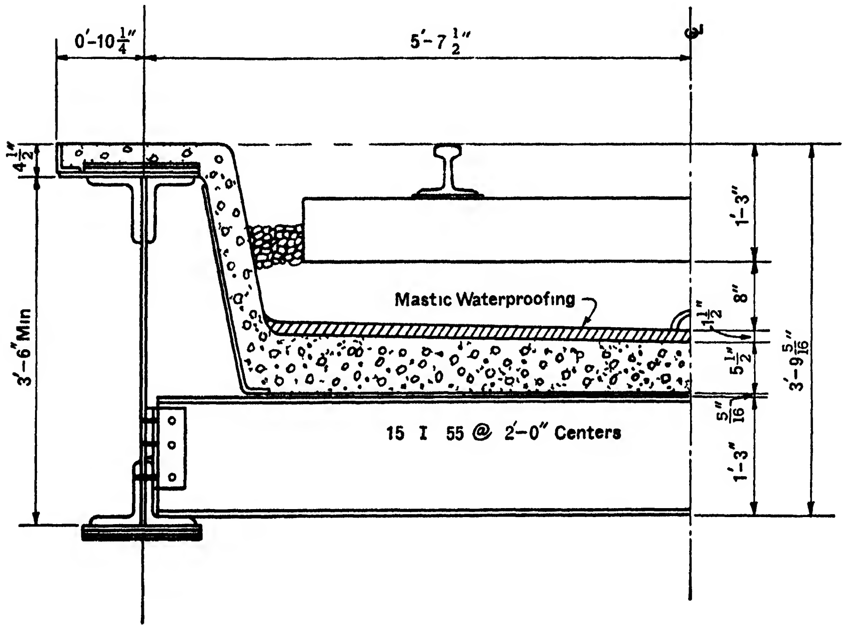


FIG. 11-9.

Figure 11-9 shows a tentative cross section for the bridge structure. It is assumed that the supporting frames may be 11 ft 3 in. center-to-center and still leave room for the top flange detail. Eight inches of ballast is provided at the center of the track under the ties, 1½ in. is allowed for waterproofing and 5½ in. for concrete.

11-7. Design of the Deck. It is assumed that the floorbeams are to be spaced 18 in. center-to-center. They are simple beams 11 ft 3 in. long. An estimate of the weight to be carried by the floorbeams, exclusive of their own weight, follows:

Track	=	200 lb per ft of track
Ballast, $9.0 \times 1.0 \times 120$	=	1,080
Mastic, $9.0 \times 0.12 \times 150$	=	162
Concrete, $10 \times 0.54 \times 150$	=	810
Floor plate, 10×12.8	=	128
		2,380 lb per ft of track

$$\text{Dead load per foot of beam, } \frac{2{,}380 \times 1.5}{10.5} = 340 \text{ lb}$$

$$\text{Assumed weight of beam per foot} = \underline{50}$$

$$\text{Total dead load} = 390 \text{ lb per ft of beam}$$

Paragraph 204 of the AREA specification indicates that the distribution of live load on transverse beams is $P = KAd/S$, in which A = axle load, P = load on beam from one track, d = beam spacing, S = axle spacing, and $K = 1\frac{1}{8}$ for a single track. Hence,

$$P = \frac{9 \times 1.5 \times A}{8 \times 5} = 0.338A.$$

The axle load for Cooper's E-72 is 72,000 lb, and $P = 0.338 \times 72{,}000 = 24{,}350$ lb. The impact load for maximum moment on the floorbeam is $100 - 0.6L = 93.25$ per cent. The student should refer to Art. 6–7 and the 1948 AREA specifications for changes in the impact allowance.

The bending moment on the floorbeam is then

$$(340 \times 5.25)(5.62 - 2.62) = 5{,}360 \text{ ft-lb, dead load}$$

$$\frac{50 \times 11.25^2}{8} = 800 \text{ ft-lb, dead load of beam}$$

$$\frac{24{,}350}{2}(5.62 - 2.62) = 36{,}525 \text{ ft-lb, live load}$$

$$0.9325 \times 36{,}525 = \underline{34{,}060} \text{ ft-lb, impact}$$

$$\text{Total bending moment} = 76{,}745 \text{ ft-lb}$$

The section modulus required is

$$S = \frac{76{,}745 \times 12}{18{,}000} = 51.2 \text{ in.}^3$$

A 12 I 55 provides an S of 53.2 in.^{3}, but the 15 I 55 is more economical. The 15-in. beam provides an $S = 67.8$ in.^{3}, and the beams may be spaced farther than 18 in. The beam spacing may be $(67.8/51.2) \times 18 = 23.8$ in. Since no value was given to the cover plate except to prevent buckling of the top flanges of the beams, a spacing of 24 in. should be satisfactory.

The sections considered in this example were standard sections prior to 1946 but have become obsolete with the 1948 edition of *Steel Construction;* use of other available sections would result in changes in the beam spacing.

With the revised spacing of 24 in.,

$$P = \frac{KAd}{S} = \frac{9 \times 2 \times A}{8 \times 5} = 0.45A,$$

or $0.45 \times 72{,}000 = 32{,}400$ lb. In computing the end reaction on the floorbeam, the rolling effect of the locomotive must be included. With a spacing of 5 ft for the rails and 11 ft 3 in. for the girders, the rolling impact to each girder will be $(20 \times 5)/11.25 = 8.9$ per cent. The total impact will then be $93.2 + 8.9 = 102.1$ per cent.

The end reaction for a floorbeam will be

Dead load above floorbeam	=	2,380 lb
Weight of floorbeam	=	300
Live load, $\frac{32,400}{2}$	=	16,200
Impact, $1.021 \times 16,200$	=	16,540
Total end reaction	=	35,420 lb

This end reaction requires $35,420/8,120 = 4$ 1—⅞-in. rivets in single shear.

11–8. Frame Trial Section. The problem selected for consideration is a railroad bridge over a street. The street is 80 ft between curbs and has a 4 ft 9 in. sidewalk on each side, thus a clear span of 89 ft 6 in. is required. It is assumed that the span length from back-to-back of the angles is 90 ft. The highway underclearance is 14 ft. The clearance diagram is shown in Fig. 11–10.

In order to calculate bending moments and thrusts, it is necessary to start with a trial section. Such a trial section may be selected from exist-

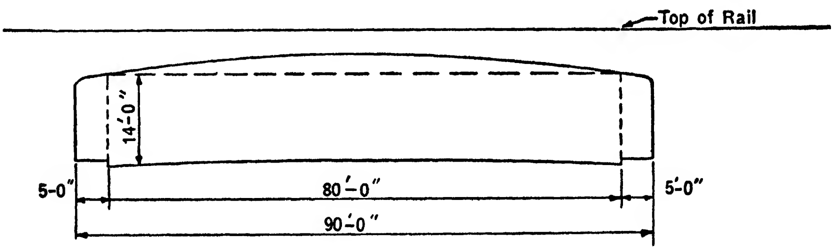


Fig. 11–10.

ing designs, but, in the absence of such information, a trial section may be prepared on the assumption that the horizontal span has fixed ends and is of constant section. Such an approximate preliminary design would assume that the section is constant and would not include the effects of temperature or forces other than live load, dead load, and impact. A third method of arriving at a trial section is to make use of influence lines for a similar frame, such as shown in Fig. 11–13. The values of bending moments obtained by use of these influence lines can then be used for a trial section.

To obtain a clear span of 90 ft will require a span of about 92 ft center-to-center of supports. With a uniform load on a fixed-end beam of constant section, the center moment is $wL^2/24$ and the end moment $wL^2/12$. The impact allowance for direct vertical effect is $100 - 0.6(92) = 44.8$

per cent. The rolling effect is $(20 \times 5)/11.25 = 8.9$ per cent, and the total impact is $44.8 + 8.9 = 53.7$ per cent. The live-load moment for Cooper's E-72 on a simple span of 92 ft, taken from tables of Cooper's loadings, is 4,997,900 ft-lb. The center moment for a fixed-end span is assumed to be one-third of the simple moment, and the end moment twice the center moment. The dead load of the girder is assumed at 400 lb per ft, and the total load at 1,700 lb per ft. Then

$$
\begin{aligned}
&\text{Live-load bending moment, } \tfrac{1}{3} \times 4{,}997{,}900 &&= 1{,}665{,}970 \text{ ft-lb}\\
&\text{Impact bending moment, } 0.537 \times 1{,}665{,}970 &&= \phantom{0{,}}894{,}630\\
&\text{Dead load, } \frac{1{,}700 \times 92^2}{24} &&= \underline{\phantom{0{,}}599{,}530}\\
&\qquad\qquad\text{Total center moment} &&= 3{,}160{,}130\\
& && \ \underline{\phantom{3{,}160{,}13}2}\\
&\qquad\qquad\text{Total end moment} &&= 6{,}320{,}260 \text{ ft-lb}
\end{aligned}
$$

Study of Fig. 11–9 indicates that the back-to-back depth at the center of the span should be about 42 in. Assuming that the back-to-back depth is equal to the effective depth of the section at this point, the flange area required will be

$$\frac{3{,}160{,}130 \times 12}{42 \times 18{,}000} = 50.1 \text{ sq in.}$$

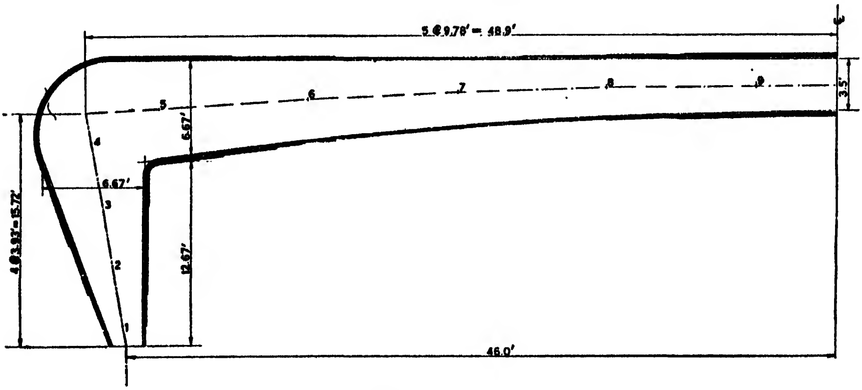


Fig. 11–11.

Approximately 50 per cent of the flange area may be in the cover plates. A trial flange section will then be made up from

$$
\begin{aligned}
&2\text{—}8 \times 6 \times 1 \text{ Ls} &&= 26.00 \text{ sq in. (short legs outstanding)}\\
&1\text{—}14 \times 2 \text{ Cov. Pl} &&= \underline{28.00}\\
& && \ 54.00 \text{ sq in.}
\end{aligned}
$$

Considering that the bending moment diagram is parabolic in shape, the bending moment at the face of the support may be computed from the end moment and the center moment as

$$\left(\frac{45}{46}\right)^2(6{,}320{,}260 + 3{,}160{,}130) - 3{,}160{,}130 = 5{,}912{,}550 \text{ ft-lb.}$$

Based on equal unit stresses, the depth of the section at the face of the vertical support should be

$$\frac{5{,}912{,}550}{3{,}160{,}130} \times 42 = 78.5 \text{ in.}$$

The depth of the horizontal section at the top of the vertical leg is made equal to the depth of the horizontal span at the face of the support. At the bottom of the vertical leg, the depth of the section need be only nominal, permitting the execution of the detail shown on Fig. 11–18. The vertical leg has a uniformly varying depth. Figure 11–11 shows the geometry of the trial frame.

It is certain that not all of the cover plates will be needed at the bottom of the vertical leg. In the trial section it is assumed that one-fourth of the total cover plate area is added in each space between the working points 1 to 4.

11–9. Properties of the Trial Frame. In the tables that follow, the gross moments of inertia are computed for all divisions of the trial frame, and the properties are listed for convenience in computations. The center line of the frame is divided into a convenient number of divisions, and the points at which properties are computed are the centers of each division. (See Table 11–1.)

11–10. Influence Lines. When bending moments are to be computed for several positions of live load, it is frequently advantageous to develop influence lines. Influence lines can be developed for a single-span frame with

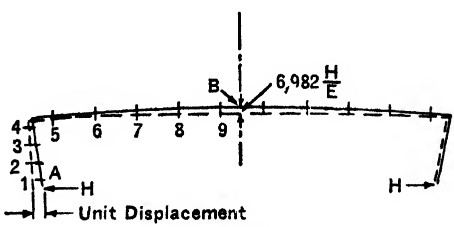


FIG. 11–12.

hinged ends with a few rather simple computations. Use may be made of Maxwell's Law of Reciprocal Displacements as applied to deflection, together with the moment-area principles. Maxwell's law states that "if a load P placed at point A produces deflection D at point B, the same load P placed at point B will produce deflection D at point A."

TABLE 11-1

MOMENT OF INERTIA—TRIAL SECTION

Point	1	2	3	4	5	6	7	8	9	Centerline
Web Pl	32 × 0.5	50 × 0.5	67 × 0.5	84 × 0.5	79 × 0.5	64 × 0.5	53 × 0.5	45 × 0.5	42 × 0.5	42 × 0.5
Cov. Pl (14 × ½)	2	4	6	8	8	4	4	6	8	8
b–b of Ls	32.64	50.16	67.56	84.96	79.16	64.08	53.28	46.08	42.48	42.00
Web *I*	1,365	5,208	12,532	24,696	20,543	10,923	6,203	3,797	3,087	3,087
I of Fl Ls	323	323	323	323	323	323	323	323	323	323
$52y^2$ (Ls)	9,717	26,161	50,392	82,494	70,919	44,916	29,927	21,619	17,971	17,510
I of Cov. Pl	3,844	18,321	50,077	105,869	92,217	29,648	20,624	23,770	27,699	27,104
Total *I* (in.^{4})	15,249	50,013	113,324	213,382	184,002	85,810	57,077	49,509	49,080	48,024
I (ft^4)	0.74	2.41	5.47	10.29	8.87	4.14	2.75	2.39	2.37	2.32
Gross area	82	105	127.5	150	147.5	112	106.5	116.5	129	129

Referring to Fig. 11–12, assume that a load F is applied horizontally at point A in proper magnitude to cause unit horizontal deflection at A. If the vertical deflection at point 8 is d when F is applied at point A, then a vertical load of F at point 8 will produce a horizontal deflection d at point A. Assume that a force H is required at point A to return point A to its original position by deflecting it a distance d. Then the ratio d to the unit deflection is the same as the ratio of the loads producing the deflections, or $d = H/F$, and $H = Fd$. This means that the deflection curve has ordinates equal to the ordinates of an influence line for the horizontal reaction at A when the deflection curve is caused by a unit displacement horizontally at A. The problem is to develop the deflection curve for the points along the horizontal span caused by a horizontal unit displacement of the point A. This can be accomplished by use of the properties of the trial section.

In the computations that follow, the vertical deflection of each point in the frame is computed with respect to a horizontal line through the centerline of the frame (point B), in terms of an unknown value of H. Using the moment-area principle, the deflection of any point from the tangent to the elastic curve at another point is equal to the moment of the M/EI area between the two points about the point for which deflection is desired.

The bending moment at any point due to a horizontal load H at point A is Hy. The area of the M/EI diagram for any section is then $Hy(ds/EI)$. The horizontal displacement of point A is the moment of the M/EI area between the hinge points with vertical lever arms, or

$$\sum Hy^2\frac{ds}{EI} = 9{,}423.16\frac{H}{E}\cdot$$

(See Table 11–2.)

TABLE 11–2

PROPERTIES OF TRIAL SECTION

Point	ds (ft)	I (ft⁴)	$\frac{ds}{I}$	y (ft)	$\frac{y\,ds}{I}$	$\frac{y^2ds}{I}$	x (ft)
1	3.93	0.74	5.31	1.96	10.41	20.40	−0.36
2	3.93	2.41	1.63	5.89	9.60	56.55	−1.09
3	3.93	5.47	0.72	9.82	7.07	69.43	−1.82
4	3.93	10.29	0.38	13.75	5.22	71.78	−2.54
5	9.78	8.87	1.10	16.04	17.64	283.01	1.99
6	9.78	4.14	2.36	16.66	39.32	655.02	11.77
7	9.78	2.75	3.56	17.11	60.91	1,042.17	21.55
8	9.78	2.39	4.09	17.41	71.21	1,239.77	31.33
9	9.78	2.37	4.13	17.56	72.52	1,273.45	41.11
					293.90	4,711.58	
						2	
						9,423.16	

The vertical deflection of any point from the tangent to the elastic curve at point B is then the moment of the M/EI area between point B and the point for which the deflection is desired, with horizontal lever arms measured from the point at which deflection is desired. In the calculations that follow, it is found that the end deflects vertically $6{,}982H/E$ upward with respect to the tangent through point B. It then follows that the point B deflects $6{,}982H/E$ downward from an unstressed position when the ends are in the unstressed position. Column 1 of Table 11-3 shows the deflection of each point upward with respect to the tangent through point B. Column 2 shows the vertical deflection of each point downward with respect to original position, and Column 3 shows the ordinates to the

TABLE 11-3

	Col. 1	Point	Col. 2	Col. 3
		9	$6{,}982H$	−0.741
$72.52H \times 9.78 =$	$709H$	8	$6{,}273H$	−0.666
$71.21H$				
$143.73H \times 9.78 =$	$1{,}406H$			
$60.91H$				
	$2{,}115H$	7	$4{,}867H$	−0.516
$204.64H \times 9.78 =$	$2{,}001H$			
$39.32H$				
	$4{,}116H$	6	$2{,}866H$	−0.304
$243.96H \times 9.78 =$	$2{,}386H$			
$17.64H$				
	$6{,}502H$	5	$480H$	−0.051
$261.60H \times 4.53 =$	$1{,}185H$			
$5.22H$				
	$7{,}687H$	4		−2.278
$266.82H \times -0.72 =$	$-192H$			
$7.07H$				
	$7{,}495H$	3	Col. 3 is the influence ordinate for force H	
$273.89H \times -0.73 =$	$-200H$			
$9.60H$				
	$7{,}295H$	2		
$283.49H \times -0.73 =$	$-207H$			
$10.41H$				
	$7{,}088H$	1		
$293.90H \times -0.36 =$	$-106H$			
	$6{,}982H$	End		

influence line for force H. The values of Column 3 are found by dividing the values of Column 2 by $-9{,}423.16H$. The term E is dropped from the computations because it cancels out in each case.

The bending moment at any point is equal to the simple moment on the 92-ft simple span combined with the Hy moment. In Table 11-4, the computed simple moment for each point in the frame with a 1-lb load

TABLE 11-4

Point	y	1-Lb Load at Point 5			1-Lb Load at Point 6			1-Lb Load at Point 7			1-Lb Load at Point 8			1-Lb Load at Point 9			Σ
		Hy	M'	$M'+Hy$	Hy	M'	$M'+Hy$	Hy	M'	$M'+Hy$	Hy	M'	$M'+Hy$	Hy	M'	$M'+Hy$	
1	1.96	−0.10	−0.35	−0.45	−0.60	−0.31	−0.91	−1.01	−0.28	−1.29	−1.30	−0.24	−1.54	−1.45	−0.20	−1.65	−5.84
2	5.89	−0.30	−1.07	−1.37	−1.79	−0.95	−2.74	−3.04	−0.83	−3.87	−3.92	−0.72	−4.64	−4.36	−0.60	−4.96	−17.58
3	9.82	−0.50	−1.78	−2.28	−2.99	−1.59	−4.58	−5.07	−1.39	−6.46	−6.54	−1.20	−7.74	−7.28	−1.01	−8.29	−29.35
4	13.75	−0.70	−2.49	−3.19	−4.18	−2.21	−6.39	−7.10	−1.95	−9.05	−9.16	−1.68	−10.84	−10.19	−1.41	−11.60	−41.07
5	16.04	−0.82	1.95	1.13	−4.88	1.74	−3.14	−8.28	1.52	−6.76	−10.68	1.31	−9.37	−11.89	1.10	−10.79	−28.93
6	16.66	−0.85	1.73	0.88	−5.06	10.27	5.21	−8.60	9.02	0.42	−11.10	7.76	−3.31	−12.34	6.51	−5.83	−2.66
7	17.11	−0.87	1.52	0.65	−5.20	9.02	3.82	−8.83	16.50	7.67	−11.40	14.20	2.80	−12.68	11.92	−0.76	14.18
8	17.41	−0.89	1.31	0.42	−5.29	7.77	2.48	−8.98	14.20	5.22	−11.60	20.65	9.05	12.90	17.33	4.43	21.60
9	17.56	−0.90	1.10	0.20	−5.34	6.51	1.17	−9.06	11.91	2.85	−11.69	17.31	5.62	−13.01	22.74	9.73	19.57
9R	17.56	−0.90	0.89	−0.01	−5.34	5.26	−0.08	−9.06	9.62	0.56	−11.69	13.99	2.30	−13.01	18.37	5.36	8.13
8R	17.41	−0.89	0.68	−0.21	−5.29	4.01	−1.28	−8.98	7.33	−1.65	−11.60	10.66	−0.94	−12.90	14.00	1.10	−2.98
7R	17.11	−0.87	0.47	−0.40	−5.20	2.76	−2.44	−8.83	5.04	−3.79	−11.40	7.33	−4.07	−12.68	9.63	−3.05	−13.75
6R	16.66	−0.85	0.25	−0.60	−5.06	1.51	−3.55	−8.60	2.75	−5.85	−11.10	4.00	−7.10	−12.34	5.26	−7.08	−24.18
5R	16.04	−0.82	0.04	−0.78	−4.88	0.25	−4.63	−8.28	0.47	−7.81	−10.68	0.68	−10.00	−11.89	0.89	−11.00	−34.22
4R	13.75	−0.70	−0.05	−0.75	−4.18	−0.33	−4.51	−7.10	−0.59	−7.69	−9.16	−0.86	−10.02	−10.19	−1.13	−11.32	−34.29
3R	9.82	−0.50	−0.04	−0.54	−2.99	−0.23	−3.22	−5.07	−0.43	−5.50	−6.54	−0.62	−7.16	−7.28	−0.81	−8.09	−24.51
2R	5.89	−0.30	−0.02	−0.32	−1.79	−0.14	−1.93	−3.04	−0.26	−3.30	−3.92	−0.37	−4.29	−4.36	−0.49	−4.85	−14.69
1R	1.96	−0.10	......	−0.10	−0.60	−0.05	−0.65	−1.01	−0.08	−1.09	−1.30	−0.12	−1.42	−1.45	−0.16	−1.61	−4.87

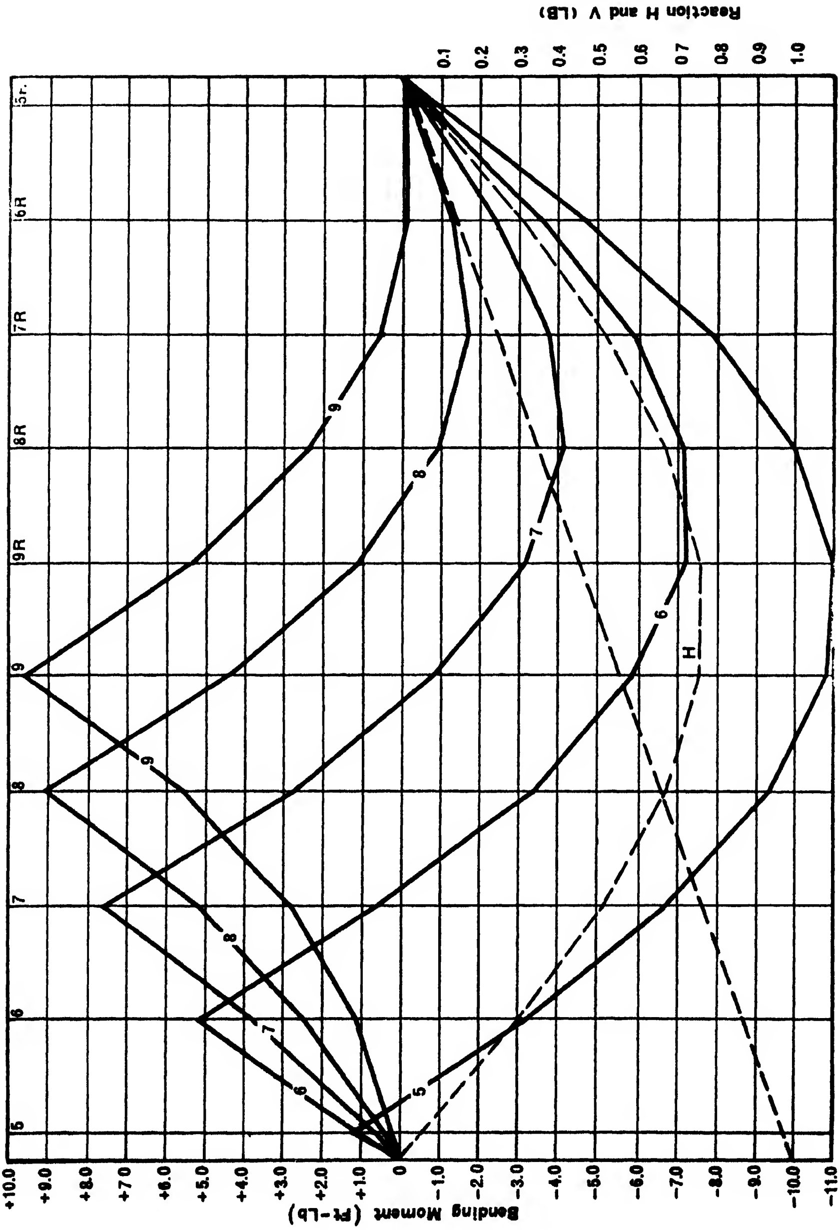


FIG. 11-13. Influence lines.

placed first at point 5, than at point 6, etc., is shown as M'. The values of $M' + Hy$ are the ordinates to influence lines for bending moments. The influence lines are plotted in Fig. 11–13. The influence line ordinates are shown as equal to zero over the supports because no live load can be applied between point 5 and the end support.

11–11. Live-Load Bending Moments. Examination of the influence lines for bending moment shows that live load may cause either positive or negative bending at any point, depending on its position on the span. At some points the amount of the live-load bending will be sufficient in magnitude to reverse the direction of the dead-load moment. Hence, it is necessary to investigate both positive and negative bending at several points in the frame.

Stress in the horizontal span will be a combination of bending stress and direct stress—the direct stress caused by the force H. For each value of bending moment, a simultaneous value for H must also be found. In Fig. 11–13 an influence line for H is plotted. When the position of live load producing the maximum moment is found, the ordinates to the influence line for H may also be scaled, and a value for H may be found. A maximum value for H must be found in order to find the maximum moments in the vertical leg, and a simultaneous value for the vertical reaction will be needed. It is also necessary to have a maximum vertical reaction for the design of the end bearing. It is found that one position of the live load produces both maximum vertical reaction V and maximum horizontal reaction H.

The computations that follow show the position of the live load used, the amount of bending moment, and the simultaneous H-force for each point in the horizontal span. Both positive and negative moments are found for points 6, 7, and 9, where reversal might be expected.

LIVE-LOAD MOMENTS

Point 9: Wheel No. 13 at point 9, train headed toward left.

Wheel	Load	Ord.	M	Wheel	Load	Ord.	H
7........	23.4	0.42	9.8	7......	23.4	−0.104	− 2.43
8........	23.4	1.00	23.4	8......	23.4	−0.265	− 6.20
9........	23.4	1.80	42.1	9......	23.4	−0.380	− 8.89
10........	18.0	3.30	59.4	10......	18.0	−0.540	− 9.72
11........	36.0	5.56	200.2	11......	36.0	−0.665	− 23.94
12........	36.0	7.56	271.8	12......	36.0	−0.702	− 25.27
13........	36.0	9.73	350.3	13......	36.0	−0.741	− 26.68
14........	36.0	7.55	271.8	14......	36.0	−0.741	− 26.68
15........	23.4	4.10	95.9	15......	23.4	−0.712	− 16.66
16........	23.4	2.47	57.8	16......	23.4	−0.668	− 15.63
17........	23.4	1.32	30.9	17......	23.4	−0.580	− 13.57
18........	23.4	0.55	12.9	18......	23.4	−0.500	− 11.70
			1,426.3				−187.37

Point 8: Wheel No. 4 at point 8, train headed toward right.

Wheel	Load	Ord.	*M*	Wheel	Load	Ord.	*H*
1.........	18 0	1.65	29.7	1.......	18.0	−0.741	− 13.34
2.........	36.0	4.40	158.4	2.......	36.0	−0.741	− 26.68
3.........	36.0	6 75	243.0	3.......	36.0	−0.704	− 25.34
4.........	36.0	9.05	325.8	4.......	36.0	−0.666	− 23.98
5.........	36 0	7.00	252.0	5.......	36.0	−0.590	− 21.24
6.........	23.4	4.00	93.6	6.......	23.4	−0.426	− 9.97
7.........	23.4	2.55	59.7	7.......	23.4	−0.315	− 7.37
8.........	23.4	1.30	30.4	8.......	23.4	−0.170	− 3.98
9.........	23.4	0.26	6.1	9.......	23.4	−0.040	− 0.94
			1,198.7				−132.84

Point 7: Wheel No. 3 at point 7, train headed toward right.

Wheel	Load	Ord.	*M*	Wheel	Load	Ord.	*H*
1.........	18.0	1.60	28.8	1.......	18.0	−0.692	−12.46
2	36.0	5.15	185.4	2.......	36.0	−0.593	−21.35
3.	36.0	7.67	276.1	3.......	36.0	−0.516	−18.58
4.........	36.0	5.63	202.7	4.......	36.0	−0.408	−14.69
5	36.0	3.50	126.0	5.......	36.0	−0.304	−10.94
6.........	23.4	0.85	19.9	6.......	23.4	−0.075	− 1.76
			838.9				−79.78

Point 6: Wheel No. 3 at point 6, train headed toward right.

Wheel	Load	Ord.	*M*	Wheel	Load	Ord.	*H*
1.........	18.0	−0.85	−15.3	1.......	18.0	−0.565	−10.17
2.........	36.0	2.75	99.0	2.......	36.0	−0.412	−14.83
3.........	36.0	5.21	187.6	3.......	36.0	−0.304	−10.94
4.........	36.0	3.10	111.6	4.......	36.0	−0.175	− 6.30
5.........	36.0	0.80	28.8	5.......	36.0	−0.050	− 1.80
			411.7				−44.04

Point 8: Wheel No. 3 at point 7*R*, train headed toward left.

Wheel	Load	Ord.	*M*	Wheel	Load	Ord.	*H*
1.........	18.0	−0.25	− 4.5	1.......	18.0	−0.691	−12.44
2.........	36.0	−1.26	− 45.4	2.......	36.0	−0.590	−21.24
3.........	36.0	−1.65	− 59.4	3.......	36.0	−0.516	−18.58
4.........	36.0	−1.46	− 52.6	4.......	36.0	−0.411	−14.80
5.........	36.0	−1.26	− 45.4	5.......	36.0	−0.304	−10.94
6.........	23.4	−0.27	− 6.3	6.......	23.4	−0.072	− 1.68
			−213.6				−79.68

Point 7: Wheel No. 4 at point $8R$, train headed toward left.

Wheel	Load	Ord.	M	Wheel	Load	Ord.	H
1.........	18.0	−1.10	− 19.8	1.......	18.0	−0.741	− 13.34
2.........	36.0	−2.96	−106.6	2.......	36.0	−0 741	− 26.68
3.........	36.0	−3.50	−126.0	3.......	36.0	−0.705	− 25.38
4.........	36.0	−4.07	−146.5	4..... .	36.0	−0.666	− 23.98
5.........	36.0	−3.90	−140.4	5..... ..	36.0	−0.588	− 21.17
6.........	23.4	−3.20	− 74.9	6.. ...	23.4	−0.427	− 9.99
7.........	23.4	−2.52	− 59 0	7	23.4	−0.316	− 7.39
8.........	23.4	−1.27	− 29.7	8	23.4	−0.170	− 3.98
9.........	23.4	−0.25	− 5 9	9.......	23.4	−0.034	− 0.80
			−708 8				−132.71

Point 6: Wheel No. 4 at point $9R$, train headed toward left.

Wheel	Load	Ord.	M	Wheel	Load	Ord.	H
1........	18.0	−3.80	− 68.4	1......	18.0	−0.676	− 12.17
2........	36.0	−5.80	− 208.8	2......	36 0	−0.741	− 26.68
3........	36.0	−6.40	− 230.4	3......	36.0	−0.741	− 26.68
4........	36.0	−7.08	− 254.9	4......	36.0	−0.741	− 26.68
5........	36.0	−7.08	− 254.9	5......	36.0	−0.705	− 25.38
6........	23.4	−6.50	− 152.1	6......	23.4	−0.600	− 14.04
7........	23.4	−5.90	− 138.1	7......	23.4	−0.525	− 12.28
8........	23.4	−4.55	− 106.5	8......	23.4	−0.400	− 9.36
9........	23.4	−3.35	− 78.4	9......	23.4	−0.292	− 6.83
10........	18.0	−0.95	− 17.1	10......	18.0	−0.087	− 1.57
			−1,509.6				−161.67

Point 5: Wheel No. 6 at point $9R$, train headed toward right.

Wheel	Load	Ord.	M	Wheel	Load	Ord.	H
1........	18.0	− 3.60	− 64.8	1......	18.0	−0.240	− 4.33
2........	36.0	− 6.36	− 229.0	2.	36.0	−0.420	− 15.13
3........	36.0	− 7.90	− 284.4	3......	36.0	−0.523	− 18.83
4........	36.0	− 9.00	− 324.0	4......	36.0	−0.600	− 21.60
5........	36.0	−10.08	− 362.9	5......	36.0	−0.670	− 24.12
6........	23.4	−11.00	− 257.4	6......	23.4	−0.741	− 17.34
7........	23.4	−10.85	− 253.9	7......	23.4	−0.741	− 17.34
8........	23.4	−10.60	− 248.0	8......	23.4	−0.730	− 17.08
9........	23.4	− 9.85	− 230.5	9......	23.4	−0.694	− 16.24
10........	18.0	− 8.16	− 146.9	10......	18.0	−0.597	− 10.75
11........	36.0	− 5.75	− 207.0	11......	36.0	−0.458	− 16.49
12........	36.0	− 3.90	− 140.4	12......	36.0	−0.344	− 12.38
13........	36.0	− 1.85	− 66.6	13......	36.0	−0.228	− 8.21
14........	36.0	+ 0.24	+ 8.6	14......	36.0	−0.100	− 3.60
			−2,807.2				−203.44

Points 1 to 4: Wheel No. 2 at point 5, train headed toward left.

Wheel	Load	Ord.	V	Wheel	Load	Ord.	H
2........	36.0	0.979	35.24	2......	36.0	−0.051	− 1.84
3........	36.0	0 925	33.30	3......	36.0	−0 175	− 6.30
4........	36.0	0.868	31.25	4......	36.0	−0 304	− 10.94
5.	36.0	0.815	29.34	5......	36.0	−0.410	− 14.76
6........	23.4	0.717	16.78	6......	23.4	−0.579	− 13.55
7........	23.4	0.662	15 49	7......	23.4	−0.655	− 15.33
8........	23.4	0.597	13.97	8	23.4	−0.706	− 16.52
9........	23.4	0.543	12.71	9......	23.4	−0.741	− 17.34
10........	18.0	0.457	8.23	10......	18.0	−0.741	− 13.34
11........	36.0	0.371	13.36	11.....	36 0	−0.688	− 24.77
12........	36.0	0.317	11.41	12......	36.0	−0.636	− 22.90
13........	36.0	0.262	9.43	13.. .	36.0	−0.555	− 19.98
14........	36.0	0.206	7.42	14......	36.0	−0.465	− 16.74
15	23.4	0.106	2.48	15......	23.4	−0.265	− 6.20
16........	23.4	0.050	1.17	16......	23.4	−0.135	− 3.16
			241.58				−203.67

LIVE LOAD MOMENTS IN VERTICAL LEG

Point	y	Hy	x	Vx	M
1...........	1.96	− 399	−0.36	− 87	− 486
2...........	5.89	−1,200	−1.09	−263	−1,463
3...........	9.82	−2,000	−1.82	−440	−2,440
4...........	13.75	−2,800	−2.54	−614	−3,414
Cor.........	15.72	−3,202	−2.90	−701	−3,903

11–12. Dead-Load Bending Moments. The weight of the girder may be estimated from the cross-sectional area at each point by multiplying the area by 3.4 (the weight of 1 sq in. of steel 1 ft long). From Art. 11–7, the dead load coming to the girder from the floorbeams may be estimated to be 1,340 lb per ft. The weight of the concrete and the floor plate carried directly by the girder must be added to the 1,340 lb. This added weight is estimated to be 213 lb per ft. An estimate of the total weight per foot at each of the points 5 to 9 follows:

Point 5 (3.4 × 147.5 + 1,553 = 2,055
Point 6 (3.4 × 112 + 1,553 = 1,934
Point 7 (3.4 × 106.5 + 1,553 = 1,915
Point 8 (3.4 × 116.5 + 1,553 = 1,949
Point 9 (3.4 × 129 + 1,553 = 1,992
Average = 1,970 lb per ft of girder

Reference is made to the influence line ordinates calculated in Table 11–4. A summation of the influence ordinates for loads at points 5 to 9 is shown for each point in the frame. The bending moment at point 5 would then be the amount of the dead load placed at each point, multiplied by the summation of the ordinates for point 5 and point $5R$. This summation will be $-28.93 - 34.22 = -63.15$ for point 5. The influence ordinates for other points are found in a similar manner.

The amount of dead load assumed for each point on the horizontal span is then $1,970 \times 9.78 = 19,270$ lb. The dead load on the vertical leg is not considered to produce an appreciable bending moment. The calculation of the dead-load moments caused by loads of the horizontal span follows:

Point	Coefficient	Section Load (Kips)	Dead-Load Moment (Ft-Kips)
1	−10.71	19.27	− 206
2	−32.27	19.27	− 622
3	−53.86	19.27	−1,038
4	−75.36	19.27	−1,452
5	−63.15	19.27	−1,217
6	−26.84	19.27	− 517
7	0.43	19.27	8
8	18.62	19.27	359
9	27.70	19.27	534

Thrust in the horizontal span is caused by the force H produced by vertical loads on the span. The total dead-load horizontal reaction is found by the use of the summation of the ordinates for H at points 5 to 9 on both halves of the frame, or $2 \times -2.278 = -4.556$. The dead-load horizontal reaction is then $-4.556 \times 19.27 = 87.8$ kips.

Thrust in the vertical member is produced by the vertical loads and is different for each point (from 1 to 4). It is necessary to estimate the dead load coming to each of these points, which includes the weight of the frame and the concrete encasement. The local load at each of the points is estimated as follows: point 1—5,000 lb; point 2—6,500 lb; point 3—8,000 lb; point 4—9,000 lb. The dead-load thrusts at points 1 to 4 are then

Point 4	$(48.9 \times 1,970) + 9,000 = 105,330$ lb
Point 3	$105,330 + 8,000 = 113,330$
Point 2	$113,330 + 6,500 = 119,830$
Point 1	$119,830 + 5,000 = 124,830$

11-13. Moments Due to Temperature Change. The AREA specification requires that the frame be calculated for 40° rise and 60° drop in temperature. The horizontal reaction due to temperature rise is that force which will prevent a change in span length. The equation can be written as

$$Lt^\circ c + \sum \frac{My}{EI} ds = 0,$$

in which L is the span length, t° is the rise in temperature, c is the coefficient of expansion, and M is Hy.

Then $$(46 \times 40 \times 0.0000065) + \frac{4{,}711.6H}{144 \times 30{,}000{,}000} = 0,$$

and $$H = -10{,}966 \text{ lb.}$$

The bending moments at the several points in the frame will be equal to Hy, the thrust in the horizontal span will be equal to H, and the thrust in the vertical leg will be zero. The effect of the 60° drop in temperature is equal to $-60/40$ of the effect of the 40° rise in temperature. The bending moments due to temperature change are as follows:

Point	y	40° Rise $-10.97y$	60° Drop $16.45y$
1.	1.96	− 21.5	32.2
2.	5.89	− 64.6	96.9
3.	9.82	−107.7	161.5
4.	13.75	−150.8	226.2
5.	16.04	−176.0	263.9
6.	16.66	−182.8	274.1
7.	17.11	−187.7	281.5
8.	17.41	−191.0	286.4
9.	17.56	−192.6	288.9

11–14. Moments Caused by Earth Loads. Earth against the vertical legs of the rigid frame produces forces which cause bending moments in the frame. The AREA specification indicates that the Rankine formula

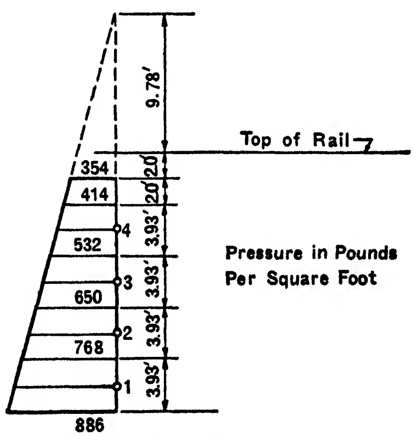


Fig. 11–14. Earth pressures.

for earth thrust is to be used in the computation of these forces. For a granular backfill, the value of $C = 0.286$, and the pressure at any depth below the surface is CwH, in which w is the weight per cu ft of the fill (averaging 105 lb), and H is the distance below the surface. The surcharge may be expressed in terms of equivalent weight of earth. An axle load is distributed over 14 ft of width when its effect is transmitted through a fill. The surcharge is then 72,000/(5 × 14) = 1,028 lb per sq ft of horizontal surface. This surcharge is equivalent to 9.78 ft of fill above the top of the rail. Fig. 11–14 shows the earth pressure diagram. The horizontal pressure acts on the frame from a point 2 ft below the rail to a point 19.72 ft below the top of rail. As shown in Fig. 11–15, the horizontal pressure is assumed to be divided into forces applied at the center of each section of the vertical leg. The pressure is 0.286 × 105 = 30 lb times the depth below the top of the surcharge, and the force is the average pressure times the area on which it is acting. Each frame must resist earth pressure for a width of 6 ft 6 in., since there are two frames for each track and the tracks are spaced at 13-ft centers. The two frames of each unit of bridge are joined by a concrete slab, as shown in section *A–A* of Fig. 11–18. This slab transmits the horizontal earth pressure to the frame.

In solving for bending moments caused by the earth pressure, it is necessary to find a value for the unknown *H*-force which prevents change in span length. As with other forces, $\Sigma(My/EI)ds = 0$, in which M is equal to the simple moment plus the Hy moment. The simple moment is the moment that would exist when $H = 0$. These simple moments are found in the following calculations:

Simple Moments, Earth Pressure

−21,100 ×	3.93 =	− 82,923 ft-lb at point 2
−18,100		
−39,200 ×	3.93 =	−154,056
−15,100		
		−236,979 at point 3
−54,300 ×	3.93 =	−213,399
−12,100		
		−450,378 at point 4
−66,400 ×	2.29 =	−152,056
− 5,000		
−71,400 ×	−0.68 =	+ 48,552
		−553,882 at point 5
−71,400 ×	0.62 =	− 44,268
		−598,150 at point 6
−71,400 ×	0.45 =	− 32,130
		−630,280 at point 7
−71,400 ×	0.30 =	− 21,420
		−651,700 at point 8
−71,400 ×	0.15 =	− 10,710
		−662,410 at point 9

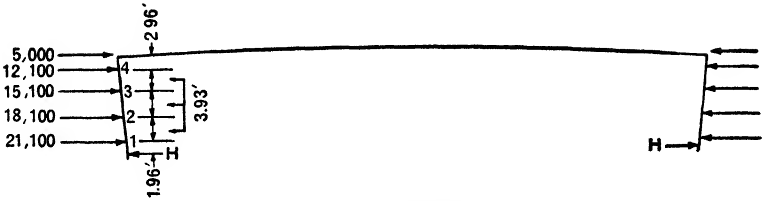


FIG. 11-15. Earth pressure forces.

From Table 11-2,

$$\sum Hy^2 \frac{ds}{I} = 4{,}711.6H.$$

Then

$$4{,}711.6H = +\ 170{,}914{,}900 \text{ (See table below.)}$$

$$H = 36{,}275 \text{ lb.}$$

This value of H is multiplied by y for each point and the Hy moment is combined with the simple moment to give the final value of bending moment.

BENDING MOMENTS, EARTH PRESSURE (kip units)

Point	Simple Moment	$\frac{yds}{I}$	$\frac{M'yds}{I}$	y	Hy (36.28y)	M
1.	. . . .	10.41	. . . .	1.96	71.1	+ 71.1
2.	− 82.9	9.60	− 759.8	5.89	213.7	+130.8
3.	−237.0	7.07	− 1,675.6	9.82	356.3	+119.3
4.	−450.4	5.22	− 2,351.1	13.75	498.9	+ 48.5
5.	−553.9	17.64	− 9,770.8	16.04	581.9	+ 28.0
6.	−598.2	39.32	− 23,521.2	16.66	604.4	+ 6.2
7.	−630.3	60.91	− 38,391.6	17.11	620.8	− 9.5
8.	−651.7	71.21	− 46,407.6	17.41	631.6	− 20.1
9.	−662.4	72.52	− 48,037.2	17.56	637.1	− 25.3
			−170,914.9			

11-15. Combined Moments. In Table 11-5 the bending moments previously found have been combined to obtain the maximum bending moments for all conditions of loading. In making the combinations, it should be noted that dead load is always present, that the temperature effects may be in either direction, and that earth loads are to be considered effective only when they increase the maximum moment. Thrusts are combined in a manner that will give simultaneous values of moment and thrust.

A tractive force equal to 25 per cent of the driver weights and applied

TABLE 11-5

SUMMARY OF BENDING MOMENTS AND DIRECT LOADS

Point	Live Load		Impact		Dead Load		Earth Load		Temperature		Maximum	
	M	*T*	*M*	*T*	*M*	*T*	*M*	*T*	*M*	*T*	*M*	*T**
1.	− 486	−242	− 261	−130	− 206	−125	+ 71	. . . .	− 21 + 32	. . . .	− 974	−497
2.	−1,463	−242	− 786	−130	− 622	−120	+131	. . . .	− 65 + 97	. . . .	−2,936	−492
3.	−2,440	−242	−1,310	−130	−1,038	−113	+119	. . . .	−108 +161	. . . .	−4,896	−485
4.	−3,414	−242	−1,833	−130	−1,452	−105	+ 49	. . . .	−151 +226	. . . .	−6,850	−477
5.	−2,807	−203	−1,507	−109	−1,217	− 88	+ 28	−36	−176 +264	−11 +16	−5,707	−406
6.	−1,510 + 412	−162 − 44	− 811 + 221	− 87 − 24	− 517	− 88	+ 6	−36	−183 +274	−11 +16	−3,021 + 396	−348 −176
7.	− 709 + 839	−133 − 80	− 381 + 450	− 71 − 43	+ 8	− 88	− 10	−36	−188 +282	−11 +16	−1,280 +1,579	−339 −195
8.	− 214 +1,198	− 79 −133	− 115 + 643	− 42 − 71	+ 359	− 88	− 20	−36	−191 +286	−11 +16	− 181 +2,486	−256 −276
9.	+1,426	−187	+ 766	−100	+ 534	− 88	− 25	−36	−193 +289		+3,015	−375

* *Simultaneous with maximum moment.*

at 6 ft above the top of the rails is required to be considered. With a maximum of 8 drivers of 36,000 lb each, a horizontal force of 36,000 lb will be taken to each footing. The vertical reaction due to traction will be

$$\frac{72{,}000 \times 25.71}{92} = 2{,}012 \text{ lb.}$$

The bending moment at point 4 is then

$$(2{,}012 \times -2.54) + (-36{,}000 \times 13.75) = -500{,}100 \text{ ft-lb.}$$

An increase of 25 per cent in unit stress is allowed when longitudinal forces are included. It is apparent that the moment due to traction is less than 25 per cent of the combined moment due to other forces and will not increase the design requirements.

11–16. Adjustment of Trial Section. In the calculations that follow, stresses are computed for the sections of the trial frame, and then adjustments are made to provide stress values within the specification limitations. Rather than compute stresses for each working point in the frame, certain controlling sections are selected. The results of these calculations will determine the necessity for any adjustment, after which each point may be checked.

The first critical point considered is at the horizontal line at the top of the vertical leg. The bending moment at this point may be taken as 12.67/13.75 times the moment at point 4, or $-6{,}850 \times (12.67/13.75) = -6{,}312$ ft-kips. The thrust at this point is about the same as for point 4, or 477 kips. The trial section shows this point to have 14 × 2 cover plates in each flange and a depth of 80 in. back-to-back of flange angles. The method of calculation of the section properties in this case is not exactly as specified by the AREA specification. Holes are deducted from the tension side only, and the neutral axis is considered to be shifted to the resulting gravity axis. This method of calculation permits easier adjustment of the section when material is added to one flange only, or when more material is added to one of the flanges, and was shown in Chap. 6 to give equivalent stress values.

Section at Top of Vertical Leg:

	A	d	Ad	Ad^2	I_0
14 × 2	28.00	83.00	2,324.0	192,892	......
2 ∟s 8 × 6 × 1	26.00	79.35	2,063.1	163,707	161.6
79.5 × 0.5 web	39.75	42.00	1,669.5	70,119	20,936.9
2 ∟s 8 × 6 × 1	26.00	4.65	120.9	562	161.6
14 × 2	28.00	1.00	28.0	28	......
Gross section	147.75		6,205.5	427,308	21,260.1

Holes out:				
2—3 × 1	−6.00	1.5	−9.0	−14
2.5 × 1	−2.50	5.0	−12.5	−62
2.5 × 1	−2.50	8.0	−20.0	−160
0.5 × 1	−0.50	12.0	−6.0	−72
0.5 × 1	−0.50	16.0	−8.0	−128
0.5 × 1	−0.50	20.0	−10.0	−200
0.5 × 1	−0.50	24.0	−12.0	−288
0.5 × 1	−0.50	28.0	−14.0	−392
0.5 × 1	−0.50	32.0	−16.0	−512
0.5 × 1	−0.50	36.0	−18.0	−648
0.5 × 1	−0.50	40.0	−20.0	−800
Net section	132.75		6,060.0	424,032
				21,260
				445,292

$$\bar{y} = \frac{6{,}060}{132.75} = 45.65 \qquad 45.65 \times 6{,}060 = -276{,}639$$

$$I = 168{,}653 \text{ in.}^4$$

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-477{,}000}{132.75} - \frac{6{,}312{,}000 \times 12 \times 38.35}{168{,}653} = -20{,}816 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-477{,}000}{132.75} + \frac{6{,}312{,}000 \times 12 \times 45.65}{168{,}653} = 16{,}909 \text{ psi, vertical component of tensile stress}$$

$$\frac{16{,}909}{0.938} = 18{,}029 \text{ psi tension}$$

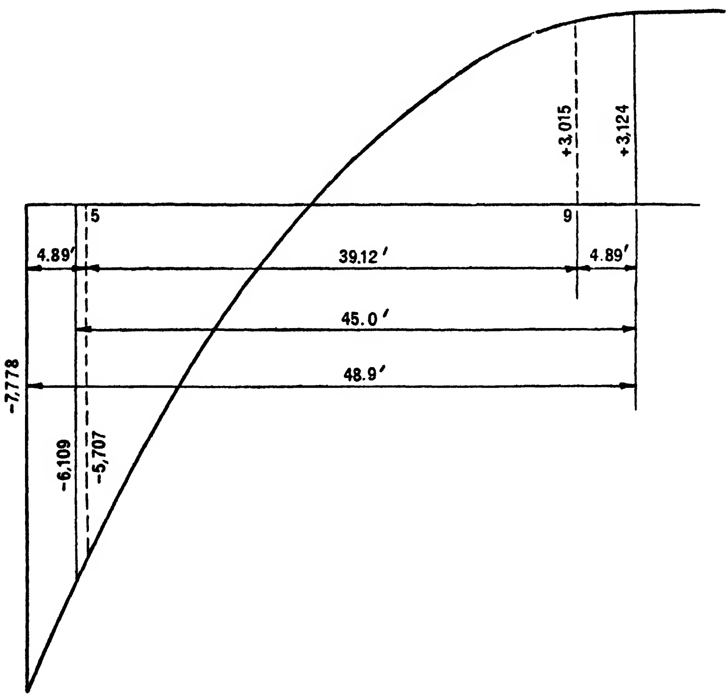


FIG. 11-16.

Critical sections also occur at the face of the vertical leg and at the center of the span. Bending moments were not calculated at these points, but approximate values may be found by extending the bending moment curve, considered as a parabolic curve.

Referring to Fig. 11–16, known values for maximum bending moment have been obtained for points 5 and 9, which are 39.12 ft apart. A value for the probable maximum moment at the center may be found as approximately equal to the ordinate to a parabola passing through the bending moment values found at points 5 and 9, as follows:

$$y = \left(\frac{4.89}{44.01}\right)^2 (8{,}722 + y) = 109;$$

$$\text{Center moment} = +\ 3{,}015 + 109 = +3{,}124 \text{ ft-kips.}$$

The bending moment at the intersection of the working lines for the vertical leg and the horizontal span may be found by extending the parabola:

$$\begin{aligned} \left(\frac{48.9}{44.01}\right)^2 \times -8{,}831 &= -10{,}902 \\ &\quad + 3{,}124 \\ \text{End moment} &= -\ 7{,}778 \text{ ft-kips} \end{aligned}$$

at the face of the support

$$\begin{aligned} \left(\frac{45.0}{44.01}\right)^2 \times -8{,}831 &= -9{,}233 \\ &\quad +3{,}124 \\ \text{End moment} &= -6{,}109 \text{ ft-kips} \end{aligned}$$

It will be seen that since the bending moment and direct force at the face of the vertical leg are less than on the horizontal line at the top of the vertical leg, and since the sections are the same, the section through the vertical leg will control.

The section at the center of the span must be checked for a maximum bending moment of +3,124 ft-kips and a thrust equal to that at point 9, or 375 kips.

Section at Center of Span:

	A	d	Ad	Ad^2	I_o
14 × 2	28.00	45.00	1,260.0	56,700	......
2 ∠s 8 × 6 × 1	26.00	41.35	1,075.1	44,455	161.6
42 × 0.5 web	21.00	23.00	483.0	11,109	3,087.0
2 ∠s 8 × 6 × 1	26.00	4.65	120.9	562	161.6
14 × 2	28.00	1.00	28.0	28	......
Gross section	129.00		2,967.0	112,854	3,410.2

Holes out:				
2—3 × 1	−6.0	1.5	−9.0	−14
2.5 × 1	−2.5	5.0	−12.5	−62
2.5 × 1	−2.5	8.0	−20.0	−160
0.5 × 1	−0.5	12.0	−6.0	−72
0.5 × 1	−0.5	16.0	−8.0	−128
0.5 × 1	−0.5	20.0	−10.0	−200
0.5 × 1	−0.5	24.0	−12.0	−288
Net section	116.00		2,889.5	111,930
				3,410
				115,340
$\bar{y} = \dfrac{2{,}889.5}{116} = 24.91$ in.			$24.91 \times 2{,}889.5 =$	−71,977
			$I =$	43,363 in.^{4}

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-375{,}000}{116.00} - \frac{3{,}124{,}000 \times 12 \times 21.09}{43{,}363} = -21{,}466 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-375{,}000}{116.00} + \frac{3{,}124{,}000 \times 12 \times 24.91}{43{,}363} = +18{,}302 \text{ psi tension}$$

A review of the stresses at the critical sections indicates that the frame is overstressed in compression for structural grade steel. There are several ways in which revisions might be accomplished. The compression flange might be increased, the depth of the section might be increased, or silicon structural steel might be used.

To increase the thickness of the compression flange will increase the rivet grip, which is already 3 in. To increase the depth of the section will increase the depth of the bridge and, consequently, the distance from top of rail to roadway.

The AREA specification allows the use of approximately one-third greater unit stresses in silicon structural steel than in common structural grade steel. The cost of the silicon steel is only slightly greater than for structural grade. The use of silicon structural steel will permit some reduction of the tension flange, since the allowable stress is 24,000 psi. Therefore silicon steel will be used.

Adjustment of Section at Top of Vertical Leg (Remove outer cover Pl):

	A	d	Ad	$Ad^2 + I_o$
Full net section.	132.75	. . .	6,060.0	445,292 in.^{4}
−14 × 0.5 cov Pl	−6.00	0.25	−1.5	. . . .
Net section	126.75		6,058.5	445,292 in.^{4}
$\bar{y} = \dfrac{6{,}058.5}{126.75} = 47.8$			$47.8 \times 6{,}058.5 =$	−289,596
			$I =$	155,696 in.^{4}

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-477{,}000}{126.65} - \frac{6{,}312{,}000 \times 12 \times 36.2}{155{,}696} = -21{,}374 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-477{,}000}{126.75} + \frac{6{,}312{,}000 \times 12 \times 47.3}{155{,}696} = +19{,}248 \text{ psi, vertical component of tensile stress}$$

$$\frac{19{,}248}{0.938} = +20{,}520 \text{ psi tension}$$

The nature of the construction is such that the compression flange is supported against lateral buckling by concrete encasement of the vertical leg.

Vertical Section at Face of Vertical Leg:

The cross section at this point is the same as for the top of the vertical leg. The unit stresses are as follows:

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-406{,}000}{126.75} - \frac{6{,}109{,}000 \times 12 \times 36.2}{155{,}696} = -20{,}247 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-406{,}000}{126.75} + \frac{6{,}109{,}000 \times 12 \times 47.3}{155{,}696} = +19{,}068 \text{ psi tension}$$

The compression flange is supported laterally at 4-ft intervals by web stiffeners which are riveted to the floorbeams, as shown in Fig. 11-17. The

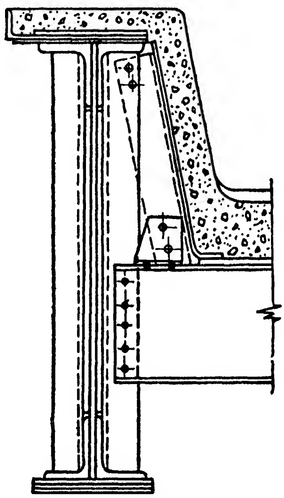


FIG. 11-17. Connection of floorbeams at stiffeners.

AREA *Specifications for the Design of Rigid Frame Steel Bridges* states that, when axial and bending stresses are combined, the value $f_a/F_a + f_b/F_b$ shall not exceed unity, when f_a is the computed axial stress, f_b is the computed bending stress, F_a is the allowable axial stress when axial stress only exists, and F_b is the allowable bending stress when bending stress only exists.

The allowable compressive bending stress for silicon steel girders is $24{,}000 - 6.67(L/b)^2$, or $F_b = 24{,}000 - 6.67(48/14)^2 = 23{,}922$ psi. The allowable axial stress is $F_a = 20{,}000 - 0.46(L/r)^2 = 20{,}000 - 0.46(48/3.6)^2 = 19{,}222$ psi. Then, for this point, $f_a/F_a + f_b/F_b = 3{,}203/19{,}922 + 17{,}044/23{,}922 = 0.87$.

It may be seen that the maximum allowable compressive stress would be 20,247/0.87 = 23,270 psi, when the bending and axial stresses are in the ratio existing at this section in the frame.

Adjustment of Section at Center of Span (Remove bottom Pl):

	A	d	Ad	$Ad^2 + I_0$
Full net section	116.00	..	2,889.5	115,340 in.[4]
-14×0.5 cov Pl	-6.00	0.25	-1.5	....
Net section	110.00		2,888.0	115,340
			$26.25 \times 2,888.0 =$	$-75,810$
			$I =$	39,530 in.[4]

$$\bar{y} = \frac{2,888.0}{110.00} = 26.25$$

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-375,000}{110.00} - \frac{3,124,000 \times 12 \times 19.75}{39,530} = -22,139 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-375,000}{110.00} + \frac{3,124,000 \times 12 \times 25.75}{39,530} = +21,011 \text{ psi tension}$$

Section at Point 6:

	A	d	Ad	Ad^2	I_0
14×1	14.00	65.58	918.1	60,210	....
2 ∟s $8 \times 6 \times 1$	26.00	62.43	1,623.2	101,335	161.6
64×0.5 web Pl	32.00	33.04	1,057.2	34,933	10,923.0
2 ∟s $8 \times 6 \times 1$	26.00	3.65	94.9	346	161.6
14×1	14.00	0.50	7.0	4	....
Gross Section	112.00		3,700.4	196,828	11,246.2
Holes out:					
2—2×1	-4.00	1.00	-4.0	-4	
2.5×1	-2.50	4.00	-10.0	-40	
2.5×1	-2.50	7.00	-17.5	-122	
0.5×1	-0.50	11.00	-5.5	-60	
0.5×1	-0.50	15.00	-7.5	-112	
0.5×1	-0.50	19.00	-9.5	-181	
0.5×1	-0.50	23.00	-11.5	-264	
0.5×1	-0.50	27.00	-13.5	-365	
0.5×1	-0.50	31.00	-15.5	-480	
Net Section	100.00		3,605.9	195,200	
				11,246	
				206,446	
			$36.06 \times 3,605.9 =$	$-130,029$	
			$I =$	76,417 in.[4]	

$$\bar{y} = \frac{3,605.9}{100.0} = 36.06$$

Maximum bending moment at point 6	=	$-3,021,000$ ft-lb
50% of minimum bending moment	=	198,000
Design bending moment	=	$-3,219,000$ ft-lb

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-348,000}{100.0} - \frac{3,219,000 \times 12 \times 30.02}{76,417} = -18,655 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-348,000}{100.0} + \frac{3,219,000 \times 12 \times 36.06}{76,417} = 14,748 \text{ psi tension}$$

Adjustment of Section at Point 6 (Remove top Pl and bottom Pl):

	A	d	Ad	$Ad^2 + I_0$
Net full section	100.00	....	3,605.9	206,446
-14×0.5	-7.00	65.83	-460.8	$-30,334$
-14×0.5 (net)	-6.00	0.25	-1.5	......
Net section	87.00		3,143.6	176,112
			$36.13 \times 3{,}143.6 =$	$-113,578$
			$I =$	62,534 in.^{4}

$$\bar{y} = \frac{3{,}143.6}{87.0} = 36.13$$

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-348{,}000}{87.0} - \frac{3{,}219{,}000 \times 12 \times 29.45}{62{,}534} = -22{,}192 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-348{,}000}{87.0} + \frac{3{,}219{,}000 \times 12 \times 35.63}{62{,}534} = 18{,}009 \text{ psi tension}$$

Section at Point 7:

It appears obvious that the trial section provided for point 7 is greater than required. Consequently, a reduced section is computed with only one 14×0.5 cover plate top and bottom.

	A	d	Ad	Ad^2	I_0
14×0.5	7.00	54.03	378.2	20,435	......
2 Ls $8 \times 6 \times 1$	26.00	51.13	1,329.4	69,971	161.6
53×0.5 web Pl	26.50	27.14	719.2	19,521	6,203.2
2 Ls $8 \times 6 \times 1$	26.00	3.15	81.9	258	161.6
14×0.5	7.00	0.25	1.7	......	......
Gross section	92.50		2,510.4	108,183	6,526.4
Holes out:					
2—1.5×1	-3.00	0.75	-2.2	-2	
2.5×1	-2.50	3.50	-8.8	-31	
2.5×1	-2.50	6.50	-16.2	-106	
0.5×1	-0.50	10.50	-5.2	-55	
0.5×1	-0.50	14.50	-7.2	-105	
0.5×1	-0.50	18.50	-9.2	-171	
0.5×1	-0.50	22.50	-11.2	-253	
0.5×1	-0.50	26.50	-13.2	-351	
Net section	82.00		2,437.2	107,109	
				6,526	
				113,635	
			$29.72 \times 2{,}437.2 =$	$-72,434$	
			$I =$	41,201 in.^{4}	

$$\bar{y} = \frac{2{,}437.2}{82.0} = 29.72$$

Maximum bending moment at point 7	=	+1,579,000 ft-lb
50% of minimum bending moment	=	640,000
Design bending moment	=	+2,219,000 ft-lb

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-195{,}000}{82.00} - \frac{2{,}219{,}000 \times 12 \times 24.56}{41{,}201} = -18{,}251 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-195{,}000}{82.00} + \frac{2{,}219{,}000 \times 12 \times 29.72}{41{,}201} = 16{,}830 \text{ psi tension}$$

Section at Point 8:

	A	d	Ad	Ad^2	I_o
14 × 1.5	21.00	48.33	1,014.9	49,052	......
2 Ls 8 × 6 × 1	26.00	44.93	1,168.2	52,486	161.6
45 × 0.5 web Pl	22.50	24.54	552.1	13,550	3,796.9
2 Ls 8 × 6 × 1	26.00	4.15	107.9	448	161.6
14 × 1.5	21.00	0.75	15.8	12	......
Gross section	116.50		2,858.9	115,548	4,120.1
Holes out:					
2—2.5 × 1	−5.00	1.25	−6.2	−8	
2.5 × 1	−2.50	4.50	−11.2	−51	
2.5 × 1	−2.50	7.50	−18.8	−141	
0.5 × 1	−0.50	11.50	−5.8	−66	
0.5 × 1	−0.50	15.50	−7.7	−120	
0.5 × 1	−0.50	19.50	−9.8	−190	
Net section	105.00		2,799.4	114,972	
				4,120	
				119,092	
			26.66 × 2,799.4 =	− 74,632	
			I =	44,460 in.^{4}	

$$\bar{y} = \frac{2,799.4}{105.00} = 26.66$$

Maximum moment at point 8	=	+2,486,000 ft-lb
50% of minimum moment	=	91,000
Design bending moment	=	+2,577,000 ft-lb

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-276,000}{105.00} - \frac{2,577,000 \times 12 \times 22.42}{44,460} = -18,223 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-276,000}{105.00} + \frac{2,577,000 \times 12 \times 26.66}{44,460} = 15,914 \text{ psi tension}$$

Adjustment of Section at Point 8 (Remove one top Pl and two bottom Pls):

	A	d	Ad	$Ad^2 + I_o$
Net full section	105.00	....	2,799.4	119,092
−14 × 0.5	−7.00	48.83	−341.8	−16,690
−14 × 1 (net)	−12.00	0.50	−6.0	−3
Net section	86.00		2,457.6	102,399
			28.65 × 2,457.6 =	−70,410
			I =	31,989 in.^{4}

$$\bar{y} = \frac{2,457.6}{86.00} = 28.65$$

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-276,000}{86.00} - \frac{2,577,000 \times 12 \times 19.93}{31,989} = -22,475 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-276,000}{86.00} + \frac{2,577,000 \times 12 \times 27.65}{31,989} = 23,520 \text{ psi tension}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{3,210}{19,922} + \frac{19,265}{23,922} = 0.97$$

Section at Point 3:

	A	d	Ad	Ad^2	I_0
14 × 1.5	21.00	69.81	1,466.0	102,342	....
2 ⌊s 8 × 6 × 1	26.00	66.41	1,726.7	114,667	161.6
67.0 × 0.5 web Pl	33.50	35.28	1,181.9	41,967	12,531.8
2 ⌊s 8 × 6 × 1	26.00	4.15	107.9	448	161.6
14 × 1.5	21.00	0.75	15.8	12	....
Gross section	127.50		4,498.3	259,436	12,855.0
Holes out:					
2—2.5 × 1	−5.00	1.25	−6.2	−8	
2.5 × 1	−2.50	4.50	−11.2	−51	
2.5 × 1	−2.50	7.50	−18.7	−141	
0.5 × 1	−0.50	11.50	−5.8	−66	
0.5 × 1	−0.50	15.50	−7.7	−120	
0.5 × 1	−0.50	19.50	−9.8	−190	
0.5 × 1	−0.50	23.50	−11.7	−276	
0.5 × 1	−0.50	27.50	−13.8	−378	
Net section	115.00		4,413.4	258,206	
				12,855	
				271,061	
			$38.38 \times 4{,}413.4 =$	−169,386	
			$I =$	101,675 in.⁴	

$$\bar{y} = \frac{4413.4}{115.00} = 38.38$$

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-485{,}000}{115.00} - \frac{4{,}896{,}000 \times 12 \times 32.18}{101{,}675} = -22{,}812 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-485{,}000}{115.00} + \frac{4{,}896{,}000 \times 12 \times 38.38}{101{,}675} = +17{,}961 \text{ psi, vertical component of tensile stress}$$

$$\frac{17{,}961}{0.938} = +19{,}148 \text{ psi tension}$$

Section at Point 2:

	A	d	Ad	Ad^2	I_0
14 × 1	14.00	51.66	723.2	37,363	......
2 ⌊s 8 × 6 × 1	26.00	48.51	1,261.2	61,184	161.6
50 × 0.5 web Pl	25.00	26.08	652.0	17,004	5,208.3
2 ⌊s 8 × 6 × 1	26.00	3.65	94.9	346	161.6
14 × 1	14.00	0.50	7.0	4	......
Gross section	105.00		2,738.3	115,901	5,531.5
Holes out:					
2—2 × 1	−4.00	1.00	−4.0	−4	
2.5 × 1	−2.50	4.00	−10.0	−40	
2.5 × 1	−2.50	7.00	−17.5	−122	
0.5 × 1	−0.50	11.00	−5.5	−60	
0.5 × 1	−0.50	15.00	−7.5	−113	
0.5 × 1	−0.50	19.00	−9.5	−180	
0.5 × 1	−0.50	23.00	−11.5	−265	
Net section	94.00		2,672.8	115,117	
				5,532	
				120,649	
			$28.43 \times 2{,}672.8 =$	− 75,988	
			$I =$	44,661 in.⁴	

$$\bar{y} = \frac{2{,}672.8}{94.0} = 28.43$$

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-492{,}000}{94.00} - \frac{2{,}936{,}000 \times 12 \times 23.73}{44{,}661} = -23{,}954 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-492{,}000}{94.00} + \frac{2{,}936{,}000 \times 12 \times 28.43}{44{,}661} = +17{,}194 \text{ psi, vertical component of tensile stress}$$

$$\frac{+17{,}194}{0.938} = +18{,}330 \text{ psi tension}$$

Adjustment of Section at Point 2 (Add one inside Pl):

	A	d	Ad	$Ad^2 + I_0$
Full net section	94.00	....	2,672.8	120,649
$+14 \times 0.5$	7.00	52.41	366.9	19,228
Net section	101.00		3,039.7	139,877
			$30.10 \times 3{,}039.7 =$	$-91{,}495$
			$I =$	48,382 in.^{4}

$$\bar{y} = \frac{3{,}039.7}{101.0} = 30.10$$

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-492{,}000}{101.00} - \frac{2{,}936{,}000 \times 12 \times 22.56}{48{,}382} = -21{,}294 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-492{,}000}{101.00} + \frac{2{,}936{,}000 \times 12 \times 30.10}{48{,}382} = +17{,}042 \text{ psi, vertical component of tensile stress}$$

$$\frac{17{,}042}{0.938} = +18{,}168 \text{ psi tension}$$

Section at Point 1:

	A	d	Ad	Ad^2	I_0
14×0.5	7.00	33.39	233.7	7,804	......
2 Ls $8 \times 6 \times 1$	26.00	30.49	792.7	24,176	161.6
32.0×0.5 web Pl	16.00	16.82	269.12	4,627	1,365.3
2 Ls $8 \times 6 \times 1$	26.00	3.15	81.9	258	161.6
14×0.5	7.00	0.25	1.8	.....	......
Gross section	82.25		1,379.2	36,760	1,688.5
Holes out:					
2—1.5×1	−3.00	0.75	−2.2	−2	
2.5×1	−2.50	3.50	−8.8	−31	
2.5×1	−2.50	6.50	−16.3	−106	
0.5×1	−0.50	10.50	−5.2	−55	
0.5×1	−0.50	14.50	−7.2	−105	
Net section	73.25		1,339.5	36,461	
				1,689	
				38,150	
			$18.35 \times 1{,}339.5 =$	−24,580	
			$I =$	13,570 in.^{4}	

$$\bar{y} = \frac{1{,}339.5}{73.00} = 18.35$$

$$\frac{P}{A} - \frac{Mc}{I} = \frac{-497{,}000}{73.00} - \frac{974{,}000 \times 12 \times 15.29}{13{,}570} = -19{,}977 \text{ psi compression}$$

$$\frac{P}{A} + \frac{Mc}{I} = \frac{-497{,}000}{73.00} + \frac{974{,}000 \times 12 \times 18.35}{13{,}570} = +8{,}997 \text{ psi, vertical component of tensile stress}$$

$$\frac{+8{,}997}{0.938} = +9{,}592 \text{ psi tension}$$

11–17. Web Design. In order to check the stresses in the web plate at critical points, it is necessary to compute the shear at these points. For all points in the vertical leg, the maximum shear is equal to the thrust for point 5, or 406,000 lb.

The AREA specification permits 14,000 psi average shear on the gross section of the web for silicon steel. The vertical leg has varying depths along its length. The gross area of the web required to resist the shear is then 406,000/14,000 = 29.0 sq in. The ½-in. web will be adequate where the depth of the section is 58 in. or more, which is at 7 ft 8⅜ in. above the base. Below this point it is necessary to reinforce the web with plates. The minimum thickness of metal is ⅜ in. and it is desirable to keep the section symmetrical. The shear value may be provided by means of a filler between the flanges and a ⅜-in. reinforcing Pl on each side, as shown in Fig. 11–18.

At point 5, the maximum live-load shear can be found to occur with wheel No. 2 at point 5. The vertical shear at this point is then

Live load	=	242,340 lb
Impact, 53.7%	=	130,135
Dead load, 4.5 × 19,270	=	86,715
		459,190 lb

The average shear in the web at point 5 is then

$$\frac{459{,}190}{79 \times 0.5} = 11{,}625 \text{ psi},$$

which is within the 14,000 psi allowable.

At point 6 the maximum live-load shear occurs with wheel No. 2 at point 6 and is equal to 194,000 lb. The shear loads are

Live load	=	194,000 lb
Impact, 53.7%	=	104,180
Dead load, 3.5 × 19,270	=	67,445
		365,625 lb.

The average shear in the web at point 6 is then

$$\frac{365{,}625}{64 \times 0.5} = 11{,}425 \text{ psi}.$$

At point 7, the maximum shear occurs with wheel No. 2 at point 7, and the shear load is

Live load	=	149,900
Impact, 53.7%	=	80,495
Dead load, 2.5 × 19,270	=	48,175
		278,570 lb.

The average shear in the web at point 7 is

$$\frac{278{,}570}{53 \times 0.5} = 10{,}510 \text{ psi.}$$

From point 7 toward the center of the span, the shear load decreases very rapidly, and the web will not need to be investigated for unit stress. For silicon steel webs, the AREA specification requires a minimum thickness of 1/145 of the clear distance between flanges, or $64/145 = 0.44$ in. No special provision is made for stiffener spacing in silicon steel webs, and it is presumed that the specification for structural steel applies, or

$$d = \frac{10{,}500t}{\sqrt{S}},$$

in which t is the thickness and S is the average shear on the gross section of the web. The maximum shear on the horizontal span is at point 6, where $S = 11{,}625$ psi. The maximum spacing of stiffeners at this point is then

$$d = \frac{10{,}500 \times 0.5}{\sqrt{11{,}625}} = 48.7 \text{ in.}$$

Stiffeners may be placed at alternate floorbeam connections, which will give a spacing of 48 in.

Along the vertical leg, the maximum shear is 14,000 psi. Where the web is ½ in. thick, a stiffener spacing will be required equal to

$$d = \frac{10{,}500 \times 0.5}{\sqrt{14{,}000}} = 44.3 \text{ in.}$$

At the bottom of the vertical leg, where the web is reinforced, the ratio of clear depth between flanges to thickness of web is less than 60, and stiffeners are not required.

The design of web stiffeners is controlled by the requirement that the outstanding leg should be 1/30 of the depth of the girder plus 2 in. This requires that a leg of 5 in. be used. The thickness of the outstanding leg should be at least 1⁄16 of the length of the leg, but the minimum thickness of metal is ⅜ in. The stiffeners must be in pairs and will be 5 x 3½ x ⅜ angles.

11–18. Splices. The frame obviously cannot be constructed without splices in the web and flange angles. Consideration should be given to the largest size that may be shipped and to maximum widths and lengths of available plates. Since the web and flange angles must be spliced at approximately the same point in order to provide for field splices, such points should be selected where there is excess section. As shown in Fig. 11–18, a point 10 ft ¾ in. from the face of the vertical leg is selected.

The web of the vertical leg and the horizontal span up to the splice may be sketched from a 180-in. width plate. According to the AISC handbook, this plate is available in lengths up to 27 ft. The parts of this plate that are cut out are not necessarily waste, since many of the details can be made from these cutouts. The web of the main span can be made from a 65 x ½ Pl. This plate is available up to 70 ft in length; hence, it will not be necessary to provide a shop splice at another point in the frame.

Rather than splice the flange angles all at one point, the near-side angles are spliced at one section and the far-side angles at another section. This keeps the maximum rivet grip shorter and provides for more effective splicing. The cover plate is spliced at the same point as the web.

The erection procedure will be to stand the vertical legs upright and then swing the horizontal span into place from the side. The vertical leg must, of course, be temporarily supported until the splices are completed.

As illustrated in Chap. 6, the web splice is accomplished by developing the tensile value of the web with splice material and rivets. The web splice between the flanges should develop a 49×0.5 silicon steel web plate, less the rivet holes taken out, or

$$\frac{0.5(49 - 12) \times 24{,}000}{11{,}800} = 38 \text{ rivets.}$$

In the detail shown, 36 rivets are provided in three rows, which should be satisfactory.

For that part of the web between the flange angles, the number of rivets would be

$$\frac{0.5(8 - 2) \times 24{,}000}{11{,}800} = 6.1 \text{ rivets.}$$

Since the splice of this part of the web is indirect, two additional rows are required, and 10 rivets are provided on each side of the splice. This splice plate should lap over and be attached to the splice between the flange angles to provide for transfer of horizontal shear between the elements of the web splice.

A flange angle is spliced by the addition of an 8 x 6 x 1 angle with sufficient rivets to develop the net section in tension. The number of rivets required is

$$\frac{24{,}000 \times 10.0}{8{,}120} = 30 \text{ rivets.}$$

Of these 30 rivets, 8/14 or 18 rivets should be in the vertical leg of the angle. The remaining 12 rivets would be in the horizontal leg.

11–19. Flange Rivet Spacing. The attachment of flanges to the web may be calculated in the same manner as for plate girders. The calcula-

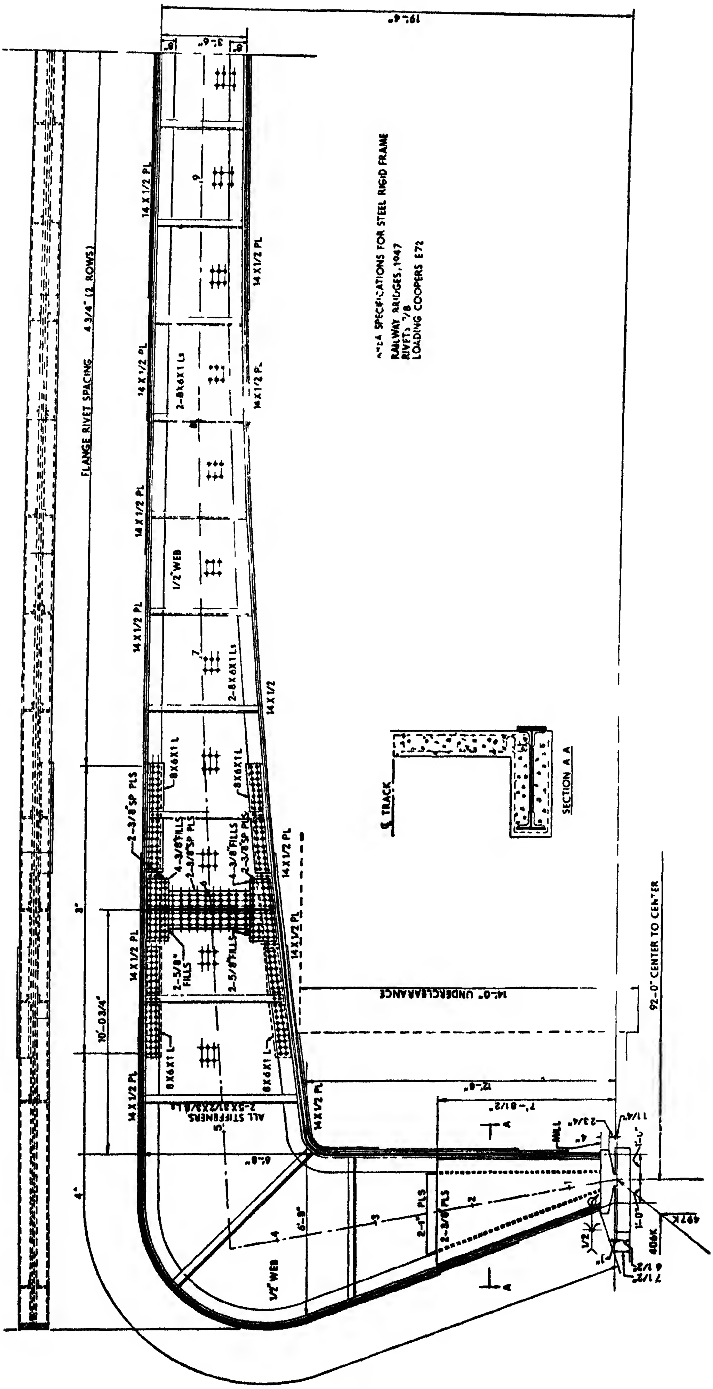

RAILWAY BRIDGES, 1947
LOADING COOPERS E72
FLANGE RIVET SPACING 4 3/4" (2 ROWS)
14'-0" UNDERCLEARANCE
92'-0" CENTER TO CENTER
SECTION A A
TRACK
1/2" WEB
14 X 1/2 PL
8X6X1 L
2-8X6X1 Ls
19'-6"
12'-6"
10'-0 3/4"
406K
497K

FIG. 11-18.

tions at a few critical points will give the required information for spacing the flange rivets.

Vertical Section at Face of Vertical Leg:

Statical moment of the flange angles and cover plates for the compression side is 1,580 in.^{3}

Shear load per linear inch is $\dfrac{459{,}190 \times 1{,}580}{155{,}696} = 4{,}660$ lb. (VQ/I)

Rivet spacing is $\dfrac{11{,}800}{4{,}660} = 2.53$ in. on one row, or 5.06 in. on two rows.

Point 6:

Statical moment of flange of the compression side is 888.7 in.^{3}

Shear load per linear inch is $\dfrac{365{,}625 \times 888.7}{62{,}534} = 5{,}196$ lb.

Rivet spacing is $\dfrac{11{,}800}{5{,}196} = 2.27$ in. on one row, or 4.54 in. on two rows.

Point 7:

Statical moment of the compression flange is 727 in.^{3}

Shear load per linear inch is $\dfrac{278{,}570 \times 727}{41{,}201} = 4{,}915$ lb.

Rivet spacing is $\dfrac{11{,}800}{4{,}915} = 2.40$ in. on one row, or 4.80 in. on two rows.

Top of Vertical Leg:

Statical moment of compression flange is 1,580 in.^{3}

Shear load per linear inch is $\dfrac{406{,}000 \times 1{,}580}{155{,}696} = 4{,}120$ lb.

Rivet spacing is $\dfrac{11{,}800}{4{,}120} = 2.86$ in. on one row, or 5.72 in. on two rows.

Point 1:

Statical moment of the compression flange is $(26 \times 12.14) + (14 \times 15.04) = 526$ in.^{3}

The moment of inertia, with fillers and reinforcing plates, is

$$13{,}570 + \left(\frac{1}{12} \times 2 \times 16^3\right) + \left(\frac{1}{12} \times \frac{3}{4} \times 28^3\right) = 15{,}625 \text{ in.}^4$$

Shear load per linear inch is $\dfrac{406{,}000 \times 526}{15{,}625} = 13{,}667$ lb.

The rivets attaching the flange angles to the web and to the web reinforcement are effective on four shear planes. The interior two shear planes are limited by the bearing value on the ½-in. Pl, or 11,800 lb. The outside two shear values are 8,120 lb. each. Then, each rivet has a value of 28,040 lb.

Rivet spacing is $\frac{28,040}{13,667} = 2.05$ in. on one row, or 4.10 in. on two rows.

Point 3:

Statical moment of the compression flange is 1,389 in.³

Shear load per linear inch is $\frac{406,000 \times 1,389}{101,675} = 5,546$ lb.

Rivet spacing is $\frac{11,800}{5,546} = 2.13$ in. on one row, or 4.26 in. on two rows.

As shown on the drawing of Fig. 11–18 the flange rivets may be spaced on two rows at 4¾ in. from the splice to the centerline and 4 in. for the remainder of the flange length.

11–20. End Bearing. Detail of the end bearing can be simplified by welding. As shown in Fig. 11–18, the end of the vertical leg would be milled to secure proper bearing on a planed billet. The end must be attached to the steel slab with enough weld to transfer the maximum horizontal load of 406,000 lb. Thus,

$$\frac{406,000}{4,800} = 85 \text{ lin in. of } \tfrac{1}{2}\text{-in. weld required.}$$

At the top of the bearing plate, the distance around the section is found to be 94 in.

The slab to which the vertical leg is welded must be designed for bending moment. At this section the distance back-to-back of angles is 27 in. The total reaction of 497,000 lb is assumed to bear uniformly over the entire section, and the bending moment will equal the moment of the acting area on one side of the centerline times the pressure per square inch. Calculation of the bending moment and the bearing pressure follows:

	A	*d*	*Ad*
Cov Pl 14 × 0.5	7.00	13.75	96.2
2 ∟s 8 × 6 × 1	26.00	10.85	282.1
Web 13.5 × 0.5	6.75	6.75	45.6
Web 5.5 × 2.0	11.00	2.75	30.2
	50.75		454.1

Then the bearing pressure is

$$\frac{497,000}{101.5} = 4,897 \text{ psi},$$

and the bending moment is

$$4,897 \times 454.1 = 2,223,700 \text{ in.-lb.}$$

The section modulus required will be

$$\frac{2,223,700}{18,000} = 123.5 \text{ in.}^3$$

Assuming that the slab is 15 in. in width, a depth of 7.0 in. will be required. An 8-in. slab, beveled as shown in Fig. 11–18, will provide the necessary section.

The base slab must be designed for bending moment produced by the concentrated load from the frame and a distributed load on the concrete masonry. The size of the base slab, as determined from the allowable masonry bearing pressure of 600 psi, is

$$\frac{497,000}{600} = 828.3 \text{ sq in.}$$

The available width of slab is 20.5 in.; hence, the slab must be 20.5 × 40.5 in. The bending moment is

$$\frac{497,000}{2} \times 10.12 = 2,514,800 \text{ in.-lb.}$$

The section modulus required for the base slab will be

$$\frac{2,514,800}{18,000} = 139.7 \text{ in.}^3$$

and the required thickness of slab is 6.4 in. Allowing 1 in. for the slot, a slab thickness of 7.5 in. will be satisfactory.

The footing must be designed to prevent spread of the bearings under the vertical legs of the frame, in order that the frame may function as designed. This requires that a horizontal bearing value of 406,000 lb be provided for the footing under each rib. Assuming that the structure is constructed on soil capable of resisting 8,000 lb per sq ft horizontal pressure, the area required for the horizontal reaction would be

$$\frac{406,000}{8,000} = 50.8 \text{ sq ft.}$$

A footing that is 4 ft deep and continuous between ribs would provide 4 × 13 = 52 sq ft of vertical face to develop the horizontal reaction. The bearing slab should be embedded to its full depth into the concrete footing.

Stresses in the haunch of the rigid frame may be computed by a curved beam analysis. These stresses may be computed in the manner illustrated in Art. 11–5.

APPENDIX A

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS

STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 1953

Division III, Articles 3.4.2 to 3.4.10, and 3.6.1 to 3.6.106

DIVISION III

DESIGN

Design Analysis

In any case where the specifications provide for an empirical formula as a design convenience, a rational analysis based on a theory accepted by the Bridge Committee of the American Association of State Highway Officials, with stresses in accordance with the specifications, will be considered as compliance with the specifications.

* * * *

Section 4. Steel Structures

Unless otherwise specified or noted on the plans, it shall be assumed that the steel is to be Structural Carbon Steel, ASTM A7-50T. The modulus of elasticity for all grades of steel shall be assumed at 29,000,000 and the coefficient of expansion .0000065 per degree Fahrenheit.

3.4.2. Structural Carbon Steel—ASTM—A 7-50 T

Rivet steel, AASHO—M 97-39 (ASTM—A 141-39).
Pins and rollers, ASTM—A 235-46 (Class C1).
Axial tension, structural steel, net section 18,000
Tension in extreme fibers of rolled shapes, girders, and built sections subject to bending 18,000
Tension in bolts at root of thread 13,500
Axial compression, gross section:
Stiffeners of plate girders 18,000

The permissible unit stress in concentrically loaded columns having values of L/r not greater than 140 may be computed from the following approximate formulas, or from the more exact formulas given in *Appendix B:*

Riveted ends .... $15{,}000 - \frac{1}{4}\frac{L^2}{r^2}$

Pin ends .... $15{,}000 - \frac{1}{3}\frac{L^2}{r^2}$

L = length of member, in inches.
r = least radius of gyration of member, in inches.

For compression members with values of L/r greater than 140, and for compression members of known eccentricity, see *Appendix B*.

Compression in extreme fibers of rolled shapes, girders, and built sections, subject to bending, gross section

(a) When compression flange is supported laterally its full length by embedment in concrete or by other means............. 18,000

(b) * When compression flange is partially supported or is unsupported... $18{,}000 - 5\frac{L^2}{b^2}$

For values of L/b not greater than 30

where L = length, in inches, of unsupported flange between lateral connections, knee braces, or other points of support. For continuous beams and girders, L may be taken as the distance from interior support to point of dead load contraflexure if this distance is less than that designated above.

b = flange width in inches.

Allowable compression in splice material, gross section.................. 18,000
Stress in extreme fiber of pins.. 27,000
Shear in girder webs, gross section... 11,000
Diagonal tension in webs of girders and rolled beams, at sections where maximum shear and bending occur simultaneously................... 18,000
Shear in power-driven rivets and pins.............................. 13,500
Shear in turned bolts and ribbed bolts.............................. 11,000
Bearing on pins not subject to rotation.............................. 24,000
Bearing on power-driven rivets, milled stiffeners, and other steel parts in contact... 27,000
(Rivets driven by pneumatically or electrically operated hammers are considered power driven.)
Bearing on pins subject to rotation (not due to deflection).............. 12,000
Bearing on turned bolts and ribbed bolts............................. 20,000
Bearing on expansion rollers and rockers, pounds per linear inch:

Diameters up to 25 inches................................ $\frac{p - 13{,}000}{20{,}000}\ 600d$

Diameters from 25 to 125 inches.................... $\frac{p - 13{,}000}{20{,}000}\ 3{,}000\sqrt{d}$

d = diameter of roller or rocker, in inches.
p = yield point in tension of steel in the roller or the base, whichever is the lesser.

In proportioning rivets the nominal diameter shall be used.

The effective bearing area of a pin, a bolt, or a rivet shall be its diameter multiplied by the thickness of the metal on which it bears.

In metal less than 3/8 inch thick, countersunk rivets shall not be assumed to carry stress. In metal 3/8 inch thick and over, one-half the depth of countersink shall be omitted in calculating bearing area.

---

* Continuous or cantilever beams or girders may be proportioned for negative moment at interior supports for an allowable unit stress 20 per cent higher than permitted by above formula but in no case exceeding allowable unit stress for compression flange supported its full length. If cover plates are used, the allowable stress at the point of cut-off shall be as determined by the formula.

3.4.3. High Strength Rivets

High Strength Structural Rivet Steel, AASHO—M 98-41 (ASTM—A 195-41).

Shear	20,000
Bearing	40,000

3.4.4. Wrought Iron

Shapes and bars, AASHO—M 100-39 (ASTM—A 207-39).
Plates (ASTM—A 42-47).
Pipe (ASTM—A 72-45).

Tension	14,000
Bending on extreme fiber	14,000

3.4.5. Cast Steel, Malleable Castings, and Cast Iron

For cast steel the allowable unit stresses in compression and bearing shall be the same as those for structural steel. Other allowable unit stresses shall be three-fourths of those for structural steel.

For cast iron of the grades specified in division IV the following allowable unit stresses shall be used:

Bending on extreme fiber	3,000
Shear	3,000
Direct compression (short columns)	12,000

For malleable castings conforming to the requirements Arts. 4.11.1 to 4.11.3, incl.

Tension	18,000
Bending on extreme fiber	18,000
Modulus of elasticity	25,000,000

3.4.6. Bronze or Copper Alloy

Bearing on bronze or copper alloy bearing and expansion plates	2,000

3.4.7. Structural Low-Alloy Steel

This steel is designated: Structural low-alloy steel, ASTM—A 242-49 T.

The foregoing specifications shall apply to low-alloy steel parts of bridges except as provided otherwise below, and the nomenclature below is the same unless otherwise defined.

Thickness	Low-alloy steel ¾ in. and under	Low-alloy steel over ¾ in. to 1½ in. incl.	Low-alloy steel over 1½ in. to 2 in. incl.
Yield point, min.	50,000	45,000	40,000
Axial tension, structural steel, net section	27,000	24,000	22,000
Tension in extreme fibers of rolled shapes, girders, and built sections subject to bending	27,000	24,000	22,000
Axial compression, gross section: Stiffeners of plate girders	27,000	24,000	22,000

The permissible unit stress in concentrically loaded columns having values of L/r not greater than 125 for low-alloy steel may be computed from the following approximate formulas or by the more exact method using the formulas given in Appendix B.

Riveted ends	$22{,}000-0.56\frac{L^2}{r^2}$	$20{,}000-0.46\frac{L^2}{r^2}$	$18{,}000-0.39\frac{L^2}{r^2}$
Pin ends	$22{,}000-0.73\frac{L^2}{r^2}$	$20{,}000-0.61\frac{L^2}{r^2}$	$18{,}000-0.48\frac{L^2}{r^2}$

For compression members with values of L/r greater than 125 for low alloy steel, and for compression members of known eccentricity see *Appendix B.*

Compression in extreme fibers of rolled shapes, girders, and built sections, subject to bending, gross section.

Thickness	Low-alloy steel ¾ in. and under	Low-alloy steel over ¾ in. to 1½ in. incl.	Low-alloy steel over 1½ in. to 2 in. incl.
(a) When compression flange is supported laterally its full length by embedment in concrete or by other means	27,000	24,000	22,000
(b) *When compression flange is partially supported or is unsupported	$27{,}000-7.5\frac{L^2}{b^2}$	$24{,}000-6.67\frac{L^2}{b^2}$	$22{,}000-6.11\frac{L^2}{b^2}$

For values of L/b not to exceed 25

where L = length, in inches, of unsupported flange between lateral connections, knee braces, or other points of support. For continuous beams and girders, L may be taken as the distance from interior support to point of dead load contraflexure if this distance is less than that designated above.

b = flange width in inches.

Compression, splice material gross section	27,000	24,000	22,000
Shear in plate girder webs, gross sections	15,000	14,000	12,000
Diagonal tension in webs of girders and rolled beams at sections where maximum shear and bending occurs simultaneously	27,000	24,000	22,000
Bearing on pins (not subject to rotation)	36,000	32,000	28,000
Bearing on milled stiffeners and other parts in contact	40,000	36,000	32,000

* Note. Continuous or cantilever beams or girders may be proportioned for negative moment at interior supports for an allowable unit stress 20 per cent higher than permitted by above formula but in no case exceeding allowable unit stress for compression flange supported its full length. If cover plates are used, the allowable stress at the point of cut-off shall be as determined by the formula.

Bearing on pins subject to rotation (not due to deflection)....	18,000	16,000	14,000
Bearing on rollers and rockers: Same formulas as in article 3.4.2.			

Note: For determining the permissible stresses for low-alloy steel, the thickness of beveled flanges or plates shall be the average thickness. For built-up members, the plate having the greatest thickness shall govern.

3.4.8. Structural Nickel and Structural Silicon Steels

These steels are designated: structural silicon steel (ASTM—A 94-46) and structural nickel steel (ASTM—A 8-46).

The foregoing specifications shall apply to silicon or nickel steel parts of bridges except as provided otherwise below, and the nomenclature below is the same unless otherwise defined:

	Silicon steel	Nickel steel
Axial tension, structural steel, net section.........	24,000	30,000
Tension in extreme fibers of rolled shapes, girders, and built sections, subject to bending..........	24,000	30,000
Axial compression, gross section: Stiffeners of plate girders..................	24,000	30,000

The permissible unit stress in concentrically loaded columns having values of L/r not greater than 130 for silicon steel or 120 for nickel steel may be computed from the following approximate formulas or by the more exact method using formulas given in *Appendix B*.

	Silicon steel	Nickel steel
Riveted ends................................	$20{,}000 - 0.46\dfrac{L^2}{r^2}$	$24{,}000 - 0.66\dfrac{L^2}{r^2}$
Pin ends....................................	$20{,}000 - 0.61\dfrac{L^2}{r^2}$	$24{,}000 - 0.86\dfrac{L^2}{r^2}$

For compression on members with values of L/r greater than 130 for silicon steel and 120 for nickel steel, and for compression members of known eccentricity, see *Appendix B*.

Compression in extreme fibers of rolled shapes, girders, and built sections, subject to bending, gross section.

(a) When compression flange is supported laterally its full length by embedment in concrete or by other means................................	24,000	30,000
(b) *When compression flange is partially supported or is unsupported..........................	$24{,}000 - 6.67\dfrac{L^2}{b^2}$	$30{,}000 - 8.33\dfrac{L^2}{b^2}$

* Note. Continuous or cantilever beams or girders may be proportioned for negative moment at interior supports for an allowable unit stress 20 per cent higher than permitted by above formula but in no case exceeding allowable unit stress for compression flange supported its full length. If cover plates are used, the allowable stress at the point of cut-off shall be as determined by the formula.

For values of L/b not to exceed 25 for silicon steel, or 20 for nickel steel,

where L = length, in inches, of unsupported flange between lateral connections, knee braces, or other points of support. For continuous beams and girders, L may be taken as the distance from interior support to point of dead load contraflexure if this distance is less than that designated above.

b = flange width in inches.

Compression, splice material, gross section	24,000	30,000
Stress in extreme fibers of pins	36,000	44,000
Shear in plate girder webs, gross section	14,000	17,500
Diagonal tension in webs of girders and rolled beams at sections where maximum shear and bending occurs simultaneously	24,000	27,000
Shear in pins	18,000	22,000
Bearing on pins (not subject to rotation)	32,000	40,000
Bearing on milled stiffeners and other parts in contact	36,000	44,000
Bearing on pins subject to rotation (not due to deflection)	16,000	18,000
Bearing on rollers and rockers: Same formulas as in Article 3.4.2.		

3.4.9. Unit Stresses for Welding

The allowable unit stresses in pounds per square inch of effective area of weld shall be as given in the current specifications of the American Welding Society for Welded Highway and Railway Bridges.

3.4.10. Bearing on Masonry

Bearing on granite masonry	800
Bearing on sandstone and limestone masonry	400
Bearing on concrete:	
Bridge seats, under hinged rockers and bolsters (not subjected to high edge loading by a deflecting beam, girder, or truss)	1,000
Bridge seats, under bearing plates or non-hinged shoes (subjected to high edge loading by the direct bearing, upon the plate or shoe, of a deflecting beam or girder), average	700

(The above bridge seat unit stresses will apply only where the edge of bridge seat projects at least 3 inches (average) beyond edge of shoe or plate. Otherwise, the unit stresses permitted will be 75 per cent of the above amounts.)

* * * *

Section 6. Structural Steel Design

3.6.1. Number of Trusses or Girders

Preferably, through spans shall have only two trusses, arches, or girders.

3.6.2. Spacing of Trusses and Girders

Main trusses, arches, and girders shall be spaced a sufficient distance apart center to center, to be secure against overturning by the assumed lateral forces.

3.6.3. Effective Span

For the calculation of stresses, span lengths shall be assumed as follows:
Beams and girders, distance between centers of bearings.
Trusses, distance between centers of end pins or of bearings.
Floorbeams, distance between centers of trusses or girders.
Stringers, distance between centers of floorbeams.

3.6.4. Effective Depth

For the calculation of stresses, effective depths shall be assumed as follows:
Riveted trusses, distance between centers of gravity of the chords.
Pin-connected trusses, distance between centers of chord pins.
Plate girders, distance between centers of gravity of the flanges, but not to exceed the distance back to back of flange angles.

3.6.5. Alternating Stresses

Members subject to alternating stresses of tension and compression, due to the combination of dead, live, impact, and centrifugal stresses, shall be proportioned for the kind of stress requiring the larger section.

If the alternating stresses occur in succession during one passage of the live load, each shall be increased by 50 per cent of the smaller. The connections of such members shall be proportioned for the sum of the net alternating stresses not so increased.

If the live load increased by 50 per cent produces live load stresses greater than and of opposite sign to the dead load stresses, counters or compression members shall be used. The dead load stresses considered as effective in counteracting the live load stresses shall be those produced by the lightest dead load of the structure as completed or with future alterations.

3.6.6. Combined Stresses

All members subject to combined bending and direct stresses shall be proportioned for the maximum unit stress specified in *Appendix B*. When bending stresses are induced by the component of externally applied loads acting perpendicular to the axis of the member, a shall be assumed equal to $+1$.

3.6.7. Secondary Stresses

The design and details shall be such that secondary stresses will be as small as practicable. Secondary stresses due to truss distortion or floor-beam deflection usually need not be considered in any member the width of which, measured parallel to the plane or distortion, is less than one-tenth of its length. If the secondary stress exceeds 4,000 pounds per square inch for tension members and 3,000 for compression members, the excess shall be treated as a primary stress.

3.6.8. Rolled Beams

Rolled beams shall be proportioned by the moment of inertia method, using their net sections. Rolled beams with cover plates attached shall be designed on the same basis as plate girders.

Suitable stiffeners shall be provided to stiffen the webs of rolled beams at bearings, when the unit shear in the web adjacent to the bearing exceeds 75 per cent of the allowable shear for girder webs.

The compression flanges of rolled beams or stringers supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide adequate support.

3.6.9. Limiting Lengths of Members

For compression members, the ratio of unsupported length to radius of gyration shall not exceed 120 for main members or those in which the major stresses result from dead or live load, or both; and shall not exceed 140 for secondary members or those whose primary purpose is to brace the structure against lateral or longitudinal forces, or to brace or reduce the unsupported length of other members, main or secondary.

In determining the radius of gyration, the area of any portion of a member may be neglected, provided that the area thus neglected shall not be used in calculating the strength of the member. For members resisting calculated compressive stress, all of the specified requirements for minimum thickness of component parts to resist buckling shall apply to material comprising any area thus neglected.

The radius of gyration and the effective area for carrying stress of a member containing perforated cover plates shall be computed for a transverse section through the maximum width of perforation. When perforations are staggered in opposite cover plates, the cross-sectional area of the member shall be considered the same as for a section having perforations in the same transverse plane.

Unsupported length shall be assumed as follows: For the top chords of half-through trusses, the length between panel points laterally supported as required under 3.6.70; for other main members, the length between panel point intersections or centers of braced points or centers of end connections; for secondary members, the length between the centers of the end connections of such member or centers of braced points.

For tension members, except rods, eyebars, cables, and plates, the ratio of unsupported length to radius of gyration shall not exceed 200 for main members and shall not exceed 240 for bracing members.

3.6.10. Deflection

The term "deflection" as used herein shall be the deflection computed in accordance with the assumptions made for loading when computing the stress in the member.

Steel beams or girders having simple or continuous spans shall be designed so that the deflection due to live load plus impact shall not exceed $1/800$ of the span, the span length being considered the distance center-to-center of bearings.

The deflection of cantilever arms due to live load plus impact shall be limited to $1/300$ of the cantilever arm.

When bridges have cross bracing or diaphragms sufficient in depth and strength to insure lateral distribution of loads, the deflection may be computed for the standard loading, considering all beams or girders as acting together and having equal deflection.

Sidewalk live load may be neglected in computing deflection.

The moment of inertia of the gross cross-sectional area shall be used for computing the deflections of beams and girders.

The gross area of each member shall be used in computing the deflections of trusses. When perforated cover plates are used in truss members, the effective area shall be the net volume divided by the length from center to center of perforations.

3.6.11. Depth Ratios

The ratio of the depth to the length of spans, preferably, shall be not less than the following:

For trusses . . . . $1/10$
For plate girders and rolled beams used as girders . . . . $1/25$
For continuous spans, the span length shall be considered as the distance between dead load points of contraflexure.

If depths less than these are used, the sections shall be so increased that the maximum deflection will be not greater than if these ratios had not been exceeded.

3.6.12. Symmetrical Sections

Main members shall be proportioned so that their gravity axes will be as nearly as practicable in the center of the section.

3.6.13. Effective Area of Angles in Tension

The effective area of a single angle tension member, or of each angle of a double tension member in which the angles are connected back to back on the same side of a gusset plate, shall be assumed as the net area of the connected leg plus one-half of the area of the unconnected leg.

If a double angle tension member is connected with the angles back to back on opposite sides of a gusset plate, the full net area of the angles shall be considered as effective. If the angles connect the separate gusset plates, as in the case of a double-webbed truss, and the angles are connected by stay plates located as near the gussets as practicable, or by other effective means, the full net area of the angles shall be considered as effective. If the angles are not so connected, only 80 per cent of the net area shall be considered as effective.

Lug angles may be considered as effective in transmitting stress, provided they are connected with at least one-third more rivets than required by the stress to be carried by the lug angle.

3.6.14. Thickness of Metal

Gusset plates for main members shall not be less than $3/8$ inch in thickness. Other structural steel, except for webs of rolled shapes, fillers, and in railings, shall be not less than $5/16$ inch in thickness. The web thickness of rolled shapes shall not be less than 0.23 inch. Silicon steel plates shall be not more than $1\frac{1}{8}$ inches thick, low-alloy steel plates not more than $1\frac{1}{8}$ inches thick, and nickel steel plates not more than 1 inch in thickness.

Metal exposed to marked corrosive influences shall be increased in thickness or specially protected against corrosion.

3.6.15. Compression Members

Compression members shall be so designed that the main elements of the section will be connected directly to the gusset plates, pins, or other members.

The center of gravity of a built-up section shall coincide as nearly as practicable with the center of the section. Preferably, the segments shall be connected by solid webs.

In 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 $\frac{1}{32}$ of the unsupported distance between the nearest rivet lines or the roots of flanges of rolled segments for carbon steel, $\frac{1}{28}$ for silicon steel, $\frac{1}{26}$ for low-alloy steel, or $\frac{1}{25}$ for nickel steel.

The thickness of cover plates or webs connecting the segments shall be not less than $\frac{1}{40}$ of the unsupported distance between the nearest lines of their connecting rivets or the roots of their rolled flanges for carbon steel, $\frac{1}{36}$ for silicon steel, $\frac{1}{34}$ for low-alloy steel, or $\frac{1}{32}$ for nickel steel.

3.6.16. Web Plates of Solid Rib Arches

The thickness of web plates in solid rib arches shall not be less than the following fractions of the depth as defined in Article 3.6.15, except that when the web is reinforced along its axis with a longitudinal rib of ample cross-sectional area and rigidity, the thickness shall not be less than one-half of these amounts:

$\frac{1}{60}$ for carbon steel.
$\frac{1}{50}$ for silicon steel.
$\frac{1}{48}$ for low-alloy steel.
$\frac{1}{45}$ for nickel steel.

3.6.17. Outstanding Legs of Angles and Bearing Stiffeners

The widths of outstanding legs of angles in compression (except where reinforced by plates) shall not exceed the following:

In girder flanges and bearing stiffeners, 12 times the thickness.

In main members carrying axial stress, 12 times the thickness.

In bracing and other secondary members, 16 times the thickness.

3.6.18. Size of Pins

Pins shall be proportioned for the maximum shears and bending moments produced by the stresses in the members connected. If there are eyebars among the parts connected, the diameter of the pin shall be not less than 0.8 of the width of the widest bar.

Details of Design

3.6.19. Size of Rivets

Rivets shall be of the size shown on the drawings, but generally shall be $\frac{3}{4}$ inch or $\frac{7}{8}$ inch in diameter. Rivets $\frac{5}{8}$ inch in diameter shall not be used in members carrying calculated stress except in $2\frac{1}{2}$-inch legs of angles and in flanges of sections requiring $\frac{5}{8}$ rivets.

The diameter of rivets in angles carrying calculated stress shall not exceed one-fourth of the width of the leg in which they are driven.

In angles whose size is not determined by calculated stress, $\frac{5}{8}$-inch rivets may be used in 2-inch legs, $\frac{3}{4}$-inch rivets in $2\frac{1}{2}$-inch legs, $\frac{7}{8}$-inch rivets in 3-inch legs, and 1-inch rivets in $3\frac{1}{2}$-inch legs.

Structural shapes which do not admit the use of $\frac{5}{8}$-inch diameter rivets shall not be used except in handrails.

3.6.20. Pitch of Rivets

The minimum distance between centers of rivets shall be three times the diameter of the rivet but, preferably, shall be not less than the following:

For 1-inch rivets, $3\frac{1}{2}$ inches.
For $\frac{7}{8}$-inch rivets, 3 inches.
For $\frac{3}{4}$-inch rivets, $2\frac{1}{2}$ inches.
For $\frac{5}{8}$-inch rivets, $2\frac{1}{4}$ inches.

3.6.21. Pitch in Ends of Compression Members

In the ends of compression members the pitch of rivets connecting the component parts of the member shall not exceed four times the diameter of the rivet for a length equal to $1\frac{1}{2}$ times the maximum width of the member. Beyond this point the pitch shall be increased gradually for a length equal to $1\frac{1}{2}$ times the maximum width of the member until the maximum pitch is reached.

3.6.22. Maximum Pitch

The pitch in the line of stress shall not exceed 6 inches or 16 times the thickness of the thinnest outside plate or angle connected, except that in angles having two gage lines with the rivets staggered, the pitch in each line may be twice that given by these rules, with a maximum of 10 inches.

3.6.23. Stitch Rivets

If two or more plates or shapes are in contact, they shall be held together by stitch rivets. In compression members, the stitch rivets shall be spaced on a gage line, in the direction perpendicular to the line of stress not more than 24 times the thickness of the thinnest plate, and, in the line of stress, not more than 12 times the thickness of the thinnest outer plate. In tension members and girders, the stitch rivets shall be spaced, on a pitch, not more than 24 times the thickness of the thinnest outer plate. In tension members composed of two angles in contact, the angles shall be held together by stitch rivets having a maximum pitch of 12 inches.

3.6.24. Edge Distance of Rivets

The minimum distance from the center of any rivet to a sheared edge shall be:

For 1-inch rivets, $1\frac{3}{4}$ inches.
For $\frac{7}{8}$-inch rivets, $1\frac{1}{2}$ inches.
For $\frac{3}{4}$-inch rivets, $1\frac{1}{4}$ inches.
For $\frac{5}{8}$-inch rivets, $1\frac{1}{8}$ inches.

The minimum distance from a rolled or planed edge, except in flanges of beams and channels, shall be:

For 1-inch rivets, $1\frac{1}{2}$ inches.
For $\frac{7}{8}$-inch rivets, $1\frac{1}{4}$ inches.
For $\frac{3}{4}$-inch rivets, $1\frac{1}{8}$ inches.
For $\frac{5}{8}$-inch rivets, 1 inch.

The maximum distance from any edge shall be eight times the thickness of the thinnest outside plate, but shall not exceed 5 inches.

3.6.25. Long Rivets

Rivets subjected to calculated stress and having a grip in excess of $4\frac{1}{2}$ diameters shall be increased in number at least 1 per cent for each additional $\frac{1}{16}$ inch of grip. If the grip exceeds six times the diameter of the rivet, specially designed rivets shall be used.

3.6.26. Rivets in Tension

Rivets in direct tension shall, in general, not be used, but if so used their value shall be one-half that permitted for rivets in shear. Countersunk rivets shall not be used in tension.

3.6.27. Parts Accessible

The accessibility of all parts of a structure for inspection, cleaning, and painting shall be secured by the proper proportioning of members and the design of their details.

3.6.28. Closed Sections and Pockets

Closed sections, and pockets or depressions which will retain water, shall be avoided so far as practicable. Pockets shall be provided with effective drain holes or be filled with waterproofing material.

Details shall be so arranged that the retention of dirt, leaves, and other foreign matter will be reduced to a minimum. Wherever angles are used, either singly or in pairs, preferably they shall be placed with the vertical legs extending downward.

3.6.29. Eccentric Connections

Members, including bracing, shall be so connected that their gravity axes will intersect in a point. Eccentric connections shall be avoided if practicable, but if unavoidable the members shall be so proportioned that the combined fiber stresses will not exceed the allowed axial stress.

3.6.30. Strength of Connections

Except as otherwise provided herein, connections shall be designed for the average of the calculated stress and the strength of the member, but they shall be designed for not less than 75 per cent of the strength of the member.

Connections shall be made symmetrical about the axes of the members in so far as practicable. Connections, except for lacing bars and handrails, shall contain not less than three rivets.

3.6.31. Splices

Compression members, such as chords and trestle columns, in riveted structures shall have milled ends and full contact bearing at the splices.

Splices, whether in tension, compression, bending, or shear, shall be designed for the average of the calculated stress and the strength of the member, or section, but they shall be designed to transmit through the splice material not less than 75 per cent of the strength of the member. The splice shall be as near a panel point as practicable.

Splices in riveted columns and chord members shall be located as close to panel points as possible and, usually, shall be on that side of the panel point where the smaller stress occurs.

The arrangement of the plates, angles, and other splice elements shall be such as to make proper provision for the stresses, both axial and bending, in the component parts of the members spliced.

3.6.32. Indirect Splices

If splice plates are not in direct contact with the parts which they connect, the number of rivets on each side of the joint shall be in excess of the number required for a direct-contact splice to the extent of two extra transverse lines of rivets for each intervening plate.

3.6.33. Fillers

(*a*) *Welding.*

When welding is called for on the plans, it shall comply with current specifications of the American Welding Society, "Welded Highway and Railway Bridges."

(*b*) *Riveting.*

If rivets carrying stress pass through fillers, the fillers preferably shall be extended beyond the connected member, and the extension secured by enough additional rivets to average the total stress in the member over the combined area of the member plus the fillers. As an alternate, the additional rivets may be passed through the connected members without extending the filler.

If the filler is less than ¼-inch thick, it shall not be extended beyond the splicing material and additional rivets are not required. Fillers ¼ inch or more in thickness shall consist of not more than two plates, unless special permission is given by the engineer.

3.6.34. Gusset Plates

Gusset or connecting plates shall be used for connecting main members, except when they are pin-connected. The rivets connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member shall be given consideration. The gusset plates shall be of ample thickness to resist shear, direct stress, and flexure, acting on the weakest or critical section of maximum stress.

Re-entrant cuts, except curves made for appearance, shall be avoided as far as practicable.

If the unsupported edge of a gusset plate exceeds the following number of times its thickness, the edge shall be stiffened:

60 for carbon steel.
50 for silicon steel.
48 for low-alloy steel.
45 for nickel steel.

3.6.35. Stay Plates

The open sides of compression members shall be provided with lacing bars and shall have stay plates as near each end as practicable. Stay plates shall be provided at intermediate points where the lacing is interrupted. In main members, the length of the end stay plates between end rivets shall be not less than 1¼ times the distance between the inner lines of rivets connecting them to the flanges; and the length of intermediate stay plates between end rivets, not less than ¾ of that distance. In lateral struts and other secondary members, the over-all length of end and intermediate stay plates shall be not less than ¾ of the distance between the inner lines of rivets connecting them to the flanges.

The separate segments of tension members composed of shapes may be connected by stay plates or end stay plates and lacing. End stay plates shall have the same minimum length as specified for end stay plates on main compression members, and intermediate stay plates shall have a minimum length of ¾ of that specified for intermediate stay plates on main compression members. The clear distance between stay plates on tension members shall not exceed 3 feet.

The thickness of stay plates shall be not less than 1/50 of the distance between the inner lines of rivets connecting them to the flanges for main members, or 1/60 of that distance for bracing members. Stay plates shall be connected by not less than three rivets on each side, and in members having lacing bars the last rivet in the stay plate, preferably, shall also pass through the end of the adjacent bar.

When perforated cover plates are used instead of lacing bars and stay plates, refer to Article 3.6.36.

3.6.36. Lacing and Perforated Cover Plates

Lacing bars of compression members shall be so spaced that the slenderness ratio of the portion of the flange included between the lacing bar connections will be not more than 40 nor more than two-thirds of the slenderness ratio of the member.

In compression members, the shearing stress normal to the member in the plane of the lacing or perforations shall be that obtained by the following formulas:

For structural carbon steel,

$$V = \frac{P}{100}\left(\frac{100}{\frac{L}{r} + 10} + \frac{\frac{L}{r}}{100}\right)$$

For structural silicon, low-alloy, and nickel steel,

$$V = \frac{P}{100}\left(\frac{100}{\frac{L}{r} + 10} + \frac{\frac{L}{r}}{60}\right)$$

in which V = normal shearing stress in pounds.
P = allowable compressive axial load on members.
L = length of member in inches.
r = radius of gyration of section about the axis perpendicular to plane of lacing in inches.

To the shear so determined shall be added any shear due to the weight of the member or to other forces, and the lacing proportioned for the combined shear.

The shear shall be considered as divided equally among all parallel planes in which there are shear-resisting elements, whether continuous plates or lacing. The section of the lacing bars shall be determined by the formula for axial compression in which L is taken as the distance along the bar between its connections to the main segments for single lacing and 70 per cent of that distance for double lacing.

If the distance across the member between the rivet lines in the flanges is more than 15 inches and a bar with a single rivet in the connection is used, the lacing shall be double and riveted at the intersections.

The angle between the lacing bars and the axis of the member shall be approximately 45 degrees for double lacing and 60 degrees for single lacing.

Lacing bars may be shapes or flat bars. For main members the minimum thickness of flat bars shall be $1/40$ of the distance along the bar between its connections for single lacing and $1/60$ for double lacing. For bracing members the limits shall be $1/50$ for single lacing and $1/75$ for double lacing.

The diameter of rivets in lacing bars shall not exceed one-third of the width of the bar. There shall be at least two rivets in each end of lacing bars riveted to flanges more than 5 inches in width.

Perforated Cover Plates.

When perforated cover plates are used instead of lacing bars and stay plates, the following provisions shall govern their design:

1. The thickness of cover plate shall not be less than $1/50$ of the unsupported distance between the nearest lines of connecting rivets for carbon steel, $1/45$ for silicon steel, $1/42$ for low-alloy steel, and $1/40$ for nickel steel.

2. The transverse distance from the perforation to the nearest line of connecting rivets or point of support shall not exceed twelve (12) times the thickness of plate when measured at the centerline of perforation.
3. The ratio of length (in the direction of stress) to width of perforation shall not exceed two.
4. The clear distance between perforations, in the direction of stress, shall not be less than the unsupported distance between the nearest lines of connecting rivets.
5. The clear distance between the end perforation and the end of the cover plate shall not be less than 1.25 times the unsupported distance between the nearest lines of connecting rivets in the cover plate.
6. The periphery of the perforation at all points shall have a minimum radius of $1\frac{1}{2}$ inches.

3.6.37. Net Section at Pin Holes

In pin-connected tension members, the net section across the pin hole shall be not less than 140 per cent and the net section back of the pin hole not less than 100 per cent of the net section of the body of the member. The ratio of the net width (through the pin hole transverse to the axis of the member) to the thickness of the segment shall not be more than 8.

3.6.38. Net Section of Riveted Tension Members

The net section of a riveted tension member is the sum of the net sections of its component parts. The net section of a part is the product of the thickness of the part multiplied by its least net width.

The net width for any chain of holes extending progressively across 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 = pitch of any two successive holes in the chain
g = gage of the same holes.

The net section of the part is obtained from the chain which gives the least net width.

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 gages from back of angle less the thickness.

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.

The diameter of the hole shall be taken as $\frac{1}{8}$ inch greater than the nominal diameter of the rivet.

3.6.39. Location of Pins

Pins shall be so located with respect to the gravity axes of the members as to reduce to a minimum stresses due to bending.

3.6.40. Pin Plates

When necessary for the required section or bearing area, the section at the pin holes shall be increased on each segment by plates so arranged as to reduce to a minimum the eccentricity of the segment. One plate on each side shall be as wide as the outstanding flanges will allow. At least one full-width plate on each segment shall extend to the far edge of the stay plate and the others not less than 6 inches beyond the near edge. These plates shall be connected by enough rivets to transmit the bearing pressure and so arranged as to distribute it uniformly over the full section.

3.6.41. Forked Ends

Forked ends will be permitted only where unavoidable. There shall be enough pin plates on forked ends to make the section of each jaw equal to that of the member. The pin plates shall be long enough to develop the pin plate beyond the near edge of the stay plate, but not less than the length required by Article 3.6.40.

3.6.42. Pins and Pin Nuts

Pins shall be of sufficient length to secure a full bearing of all parts connected upon the turned body of the pin. They shall be secured in position by hexagonal recessed nuts or by hexagonal solid nuts with washers. If the pins are bored, through rods with cap washers may be used. Pin nuts shall be malleable castings or steel. They shall be secured by cotter pins in the screw ends or else the screw ends shall be long enough to permit burring the threads.

Members shall be held against lateral movement on the pins.

3.6.43. Bolts

Bolted connections shall not be used unless specifically authorized. Bolts shall be unfinished; turned as specified and meeting the requirements of division IV; or an approved form of ribbed bolt.

Bolts in tension shall have single self-locking nuts or double nuts.

3.6.44. Upset Ends

Bars and rods with screw ends shall be upset to provide a section at the root of the thread, which will exceed the net section of the body of the member by at least 15 per cent.

3.6.45. Sleeve Nuts

Sleeve nuts shall not be used.

3.6.46. Expansion and Contraction

The design shall be such as to allow for expansion and contraction at the rate of 1¼ inches in 100 ft. Provision shall be made for changes in length of span resulting from live load stresses. In spans more than 300 feet long, allowance shall be made for expansion and contraction in the floor. The expansion end shall be secured against lateral movement.

3.6.47. Expansion Bearings

Spans of less than 50 feet may be arranged to slide upon metal plates with smooth surfaces, and no provisions for deflection of the spans need be made. Spans of 50 feet and greater shall be provided with rollers, rockers, or sliding plates for expansion purposes and shall also be provided with a type of bearing employing a hinge, curved bearing plates, or pin arrangement for deflection purposes.

3.6.48. Bronze or Copper Alloy Sliding Expansion Bearing

Bronze or copper alloy sliding plates shall be chamfered at the ends. They shall be held securely in position, usually by being inset into the metal of the pedestals and sole plates. Provision shall be made against any accumulation of dirt which will obstruct free movement of the span.

3.6.49. Fixed Bearings

Fixed ends shall be firmly anchored. Bearings for spans less than 50 feet need have no provision for deflection. Spans of 50 feet or greater shall be provided with a type of bearing employing a hinge, curved bearing plates, or pin arrangement for deflection purposes.

3.6.50. Pedestals and Shoes

Pedestals and shoes, preferably, shall be made of cast steel or structural steel. The difference in width between the top and bottom bearing surfaces shall not exceed twice the distance between them. For hinged bearings, this distance shall be measured from the center of the pin. In built pedestals and shoes, the web plates and angles connecting them to the base plate shall be not less than $\frac{5}{8}$ inch thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely. The minimum thickness of the metal in cast steel pedestal shall be 1 inch. Pedestals and shoes shall be so designed that the load will be distributed uniformly over the entire bearing. Spans of 70 feet and greater shall have hinged or pin bearings at both ends.

3.6.51. Rollers

Expansion rollers shall be not less than 6 inches in diameter. They shall be connected by substantial side bars and shall be guided by gearing or other effectual means to prevent lateral movement, skewing, and creeping. The rollers and bearing plates shall be protected from dirt and water as far as practicable, and the design shall be such that water will not be retained and that the roller nests may be inspected and cleaned easily.

3.6.52. Inclined Bearings

For spans on an inclined grade and without hinged bearings, the sole plates shall be beveled so that the masonry surfaces and the sliding surfaces will be level.

3.6.53. Anchor Bolts

Trusses, girders, and I-beam spans shall be securely anchored to the substructure. Anchor bolts shall be swedged or threaded to secure a satisfactory grip upon the material used to embed them in the holes.

The following are the minimum requirements for each bearing:

For I-beam spans the outer beams shall be anchored at each end with 2 bolts, 1 inch in diameter, set 10 inches in the masonry.

For trusses and girders:

Spans 50 feet in length or less, 2 bolts, 1 inch in diameter, set 10 inches in the masonry.

Spans 51 to 100 feet, 2 bolts, $1\frac{1}{4}$ inches in diameter, set 12 inches in the masonry.

Spans 101 to 150 feet, 2 bolts, $1\frac{1}{2}$ inches in diameter, set 15 inches in the masonry.

Spans greater than 150 feet, 4 bolts, $1\frac{1}{2}$ inches in diameter, set 15 inches in the masonry.

Anchor bolts subject to tension shall be designed to engage a mass of masonry which will provide a resistance equal to $1\frac{1}{2}$ times the calculated uplift.

3.6.54. Name Plates

Unless otherwise specified, there shall be a name plate, showing in raised letters and figures the name of the manufacturer and the year of construction, bolted to the bridge near each end, at a point convenient for inspection.

Welding

3.6.55. Welding—General

All welding shall conform to the current Specifications for Welded Highway and Railway Bridges, Design, Construction, and Repair, of the American Welding Society.

This specification provides for welding (and gas cutting) of base metal consisting of structural carbon steel (Article 4.6.2.), or similar low-carbon steel or wrought iron approved by the engineer. Wrought iron shall conform to the requirements of division IV, section 7.

Welding of the following items is permissible under these specifications but shall be done only if called for on the plans or in the special provisions:

1. Floor expansion devices.
2. Railings.
3. Built-up shoes, pedestals, or expansion rockers.
4. Diaphragm connection to beams or other members.
5. Stiffeners except that welding transversely across the tension flanges of beams or girders, which have a flange stress of more than 75 per cent of their capacity, will not be permitted.
6. Filler plates.
7. Stay plate and lacing connections to members.

8. Connections and details of bracing.
9. Caps and base plates for trestle columns except where caps supporting stringers are welded to the sides of the pile.
10. Splicing of steel piling.
11. Sidewalk brackets except main tension connection.
12. Fastening of cover plates to rolled beams.
13. Shop fabricated, all-welded plate girders.
14. Other incidental parts of the structure.

Where a definite amount of riveting is specified as a minimum for connections, the welded connection shall develop an equivalent strength.

Floor System

3.6.56. Stiffness of Floor Members

Floor members shall be designed with special reference to stiffness by making them as deep as economy or the limiting under clearances will permit.

3.6.57. Stringers

Stringers, preferably, shall be riveted between the floorbeams or be continuous over two or more panels.

3.6.58. Cross Frames

In bridges with wooden floors and steel stringers, intermediate cross frames (or diaphragms) shall be placed between stringers more than 20 feet long.

3.6.59. Floorbeams

Floorbeams, preferably, shall be at right angles to the trusses or main girders and shall be rigidly connected thereto. Usually floorbeam connections shall be located above the bottom chord and, in riveted work, the bottom lateral system shall engage both the bottom chord and the floorbeam. In pin-connected trusses, if the floorbeams are located below the bottom chord pins, the vertical posts shall be extended below the pins to make a rigid connection to the floorbeam.

3.6.60. End Floorbeams

There shall be end floorbeams in all square-ended trusses and girder spans and, preferably, in skew spans. End floorbeams for truss spans, preferably, shall be designed to permit the use of jacks for lifting the superstructure. For this case the allowable stresses may be increased 50 per cent.

End floorbeams shall be arranged to permit painting of the side of the beam adjacent to the abutment backwall.

3.6.61. End Panels or Skew Bridges

In skew bridges without end floorbeams, the end panel stringers shall be secured in correct position by end struts connected to the stringers and to the main trusses or girders. The end panel lateral bracing shall be attached to the main trusses or girders and also to the end struts. Adequate provision shall be made for the expansion movement of stringers.

3.6.62. End Connection of Floorbeams and Stringers

The end connection shall be designed for the loads specified. The end connection angles of floorbeams and stringers shall be not less than $3/8$ inch in finished thickness. Except in cases of special end floorbeam details, each end connection for floorbeams and stringers shall be made with two angles. The length of these angles shall be as great as the flanges will permit. Bracket or shelf angles which may be used to furnish support during erection shall not be considered in determining the number of rivets required to transmit end shear.

End connection details shall be designed with special care to provide clearance for the driving of field connection rivets.

Where timber stringers frame into floorbeams, shelf angles with stiffeners shall be provided to carry the whole reaction. Shelf angles shall be not less than $7/16$ inch thick.

No type of floorbeam hanger which will permit the rotation or the longitudinal motion of the floorbeam shall be used.

3.6.63. Sidewalk Brackets

Sidewalk brackets shall be connected in such a way that the bending stresses will be transferred directly to the floorbeams.

3.6.64. Expansion Joints

To provide for expansion and contraction movement, floor expansion joints shall be provided at the expansion ends of all spans and at other points where they may be necessary.

Apron plates, when used, shall be designed to bridge the joint and to prevent, so far as practicable, the accumulation of roadway debris upon the bridge seats. Preferably, they shall be connected rigidly to the end floorbeam.

Bracing

3.6.65. General

Bracing shall be composed of angles or other shapes.

If a double system of bracing is used, both systems may be considered effective simultaneously if the members meet the requirements both as tension and compression members. The members shall be connected at their intersections.

3.6.66. Minimum Size of Angles

The smallest angle used in bracing shall be 3 by $2\frac{1}{2}$ inches. There shall be not less than three rivets in each end connection of the angles.

3.6.67. Lateral Bracing

Through truss spans, deck truss spans, and spandrel braced arches shall have top and bottom lateral bracing.

Lateral bracing is not required for deck plate girder or beam spans of less than 125 feet which have composite timber-concrete slabs or a steel or concrete floor keyed or attached to the supporting members and which are supported laterally by substantial cross frames or diaphragms.

Deck plate girder or beam spans more than 40 feet in length which have a timber floor shall have at least one system of lateral bracing in combination with substantial cross frames or diaphragms.

The spacing of cross frames or diaphragms shall not exceed 25 feet. Adequate temporary bracing shall be placed, if necessary during construction.

The lateral bracing of compression chords preferably shall be as deep as the chords and effectively connected to both flanges.

3.6.68. Portal and Sway Bracing

Through truss spans shall have portal bracing, preferably, of the 2-plane or box type, rigidly connected to the end post and the top chord flanges, and as deep as the clearance will allow. If a single-plane portal is used, it shall be located, preferably, in the central transverse plane of the end posts, with diaphragms between the webs of the posts to provide for a distribution of the portal stresses. The portal bracing shall be designed to take the full end reaction of the top chord lateral system, and the end posts shall be designed to transfer this reaction to the truss bearings.

Deck truss plans shall have sway bracing in the plane of the end posts and at all intermediate panel points. This bracing shall extend the full depth of the trusses below the floor system. The end sway bracing shall be proportioned to carry the entire upper lateral stress to the supports through the end posts of the truss.

Through truss spans shall have sway bracing 5 feet or more deep at each intermediate panel point. Top lateral struts shall be at least as deep as the top chord.

3.6.69. Deck Plate Girder Spans

Deck plate girder spans shall be provided with cross frames at each end, proportioned to resist the lateral forces, and shall have intermediate cross frames at intervals not exceeding 25 feet. Cross frames shall be connected to the outstanding legs of the stiffener angles and to the girder flanges.

3.6.70. Half-Through Truss Spans

The vertical truss members and the floorbeams and their connections in half-through truss spans shall be proportioned to resist a lateral force of not less than 300 pounds per linear foot, applied at the top chord panel points of each truss.

The top chord shall be considered as a column with elastic lateral supports at the panel points. The critical buckling force of the column, so determined, shall exceed the maximum force from dead load, live load, and impact in any panel of the top chord by not less than 50 per cent.*

* For a discussion of columns with elastic lateral supports, refer to Timoshenko, *Theory of Elastic Stability*, McGraw-Hill Book Co., first edition, page 122.

3.6.71. Through Plate Girder Spans

Through plate girder spans shall be stiffened against lateral deformation by means of gusset plates, or knee braces with solid webs, attached to the stiffener angles and floorbeams. These braces generally shall extend to the clearance line. If the unsupported length of the inclined edge of the gusset plate exceeds 60 times its thickness, the gusset plate shall have 1 or 2 stiffening angles riveted along its edge.

3.6.72. Bracing of Long Columns

The bracing of long columns shall be designed to fix the column about both axes at or near the same point.

Plate Girders

3.6.73. General

Plate girders shall be proportioned by the moment of inertia method. In calculating the net moment of inertia, the gravity axis of the gross section shall be used and the moment of inertia of all holes each side of the axis shall be deducted. The tensile stress shall be computed from the moment of inertia of the entire net section and the compressive stress from the moment of inertia of the entire gross section.

The compression flanges of plate girders supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide adequate support.

3.6.74. Flange Sections

The flange angle shall form as large a part of the area of the flange as practicable. Side plates shall not be used except where flange angles exceeding $\frac{7}{8}$ inch in thickness otherwise would be required.

The gross area of the compression flange, except for composite design, shall be not less than the gross area of the tension flange.

Flange plates shall be of equal thickness, or shall decrease in thickness from the flange angles outward. No plate shall have a thickness greater than that of the flange angles.

If flange plates are used, at least one plate of the top flange shall extend the full length of the girder, except where the flange is to be covered with concrete. Any additional flange plates shall extend at least 1 foot beyond the theoretical end but shall be not less than $2d + 3$ feet in length, and there shall be a sufficient number of rivets at each end of each plate to develop its full stress value before the end of the next outside plate is reached. The term "d" equals the depth of girder.

3.6.75. Thickness of Web Plates

The thickness of web plates, except those to be encased in concrete or those properly stiffened longitudinally, shall be not less than $\frac{1}{170} D$ for carbon steel, not less than $\frac{1}{145} D$ for silicon steel, not less than $\frac{1}{140} D$ for low-alloy steel, and not less than $\frac{1}{130}$ D for nickel steel, in which "D" is the clear distance in inches between flanges (including side plates).

When one longitudinal stiffener is used, properly located with respect to the toe of the compression flange and of adequate rigidity, both as provided in Article 3.6.81, the thickness of web plates shall be not less than $\frac{1}{270}$ D for carbon steel, not less than $\frac{1}{230}$ D for silicon steel, not less than $\frac{1}{220}$ D for low-alloy steel, and not less than $\frac{1}{210}$ D for nickel steel, in which "D" is the clear distance in inches between flanges (including side plates).

3.6.76. Flange Rivets

The number of rivets connecting the flange angles to the web plate shall be sufficient to develop the increment of flange stress transmitted to the flange angles, combined with any load that is applied directly to the flange.

Legs of angles 5 inches or greater in width, connected to either web plates or cover plates, shall have two lines of rivets. Cover plates over 14 inches wide shall have four lines of rivets.

3.6.77. Flange Splices

Splices in flange parts shall not be used except by special permission of the engineer. In general, not more than one part shall be spliced at the same cross section. If practicable, splices shall be located at points where there is an excess of section. The strength of the splice shall be as specified in Article 3.6.31. Flange angle splices shall consist of two angles, one at each end of the girder.

3.6.78. Web Splices

Web plates shall be spliced symmetrically by plates on each side. The strength of the splice for shear and moment shall be as specified in Article 3.6.31. The splice plates for shear shall extend the full depth of the girder between flanges. In the splice there shall be not less than two rows of rivets on each side of the joint.

3.6.79. Transverse End Stiffeners

Over the end bearings of plate girders there shall be stiffener angles, the outstanding legs of which shall extend as nearly as practicable to the outer edge of the flange angles. End stiffeners shall be proportioned for bearing on the outstanding legs of the flange angles, no allowance being made for the portions of the legs fitted to the fillets of the flange angles. End stiffeners shall be arranged, and there shall be a sufficient number of rivets in their connection to the web to transmit the entire end reaction to the bearings. They shall not be crimped.

3.6.80. Transverse Intermediate Stiffeners

The webs of plate girders shall be stiffened at intervals not greater than:

1. 6 feet, or the clear unsupported depth of the web
2. The distance given by the formula

$$d = \frac{9000\,t}{\sqrt{s}}$$

in which

d = the clear distance between stiffeners in inches.
t = the thickness of web plates in inches.
s = the average unit shearing stress, gross section in the web at the point considered.

Intermediate transverse stiffeners may be omitted if the depth of the web between the flange angles, or between the side plates, if there are side plates, is less than 60 times the thickness of web for carbon steel, 55 times for silicon steel, 52 times for low-alloy steel, or 50 times for nickel steel.

Intermediate stiffener angles shall be placed at points of concentrated loading and shall be so designed as to transmit the reactions to the girder web. Such stiffeners shall not be crimped.

Intermediate transverse stiffeners for plate girder webs preferably should be in pairs. However, they may be any one of the following types:

1. A single plate or angle welded to the plate girder web with intermittent welds.
2. Either single or double angles riveted to the plate girder web.

When angles or plates are used on one side only, they shall be attached to the outstanding leg of the compression flanges.

The width of a plate or outstanding leg of an angle transverse intermediate stiffener shall not be more than 16 times its thickness, and not less than 2 inches plus 1/30 of the depth of the girder.

The minimum moment of inertia of any type of transverse intermediate stiffener shall be:

$$I\ \text{Min} = \frac{dt^3J}{11.0}$$

where

I = the min. required moment of inertia in inches^4 of one transverse stiffener. For a stiffener on both sides of the web plate, I is to be computed about the centerline of the web plate. For a stiffener on one side of the web plate, I is to be computed about the edge in contact with the web plate.
d = the required clear distance between transverse stiffeners.
D = the clear distance between flanges.

$$J = 3.75\left(\frac{D}{d}\right)^4$$

t = thickness of web plates in inches.

3.6.81. Longitudinal Stiffeners

The gage line of the longitudinal stiffener shall be 1/5 D from the toe of the compression flange. The longitudinal stiffener shall be proportioned so that:

$$I_B = Dt^3\left(2.4\,\frac{d^2}{D^2} - 0.13\right)$$

where

I_E = Minimum moment of inertia of longitudinal stiffener about the edge in contact with web plate.

D = Clear distance between flanges.

t = Thickness of web plate.

d = Clear distance between transverse stiffeners.

Longitudinal stiffeners need not be continuous and may be cut at their intersection with transverse stiffeners.

3.6.82. Ends of Through Girders

The upper corners of through plate girders, where exposed, shall be rounded to a radius consistent with the size of the flange angles and the vertical height of the girder above the roadway. The first flange plate, or a plate of the same width, shall be bent around the curve and continued to the bottom of the girder. In a bridge consisting of 2 or more spans, only the corners at the extreme ends of the bridge need be so rounded unless the spans have girders of different heights. In such a case the higher girders shall have their top flanges curved down at the ends to meet the top corners of the girders in the adjacent spans.

3.6.83. Sole Plates

Sole plates of plate girders shall have a thickness of not less than ¾ inch and not less than the thickness of the flange angles plus ⅛ inch. Preferably, they shall not be longer than 18 inches.

3.6.84. Masonry Bearings

Ends of girders on masonry shall be so supported on metal pedestals that the bottom flanges will be above the bridge seat, preferably not less than 6 inches.

3.6.85. Camber

In general, camber will not be required in plate girders except for long spans or special conditions. When required, it shall be in the amount specified by the engineer.

Trusses

3.6.86. General

Preference will be given to trusses with single intersection web systems. Members shall be symmetrical about the central plane of the truss.

Trusses, preferably, shall have inclined end posts. Laterally unsupported hip joints shall be avoided.

3.6.87. Top Chords and End Posts

Top chords and end posts usually shall be made of two side segments with one cover plate, and with stay plates and lacing on the open side.

If the shape of the truss permits, compression chords shall be continuous. The splice shall be as near the panel point as practicable and, preferably, on the side of the panel point where the smaller stress occurs.

3.6.88. Bottom Chords

The bottom chords of riveted trusses generally shall be spliced near panel points and on the side of the panel points where the smaller stress occurs.

In bottom chords composed of angles the vertical legs of the angles shall preferably extend downward.

3.6.89. Working Lines and Gravity Axes

In compression members of unsymmetrical section, such as chord sections formed of side segments and a cover plate, the gravity axis of the section shall coincide as nearly as practicable with the working line, except that eccentricity may be introduced to counteract dead load bending. In 2-angle bottom chord or diagonal members, the working line may be taken as the gage line nearest the back of the angle.

3.6.90. Camber

The length of the truss members shall be such that the camber will be equal to or greater than the deflection produced by the dead load.

3.6.91. Riveted Tension Members in Pin-Connected Trusses

In pin-connected trusses, the hip verticals and members performing similar functions, and the bottom chords in the first two panels at each end, shall be riveted members.

3.6.92. Counters

If web members are subject to reversal of stress, their end connection shall be riveted. Counters, preferably, shall be rigid. Adjustable counters, if used, shall have open turnbuckles, and in the design of these members an allowance of 10,000 pounds shall be made for initial stress. Only one set of diagonals in any panel shall be adjustable. Sleeve nuts and loop bars shall not be used.

3.6.93. Eyebars

The thickness of eyebars shall be not less than ⅛ of the width, nor less than ½ inch, and not greater than 2 inches. The section of the head through the center of the pin hole shall exceed that of the body of the bar by at least 35 per cent. The form of the head shall be submitted to the engineer for approval before the bars are made. The diameter of the pin shall be not less than 0.8 of the width of the widest bar through which it passes.

3.6.94. Packing of Eyebars

The eyebars of a set shall be symmetrical about the central plane of the truss and as nearly parallel as practicable. The inclination of any bar to the plane of the truss shall not exceed $\frac{1}{16}$ inch to a foot. Bars shall be as close together as practicable and held against lateral movement, but they shall be so arranged that adjacent bars in the same panel will be separated by at least $\frac{1}{2}$ inch.

Intersecting diagonal bars not far enough apart to clear each other at all times shall be clamped together at the intersection.

Steel filling rings shall be provided, if needed, to prevent lateral movement of eyebars or other members connected on the pin.

3.6.95. Diaphragms

There shall be diaphragms in the trusses at the end connections of floorbeams.

The gusset plates engaging the pedestal pin at the end of the truss shall be connected by a diaphragm. Similarly, the webs of the pedestal shall, if practicable, be connected by a diaphragm.

There shall be a diaphragm between gusset plates engaging main members if the end tie plate is 4 feet or more from the point of intersection of the members.

3.6.96. Sole Plates

Sole plates of trusses shall be not less than $\frac{3}{4}$ inch thick.

3.6.97. Masonry Bearings

Trusses on masonry shall be so supported on metal plates or pedestals that the bottom chords will be above the bridge seat, preferably not less than 6 inches.

Viaducts

3.6.98. Type

Viaducts shall consist of rolled beam sections, plate girders, or riveted trusses supported on bents, and usually arranged in alternate tower spans and free spans.

3.6.99. Bents and Towers

Bents, preferably, shall be composed of two supporting columns, and the bents usually shall be united in pairs to form towers.

3.6.100. Batter

Bents, preferably, shall have a sufficient spread at the base to prevent uplift under the assumed lateral loadings. In general, the width of a bent at its base shall be not less than one-third of its height.

3.6.101. Single Bents

Single bents shall have hinged ends or else shall be designed to resist bending.

3.6.102. Bracing

Towers shall be braced, both transversely and longitudinally, with stiff members having riveted connections. The sections of members of longitudinal bracing in each panel shall not be less than those of the members in corresponding panels of the transverse bracing.

Column splices shall be above and close to the panel points of the bracing.

Horizontal diagonal bracing shall be placed in all towers having more than two vertical panels, at alternate intermediate panel points.

3.6.103. Bottom Struts

The bottom struts of viaduct towers shall be strong enough to slide the movable shoes with the structure unloaded, the coefficient of friction being assumed at 0.25. Provision for expansion of the tower bracing shall be made in the column bearings.

3.6.104. Depth of Girders

The depth of girders in viaducts preferably shall be uniform.

3.6.105. Girder Connections and Bracing

Girders of tower spans shall be fastened at each end to the tops of the columns or to the cross girders. Preferably, there shall be a line of girders resting directly over the columns. One end of the girders between towers shall be riveted to the support, and there shall be an effective expansion bearing at the other end. No bracing or sway frame shall be common to abutting spans.

If girders are not supported directly on the column, provision shall be made for the transmission of the longitudinal forces to the tower bracing.

3.6.106. Sole and Masonry Plates

Sole plates, masonry plates, and cap plates shall be not less than ¾ inch thick.

APPENDIX B

AMERICAN RAILWAY ENGINEERING ASSOCIATION

SPECIFICATIONS FOR STEEL RAILWAY BRIDGES, 1956

Pages 15-1-3 to 15-1-23 and 15-1-38 to 15-1-43

A. DESIGN AND MANUFACTURE

Proposals and Drawings

1. Definition of Terms

The term "company" means the railway company party to the contract. The term "engineer" means the chief engineer of the company or his authorized representatives. The term "inspector" means the inspector representing the company. The term "contractor" means the manufacturing or fabricating contractor party to the contract.

2. Proposals

Bidders shall submit proposals conforming to the terms in the letter of invitation. The proposals preferably shall be based on plans and specifications furnished by the company. Such plans will show the conditions determining the design of the bridge, the general dimensions, stresses, and typical details.

If the invitation requires the contractor to furnish the design, the invitation shall state the general conditions at the site, such as the track spacing, character of foundations, presence of old structures, traffic conditions, etc.

3. Shop Drawings

(a) After the contract has been awarded and before the work is begun, the contractor shall submit to the engineer, for approval, prints in duplicate of the stress sheets and shop drawings, unless such drawings shall have been prepared by the company.

Shop drawings shall be made in ink on the dull side of the tracing cloth, 24 in. by 36 in. in size, including margins. The margin at the left end shall be 1½ in. wide, and the others ½ in. The title shall be in the lower right-hand corner. Shop drawings may be made in pencil on tracing cloth or vellum, provided that they are photographically reproduced on tracing cloth after completion of the fabrication and/or the erection of the steel.

No change shall be made on any approved drawing without the written consent of the engineer.

The contractor shall furnish to the company as many prints of the drawings as are necessary for carrying out the work.

Legible tracings of the drawings, in good condition, shall be delivered to, and become the property of, the company upon completion of the contract.

(b) The contractor shall be responsible for the correctness and completeness of his drawings, although the drawings may have been approved by the engineer.

(c) If material is ordered by the contractor before the drawings have been approved, the contractor shall assume the risk.

4. Drawings to Govern

If the drawings and the specifications conflict, the drawings shall govern.

5. Patented Devices

The contractor shall protect the company against claims arising from the use of patented devices or parts proposed by him.

6. Notice to Engineer

No material shall be rolled or work done before the engineer has been notified where the orders have been placed.

General Features of Design

7. Materials

Structures shall be made wholly of structural steel except where otherwise specified. Rivet steel shall be used for rivets only. Forged steel shall be used for large pins, large expansion rollers, and other parts if specified by the engineer. Preferably, cast steel shall be used for shoes, rockers, and bearings. Cast iron may be used only where specifically authorized by the engineer.

8. Types of Bridges

The preferred types of bridges are as follows:

Rolled beams for spans up to 50 ft.
Plate girders for spans up to 125 ft.
Riveted trusses for spans 100 ft or longer.
Pin-connected trusses for spans 300 ft or longer may be used where conditions warrant.

9. Spacing of Trusses, Girders, and Stringers

The distance between centers of trusses or girders shall be sufficient to prevent overturning by the specified lateral forces. In no case shall it be less than $\frac{1}{20}$ of the span for through spans, nor $\frac{1}{15}$ of the span for deck spans.

The girders of deck girder spans and the stringers of deck and through spans shall be spaced not less than 6 ft 6 in. between centers, except that if there are four stringers or girders under one track, they shall be in pairs, one pair symmetrical about each rail.

10. Depth Ratios

The depth of trusses preferably shall be not less than $\frac{1}{10}$ of the span. The depth of plate girders preferably shall be not less than $\frac{1}{12}$ of the span. The depth of rolled beams used as girders and the depth of solid floors preferably shall be not less than $\frac{1}{15}$ of the span.

Where shallower depth spans are required, they shall be proportioned so that all deflection from the combined effect of live load and impact shall not exceed that obtained for spans of the depth ratios given above.

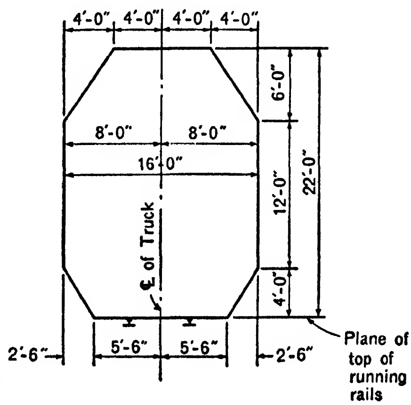


Fig. 1.

11. Clearances

The clearances on straight track shall not be less than those shown in Fig. 1. On curved track the clearance shall be increased to allow for the overhanging and the tilting of a car 85 ft long, 60 ft between centers of trucks, and 14 ft high.

Where legal requirements provide greater clearances, such requirements shall govern.

The superelevation of the outer rail shall be as specified by the engineer. The distance from the top of rail to the top of tie shall be taken as 8 in., or as specified by the engineer.

12. Dimensions for Calculation

For the calculation of stresses:

The length shall be:

For trusses and girders, the distance between centers of bearings.

For floorbeams, the distance between centers of trusses or girders.

For stringers, the distance between centers of floorbeams.

For timber cross ties, the distance between centers of stringers or deck girders.

The depth shall be:

For pin-connected trusses, the distance between center lines of chord pins.

For riveted trusses, the distance between gravity axes of chords.

13. End Floorbeams

Spans with floor systems shall have end floorbeams unless otherwise specified. Except where other means are provided, end floorbeams shall be proportioned for lifting the span without exceeding the design unit stresses more than 50 per cent.

14. Skew Bridges

At the ends of skew bridges, the ends of the supports for each track shall be square with the line of the track.

15. Open Deck

Timber bridge ties shall be not less than 10 ft long, and spaced not more than 6 in. apart. They shall be secured against bunching.

Loads and Stresses

16. Loads and Forces

Bridges shall be proportioned for the following loads and forces:

(a) Dead load.
(b) Live load.
(c) Impact.
(d) Centrifugal force.
(e) Other lateral forces.
(f) Longitudinal force.

Stresses from each of these loads and forces shall be shown separately on the stress sheet.

17. Dead Load

In estimating the weight for the purpose of computing dead load stresses, the following unit weights shall be used:

	Lb per Cu Ft
Steel	490
Concrete	150
Sand, gravel, and ballast	120
Asphalt-mastic and bituminous macadam	150
Granite	170
Paving bricks	150
Timber	60

The track rails, inside guard rails, and fastenings shall be assumed to weigh 200 lb per lin ft for each track.

18. Live Load

The recommended live load for each track is the Cooper E 72 load, shown in Fig. 2.

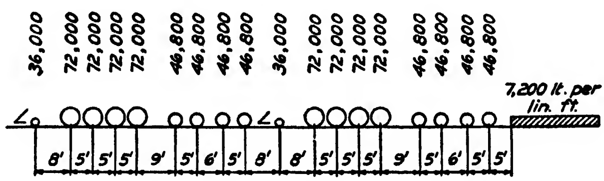


FIG. 2.

The engineer shall specify the live load to be used, such load to be proportional to the recommended load, with the same axle spacing.

For bridges on curves, provision shall be made for the increased proportion carried by any truss, girder, or stringer due to the eccentricity of the load.

For members receiving load from more than one track, the proportions of full live load on the tracks shall be as follows:

For two tracks, full live load.
For three tracks, full live load on two tracks and ½ on the other track.
For four tracks, full live load on two tracks, ½ on one track, and ¼ on the remaining one.
For more than four tracks, as specified by the engineer.

The selection of the tracks for these proportions shall be such as will give the greatest live load stress.

19. Distribution of Live Load

(a) Where an open or ballasted track is carried on transverse steel beams without stringers, the portion of any axle load on a single beam shall be as follows:

$$P = \frac{KAd}{S}$$

where A = Axle load
P = Load on a beam from one track
d = Beam spacing
S = Axle spacing
K = $1\frac{1}{8}$ for single track and $1\frac{1}{4}$ for double track.

The load P shall be assumed distributed as concentrated loads on the beam under each rail.

The effects of eccentricity of track and centrifugal force shall be included.

(b) Timber bridge ties shall be designed on the assumption that the maximum wheel load on a rail is uniformly distributed over three ties, and is applied without impact.

(c) Where the track is carried on longitudinal beams or girders with a ballast floor, the live load shall be considered as uniformly distributed over those beams within a width of 14 ft for single track, but not to exceed the distance between track centers for multiple tracks. In open deck structures where two or more longitudinal beams per rail are properly diaphragmed and symmetrically spaced under the rail, they may be considered as equally loaded.

20. Impact

(a) To the axle loads specified in Sec. A, Art. 18, there shall be added impact forces, applied at the top of rail and distributed thence to the supporting members, comprising:

1. The rolling effect:
Vertical forces due to the rolling of the train from side to side, acting downward on one rail and upward on the other, the forces on each rail being equal to 10 per cent of the axle loads.

2. The direct vertical effect:
Downward forces, distributed equally to the two rails and acting normal to the top-of-rail plane, due, in the case of steam locomotives, to hammer blow, track irregularities, speed effect and car impact, and equalling the following percentage of the axle loads:

a. For beam spans, stringers, girders, floorbeams, posts of deck truss spans carrying load from floorbeam only, and floorbeam hangers—

For L less than 100 ft. . . . . $60 - \frac{L^2}{500}$

For L 100 ft or more. . . . . $\frac{1800}{L - 40} + 10$

b. For truss spans . . . . $\dfrac{4000}{L + 25} + 15$

or due, in the case of rolling equipment without hammer blow (diesels, electric locomotives, tenders alone, etc.) to track irregularities, speed effect and car impact, and equalling the following percentage of axle loads:

For L less than 80 ft . . . . $40 - \dfrac{3L^2}{1600}$

For L 80 ft or more . . . . $\dfrac{600}{L - 30} + 16$

L = length, in feet, center to center of supports for stringers, transverse floorbeams without stringers, longitudinal girders and trusses (main members),

or L = length, in feet, of the longer adjacent supported stringer, longitudinal beam girder or truss for impact in floorbeams, floorbeam hangers, subdiagonals of trusses, transverse girders, supports for longitudinal and transverse girders and viaduct columns.

(b) For members receiving load from more than one track, the impact percentage shall be applied to the static live load on the number of tracks shown below:

Load received from:

Two tracks:

For L less than 175 ft . . . . Full impact on two tracks.

For L from 175 ft to 225 ft . . . . Full impact on one track and a percentage of full impact on the other as given by the formula $450–2L$.

For L greater than 225 ft . . . . Full impact on one track and none on the other.

More than two tracks:

For all values of L . . . . Full impact on any two tracks.

21. Centrifugal Force

On curves, a centrifugal force corresponding to each axle load shall be applied horizontally through a point 6 ft above the top of rail measured along a line perpendicular to the line joining the tops of the rails and equidistant from them. This force shall equal the percentage $0.00117S^2D$ of the specified axle load without impact.

S = speed in mph

D = degree of curve

On curves, each axle load on each track shall be applied vertically through the point defined in the first paragraph of this article. The impact forces shall be computed and applied as specified in Sec. A, Art. 20.

Preferably, the section of the stringer, girder, or truss on the high side of the superelevated track shall be used also for the member on the low side, if this is greater than the computed section of the latter. If, under the foregoing provisions, the member on the low side shall be computed for the increased live load forces due thereto, no impact forces shall be added except upon estimating a speed consistent therewith, including the relief from the centrifugal force at such speed. If such computations require greater section in the member on the low side, it shall be so designed.

The table following gives the permissible speeds and the corresponding centrifugal force percentages for curves with the amounts of superelevation shown. It is based on a maximum speed of 100 mph and a maximum superelevation of 7 in. resulting in a maximum centrifugal force of 17.5 per cent.

If the conditions at the site restrict the permissible speeds to less than those shown in the table, the centrifugal force percentage shall be taken for the greatest speed expected.

D	E	S	C	D	E	S	C
0°–10′		100	1.95	2°–30′	7	77	17.5
0°–20′		100	3.90	3°–00′	7	71	17.5
0°–30′	0.33	100	5.85	3°–30′	7	65	17.5
0°–40′	1.44	100	7.80	4°	7	61	17.5
0°–50′	2.56	100	9.75	5°	7	55	17.5
1°–00′	3.67	100	11.7	6°	7	50	17.5
1°–15′	5.33	100	14.6	8°	7	43	17.5
1°–30′	7	100	17.5	10°	7	39	17.5
1°–45′	7	93	17.5	15°	7	32	17.5
2°–00′	7	87	17.5	20°	7	27	17.5
2°–15′	7	82	17.5				

D = degree of curve
E = superelevation in inches
S = permissible speed in mph
C = centrifugal force in percentage of the live load

$$C = 0.00117\,S^2D = 1.755\,(E + 3)$$

$$E = \frac{S^2D}{1500} - 3 = \frac{C - 5.265}{1.755}$$

$$S^2 = \frac{1500}{D}(E + 3)$$

22. Wind on Loaded Bridge

The wind force shall be considered as a moving load acting in any horizontal direction. On the train it shall be taken at 300 lb per lin ft on the one track, applied 8 ft above the top of rail. On the bridge it shall be taken at 30 lb per sq ft of the following surfaces:

(a) For girder spans, 1½ times the vertical projection of the span.
(b) For truss spans, the vertical projection of the span plus any portion of the leeward trusses not shielded by the floor system.
(c) For viaduct towers and bents, the vertical projections of all columns and tower bracing.

The wind force on girder spans and truss spans, however, shall not be taken at less than 200 lb per lin ft for the loaded chord or flange, and 150 lb per lin ft for the unloaded chord or flange.

23. Wind on Unloaded Bridge

If a wind force on the unloaded bridge of 50 lb per sq ft of surface as defined in

Sec. A, Art. 22, combined with the dead load, produces greater stresses than those produced by the wind forces specified in Sec. A, Art. 22, combined with the stresses from dead load, live load, impact, and centrifugal force, the members wherein such greater stresses occur shall be designed therefor.

24. Nosing of Locomotives

For bracing systems or for longitudinal members entirely without a bracing system, the lateral force to provide for the effect of the nosing of locomotives (in addition to the other lateral forces specified) shall be a single moving force of 20,000 lb applied at the top of the rail, in either lateral direction, at any point of the span. The resulting vertical forces shall be disregarded.

25. Stability of Spans and Towers

In calculating the stability of spans and towers, the live load on one track shall be 1200 lb per lin ft, taken without impact. On multiple-track bridges, this live load shall be on the leeward track.

26. Bracing Between Compression Members

The lateral bracing of the compression chords or flanges of trusses and deck girders and between the posts of viaduct towers shall be proportioned for a transverse shear in any panel equal to 2½ per cent of the total axial stress in both members in that panel, in addition to the shear from the specified lateral forces.

27. Longitudinal Force

The longitudinal force resulting from the starting and stopping of trains shall be the larger of:

(a) Force due to braking.
    Fifteen percent of the live load without impact.
(b) Force due to traction.
    Twenty-five per cent of the weight on the driving wheels, without impact.

The longitudinal force shall be taken on one track only and shall be assumed to act 6 ft above the top of the rail.

For bridges where, by reason of continuity of members or frictional resistance, much of the longitudinal force will be carried directly to the abutments (such as ballasted deck bridges of only 3 or 4 spans), only ½ of the longitudinal force shall be considered effective.

28. Reversal of Stress

Members subject to reversal of stress (whether axial, bending, or shearing) during the passage of the live load shall be proportioned as follows:

Determine the maximum stress of one sign, and the maximum stress of the opposite sign and increase each by 50 per cent of the smaller. Proportion the member so that it will be capable of resisting either stress so increased. The connections shall be proportioned for the sum of the maximum stresses.

29. Combined Stresses

Members subject to both axial and bending stresses, including bending due to floorbeam deflection, shall be so proportioned that the combined fiber stresses will not exceed the allowed axial stress. In members continuous over panel points, only ¾ of the bending stress computed as for simple beams shall be added to the axial stress.

Members subject to stresses produced by a combination of dead load, live load, impact, and centrifugal force, with other lateral forces and with longitudinal force, or with bending due to such forces, may be proportioned for unit stresses 25 per cent greater than those specified in Sec. A, Art. 31; but the section of the number shall not be less than that required for the combination of dead load, live load, impact, and centrifugal force.

30. Secondary Stresses

The design and details shall be such that secondary stresses will be as small as practicable. Secondary stresses due to truss distortion usually need not be con-considered in any member the width of which, measured parallel to the plane of distortion, is less than 1/10 of its length. If the secondary stress exceeds 4000 psi for tension members and 3000 psi for compression members, the excess shall be treated as a primary stress.

Unit Stresses

31. Unit Stresses

The allowable unit stresses to be used in proportioning the parts of a bridge shall be as follows:

	Lb per Sq In.
(a) Structural and Rivet Steel.	
Axial tension, structural steel, net section	18,000
Tension in extreme fibers of rolled shapes, girders, and built sections, subject to bending	18,000
Axial compression, gross section:	
For stiffeners of plate girders	18,000
For compression members centrally loaded and with values of l/r not greater than 140:	
Riveted ends	$15{,}000 - \frac{1}{4}\frac{l^2}{r^2}$
Pin ends	$15{,}000 - \frac{1}{3}\frac{l^2}{r^2}$

l = length of member in inches.
r = least radius of gyration of member, in inches.

For compression members with values of l/r greater than 140 and for compression members of known eccentricity, see Appendix A.

	Lb per Sq In.
Compression in extreme fibers of rolled shapes, girders, and built sections, subject to bending (for values of l/b not greater than 40)	$18{,}000 - 5\frac{l^2}{b^2}$
l = length, in inches, of unsupported flange between lateral connections or knee braces.	
b = flange width, in inches.	
Diagonal tension in webs of girders and rolled beams at sections where maximum shear and bending occur simultaneously	18,000
Stress in extreme fibers of pins	27,000
Shear in plate girder webs, gross section	11,000
Shear in power-driven rivets and pins	13,500
Shear in turned bolts and hand-driven rivets	11,000
Bearing on pins	24,000
Bearing on power-driven rivets, milled stiffeners, and other steel parts in contact	27,000
(Rivets driven by pneumatically or electrically operated hammers are considered power-driven.)	
Bearing between rockers and rocker pins	12,000
Bearing on turned bolts and hand-driven rivets	20,000
Bearing on expansion rollers and rockers, lb per lin in.:	

For diameters up to 25 in. . . . . $\frac{p - 13{,}000}{20{,}000}\,600d$

For diameters from 25 in. to 125 in. . . . . . $\frac{p - 13{,}000}{20{,}000}\,3000\sqrt{d}$

d = diameter of roller or rocker, in inches.
p = yield point in tension of the steel in the roller or in the base, whichever is less.

(b) Cast Steel.

For cast steel, the allowable unit stresses in compression and bearing shall be the same as those for structural steel. Other allowable unit stresses shall be ¾ of those for structural steel.

(c) Masonry.

Bearing Pressure	Lb per Sq In.
Granite	800
Sandstone and limestone	400
Concrete—0.25 of the design ultimate compressive strength. (When the strength of concrete is unknown, use 2500 psi for the design ultimate compressive strength.)	

(d) Timber Cross Ties.

Those specified in the AREA specifications for Structural Timbers, Part 1, Chapter 7.

32. Effective Diameter of Rivets

The nominal diameter of rivets shall be considered the effective diameter.

33. Effective Bearing Area

The effective bearing area of pins, bolts, and rivets shall be the diameter multiplied by the length in bearing; except that for countersunk rivets, ½ the depth of the countersink shall be deducted.

34. Slenderness Ratio

The slenderness ratio (ratio of length to least radius of gyration) shall not exceed:

100 for main compression members.
120 for wind and sway bracing in compression.
140 for single lacing.
200 for double lacing.
200 for tension members other than eyebars.

35. Proportioning Web Members

Web members shall be so proportioned that an increase in the specified live load that will increase the total unit stress in the chords at the center of the span by one-third will produce total unit stresses in the web members not in excess of one and one-third times the specified designing stresses. Members and their connections subject to reversal of stress under such increased live load shall be proportioned in accordance with Sec. A, Art. 28, using the increased designing unit stresses.

Details of Design

36. Thickness of Material

Metal, except for fillers, shall be not less than ⅜ in. thick. Parts subject to marked corrosive influences shall be of greater thickness than otherwise or else protected against such influences.

The thickness of gusset plates connecting the chords and web members of a truss shall be proportionate to the stress to be transferred but not less than ½ in.

37. Accessibility of Parts

Details shall be such that all parts will be accessible for inspection, cleaning, and painting. Closed sections shall be avoided wherever possible.

38. Drainage of Pockets

Pockets or depressions that would hold water shall have effective drain holes or else shall be filled with concrete or other suitable material.

39. Eccentric Connections

Connections of members shall be so arranged, if practicable, that their gravity axes will intersect in a point. If eccentric connections are unavoidable, the members shall be proportioned for the combined stresses.

40. Compression Members

Compression members shall be so designed that the main elements of the section will be connected directly to the gusset plates, pins, or other members.

Built-up sections shall be so arranged that the center of gravity will coincide as nearly as practicable with the center line of the section. Preferably, the segments shall be connected by solid webs.

In members consisting of segments connected by cover plates or lacing, or segments connected by webs, which members receive their full allowable unit stress, the thickness of the webs of the segments shall be not less than $\frac{1}{32}$ of the unsupported distance between the nearest lines of their connecting rivets or the roots of their rolled flanges. The thickness of the cover plates or of the webs connecting the segments shall be not less than $\frac{1}{40}$ of the unsupported distance between the nearest lines of their connecting rivets or the roots of their rolled flanges. For such members in which the stress is less than that allowable, the denominators 32 and 40 may be multiplied by the factor $\sqrt{p/f}$.

p = the allowable unit stress.
f = the unit stress in the member.

41. Outstanding Legs of Angles

The width of the outstanding legs of angles in compression, except those reinforced by plates, shall not exceed the following:

(a) For stringers and girders, where the ties rest on the flange, 10 times the thickness.
(b) For main members carrying axial stress, and for stringers and girders not included in (a), 12 times the thickness.
(c) For bracing and other secondary members, 14 times the thickness.

42. Rigid Members

The bottom chord sections in the first and second panels at each end of single-track pin-connected truss spans, and the hip verticals and members with similar functions in all spans, shall be rigid.

43. Strength of Connections

Connections shall have a strength not less than that of the member connected, based on the allowable unit stress in the member. Connections shall be made as nearly symmetrical as practicable about the axes of the members.

44. Net Section*

The net section of a riveted tension member is the sum of the net sections of its component parts. The net section of a part is the product of the thickness of the part multiplied by its least net width.

* A history and elucidation of this method of obtaining net sections was published in the *Proceedings*, Vol. 36, 1935, p. 775.

The net width for any chain of holes extending progressively across 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 = pitch of any two successive holes in the chain.
g = gage of the same holes.

The net section of the part is obtained from that chain which gives the least net width.

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.

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.

The diameter of the hole shall be taken as $\frac{1}{8}$ in. greater than the nominal diameter of the rivet.

45. Effective Sections of Angles

If angles in tension are so connected that bending cannot occur in any direction, the effective section shall be the net section of the angle. If such angles are connected on one side of a gusset plate, the effective section shall be the net section of the connected leg plus $\frac{1}{2}$ the section of the unconnected leg.

46. Grip of Rivets

If the grip of rivets carrying calculated stress exceeds $4\frac{1}{2}$ times the diameter, the number of rivets shall be increased at least 1 per cent for each additional $\frac{1}{16}$ in. of grip. If the grip exceeds 6 times the diameter, the shanks shall be specially designed to fill the holes completely when driven.

47. Pitch and Gage of Rivets

(a) The pitch of rivets is the distance, in inches, between centers of adjacent rivets, measured along one or more rivet lines. The gage of rivets is the distance, in inches, between adjacent rivet lines or the distance from the back of an angle or other shape to the first rivet line.

(b) For sealing, the pitch on a single line adjacent to a free edge of an outside plate or shape shall not exceed

$$4 + 4t, \text{ nor } 7 \text{ in.}$$

If there be a second line of rivets uniformly staggered with those in the line adjacent to the free edge, at a gage, g, less than $1\frac{1}{2} + 4t$ therefrom, the staggered pitch of the rivets in such two lines shall not exceed

$$4 + 4t - \frac{3}{4}g, \text{ nor } 7 - \frac{3}{4}g$$

but need not be less than one-half the requirement for a single line.

t = thickness, in inches, of the thinner outside plate or shape.

(c) Where two or more plates or shapes are in contact, there shall be stitch rivets to make them act in unison, and in compression members, to prevent buckling.

In compression members, the pitch of stitch rivets on any single line in the direction of stress shall not exceed $12t$; except that if the rivets on adjacent lines be staggered and the gage, g, between the line under consideration and the farther adjacent line is less than $24t$, the staggered pitch in such two lines shall not exceed

$$12t, \text{ nor } 15t - \frac{3}{8}g$$

The gage between adjacent lines of stitch rivets shall not exceed $24t$.

t = thickness, in inches, of the thinner outside plate or shape.

(d) At the ends of compression members, the pitch on any line in the direction of stress shall not exceed 4 times the diameter of the rivets for a distance equal to $1\frac{1}{2}$ times the width of the member.

In tension members, the pitch shall not exceed twice that specified for compression members, and the gage shall not exceed that specified for compression members.

48. Minimum Spacing of Rivets

The distance between centers of rivets shall be not less than 3 times the diameter of the rivets.

49. Edge Distance of Rivets

The distance from the center of a rivet to a sheared edge shall not be less than $1\frac{3}{4}$ times the diameter, nor to a rolled or planed edge less than $1\frac{1}{2}$ times the diameter, except in flanges of beams and channels, where the minimum distance may be $1\frac{1}{4}$ times the diameter.

The distance from the free edge of an outside plate or shape to the first line of rivets shall not exceed

$$1\tfrac{1}{2} + 4t, \text{ nor } 6 \text{ in.}$$

t = thickness, in inches, of the plate or shape.

50. Sizes of Rivets in Angles

In angles, the size of which is determined by calculated stress, the diameter of the rivets shall not exceed $\frac{1}{4}$ of the width of the leg in which they are driven. In angles, the size of which is not so determined, 1-in. rivets may be used in $3\frac{1}{2}$-in. legs, $\frac{7}{8}$-in. rivets in 3-in. legs, and $\frac{3}{4}$-in. rivets in $2\frac{1}{2}$-in. legs.

51. Compression Splices

Members subject to compression only, if faced for bearing, shall be spliced on 4 sides sufficiently to hold the abutting parts true to place. The splice shall be as near a panel point as practicable and shall be designed to transmit at least ½ of the stress through the splice material. Members not faced for bearing shall be fully spliced.

52. Extra Rivets

If splice plates are not in direct contact with the parts which they connect, there shall be rivets on each side of the joint in excess of the number required in the case of direct contact, to the extent of two extra lines for each intervening plate.

If rivets carrying stress pass through fillers, the fillers shall be extended beyond the connected member and the extension secured by enough rivets to distribute the total stress in the member uniformly over the combined sections of the member and the fillers, except that if a filler is less than ¼ in. thick it shall not be extended beyond the splicing material, and additional rivets are not required.

53. Stay Plates

On the open sides of compression members, the segments shall be connected by lacing bars, and there shall be stay plates as near each end as practicable. There shall be stay plates at intermediate points where the lacing is interrupted. In main members, the length of the end stay plates shall be not less than 1¼ times the distance between the lines of rivets connecting them to the outer flanges. The length of intermediate stay plates shall be not less than ¾ of that distance.

The segments of tension members composed of shapes shall be stayed together. The length of the stay plates shall be not less than ⅔ of the lengths specified for stay plates on compression members. They shall be connected to each segment by at least three rivets.

The thickness of stay plates shall be not less than 1/50 of the distance between the lines of rivets connecting them to the outer flanges for main members, or 1/60 of that distance for bracing members.

54. Lacing

Lacing bars of compression members shall be so spaced that the slenderness ratio of the portion of the flange included between the lacing bar connections will be not more than 40 nor more than ⅔ of the slenderness ratio of the member.

In compression members, the shearing force normal to the member in the plane of the lacing shall be that obtained by the following formula:

$$V = \frac{P}{100}\left(\frac{100}{\frac{l}{r} + 10} + \frac{\frac{l}{r}}{100}\right)$$

where V = normal shearing force.
P = allowable compressive axial load on member.

l = length of member, in inches.
r = radius of gyration of section about the axis perpendicular to plane of lacing, in inches.

To the shear so determined shall be added any shear due to the weight of the member or to other forces, and the lacing proportioned for the combined shear.

The shear shall be considered as divided equally among all parallel planes in which there are shear resisting elements, whether continuous plates or lacing. The section of the lacing bars shall be determined by the formula for axial compression in which l is taken as the distance along the bar between its connections to the main segments for single lacing, and as 70 per cent of that distance for double lacing.

If the distance across the member between rivet lines in the flanges is more than 15 in. and a bar with a single rivet in the connection is used, the lacing shall be double and riveted at the intersections.

The angle between the lacing bars and the axis of the member shall be approximately 45 deg for double lacing and 60 deg for single lacing.

Lacing bars may be shapes or flat bars. For main members the minimum thickness of flat bars shall be $\frac{1}{40}$ of the distance along the bar between its connections for single lacing, and $\frac{1}{60}$ for double lacing. For bracing members, the limits shall be $\frac{1}{50}$ for single lacing and $\frac{1}{75}$ for double lacing.

The diameter of the rivets in lacing bars shall not exceed $\frac{1}{3}$ of the width of the bar. There shall be at least 2 rivets in each end of lacing bars riveted to flanges more than 5 in. in width.

55. Section at Pin Holes

In pin-connected riveted tension members, the net section beyond the pin hole, parallel with the axis of the member, shall be not less than the required net section of the member. The net section through the pin hole, transverse to the axis of the member, shall be at least 40 per cent greater than the required net section of the member. The ratio of the net width (through the pin hole transverse to the axis of the member) to the thickness of the segment shall not be more than eight.

56. Reinforcing Plates at Pin Holes

Where necessary for the required section or bearing area, the section at pin holes shall be increased on each segment by plates so arranged as to reduce the eccentricity of the segment to a minimum. One plate on each side shall be as wide as the outstanding flanges will allow. At least one full-width plate on each segment shall extend to the far edge of the stay plate, and the others, not less than 6 in. beyond the near edge. These plates shall be connected by enough rivets to transmit the bearing pressure and so arranged as to distribute it uniformly over the full section.

57. Forked Ends of Compression Members

Forked ends of compression members will be permitted only where unavoidable. There shall be enough pin plates on forked ends to make the section of each jaw equal to that of the member. The pin plates shall be long enough to develop the pin plate beyond the near edge of the stay plate, but not less than the length required by Sec. A, Art. 56.

58. Floorbeams

Floorbeams preferably shall be square to the girders or trusses.

59. End Connection Angles

The connection angles for stringers, floorbeams, and beams in solid floor sections shall be not less than 4 in. in width and ½ in. in finished thickness.

For stringers, the gage of the outstanding legs of the connection angles over the top ⅓ of the stringer depth shall be not less than the quantity $\sqrt{\frac{lt}{8}}$.

l = length of stringer span, in inches.
t = thickness of angle, in inches.

60. Proportioning Girders and Beams

Plate girders, I-beams, and other members subject to bending that produces tension on one face, shall be proportioned by the moment-of-inertia method. The neutral axis shall be taken along the center of gravity of the gross section. The tensile stress shall be computed from the moment of inertia of the entire net section and the compressive stress from the moment of inertia of the entire gross section.

61. Flange Section

In order to offset the effects of corrosion and the possible crookedness of the compression flange of a plate girder or a rolled beam, the gross section of the compression flange shall not be less than the gross section of the tension flange.

Flanges of plate girders preferably shall be made without cover plates or side plates unless angles of greater section than 6 in. by 6 in. by ⅞ in. would otherwise be required.

Cover plates shall be equal in thickness, or shall diminish outwardly in thickness. No plate shall be thicker than the flange angles of a girder or the nominal thickness of the flange of a rolled beam. When cover plates are used, at least one plate of each flange shall extend the full length of the girder or beam. Other flange plates shall extend far enough to develop the capacity of the plate beyond the theoretical end. The gross area of cover plates in each flange shall not exceed 70 per cent of the total flange, consisting of cover plates, flange angles directly connected to cover plates, and side plates, if any, for a plate girder; and cover plates and the beam flange for a rolled beam.

In through bridges, there shall be end and corner cover plates.

62. Flange Rivets

The flanges of plate girders shall be connected to the web with enough rivets to transmit to the flange section the horizontal shear at any point together with any load that is applied directly on the flange. Where the ties rest on the flange, one wheel load, including 80 per cent impact, shall be assumed to be distributed over 3 ft.

On ballasted deck girders, the wheel load, including 80 per cent impact, shall be assumed to be distributed over 5 ft.

63. Flange Splices

Flange members that are spliced shall be covered by extra material equal in section to the member splice. There shall be enough rivets on each side of the splice to transmit to the splice material the stress value of the part cut.

Flange angles shall be spliced with angles. No two members shall be spliced at the same cross section.

64. Web Splices

Splices in the webs of plate girders shall be designed for the full strength of the web in both shear and bending.

65. Thickness of Web Plates

The thickness of the webs of plate girders shall be not less than $\frac{1}{170}$ of the clear distance between the flanges, except that if the extreme fiber stress in the compression flange is less than that allowable, the denominator 170 may be multiplied by the factor $\sqrt{p/f}$.

p = the allowable extreme fiber stress.
f = the extreme fiber stress in the compression flange.

66. Stiffeners at Points of Bearing

Stiffeners shall be placed at end bearings of plate girders and beams, and at points of bearing of concentrated loads. They shall extend as nearly as practicable to the edges of the flange to give effective distribution and shall be connected to the web by enough rivets to transmit the load. Such stiffeners shall not be crimped. Only that part of the stiffener cross section that lies outside the fillet of the flange shall beconsidered effective in bearing.

67. Intermediate Stiffeners

If the depth of the web between the flanges or side plates of a plate girder exceeds 60 times its thickness, it shall be stiffened by pairs of angles riveted to the web. The clear distance between stiffeners shall not exceed 72 in. nor that given by the formula:

$$d = \frac{10{,}500t}{\sqrt{S}}$$

d = clear distance between stiffeners, in inches.
t = thickness of web, in inches.
S = unit shearing stress, gross section, in web at point considered.

The width of the outstanding leg of each angle shall be not more than 16 times its thickness and not less than 2 in. plus $\frac{1}{30}$ of the depth of the girder.

Stiffeners on one side of a plate girder may be used, provided they have the same stiffness as the minimum acceptable pairs of angles. They shall be riveted to the outstanding leg of the compression flange.

68. Bracing of Top Flanges of Through Girders

The top flanges of through plate girders shall be braced at the panel points by brackets with web plates. The brackets shall extend to the top flange of the main girder and be as wide as the clearance will allow. They shall be attached securely to a stiffener angle on the girder and to the top flange of the floor beam. On solid floor bridges the brackets shall be not more than 12 ft apart.

69. Lateral Bracing

There shall be bottom lateral bracing in all spans except deck spans less than 50 ft long. There shall be top lateral bracing in all deck spans and in through spans that have enough head room.

If the construction of the floor is such as to afford the specified lateral resistance, the floor shall be taken as the lateral bracing required in its plane.

70. Portal and Sway Bracing

In through truss spans there shall be portal bracing, with knee braces, as deep as the clearance will allow. There shall be sway bracing at the intermediate panel points if the trusses are high enough to allow a depth of 6 ft or more for such bracing. If they are not high enough to allow that depth, the top lateral struts shall be of the same depth as the chord, and there shall be knee braces as deep as the clearance will allow.

In deck truss spans there shall be sway bracing at the panel points. The top lateral forces shall be carried to the supports by means of a complete system of bracing in the planes of the top chords and the main end posts.

71. Rigid Bracing

Lateral bracing shall be rigid and there shall be not less than 3 rivets in each end connection.

If the bracing is a double system and the members meet the requirements for both tension and compression members, both systems may be considered effective simultaneously.

72. Cross Frames and Diaphragms

In deck plate girder spans there shall be cross frames at the ends and at intervals not exceeding 18 ft. The end frames shall be proportioned for the centrifugal and lateral forces.

If there are two lines of stringers under each track in panels more than 20 ft in length, they shall be connected by cross frames.

Where the track is carried on transverse beams, without stringers, the beams shall be connected by diaphragms, there being at least one line for each track.

Longitudinal beams shall have diaphragms at the ends and at intermediate points. To obtain lateral distribution of the load on spans with ballast floors, the intermediate diaphragms shall be spaced not more than 8 ft apart. The diaphragms shall be as deep as the depth of the beams will permit and be rigidly connected, by means of double angles, to the beam web.

If I-beams are used for diaphragms, they shall be rigidly connected by means of one or more lines of rivets on each side of the diaphragm web.

73. Viaducts

Viaducts shall consist of spans supported on bents. The bents preferably shall be composed of not more than two columns braced together. They shall be united in pairs by bracing to form towers except where single bents are necessary.

The columns preferably shall be made with a transverse batter of 1 horizontal to 6 vertical for single track viaducts, and 1 horizontal to 8 vertical for double track viaducts.

The bracing of bents and towers shall consist of double systems of rigid diagonals with struts at caps and bases and at intermediate panel points. In double track towers there shall be bracing to transmit the longitudinal force to both sides.

The bottom struts shall be proportioned for either the calculated stresses or a stress in tension or compression equal to $\frac{1}{4}$ of the dead load reaction on 1 pedestal, whichever is greater. The column bearings shall be designed to allow for the expansion and contraction of the tower bracing.

Where long spans are supported on short single bents, the bents shall be made with hinged ends, or else the columns and anchorage shall be proportioned for the bending stresses produced by temperature changes and the elastic deformation of the span.

74. Eyebars

The thickness of eyebars shall be not less than 1 in. nor more than 2 in. The section of the head through the center of the pin-hole shall exceed that of the body of the bar by at least 35 per cent. The form of the head shall be submitted to the engineer for approval before the bars are made. The diameter of the pin shall be not less than $\frac{8}{10}$ of the width of the widest bar attached.

75. Eyebar Packing

The eyebars of a set shall be symmetrical about the central plane of the truss and as nearly parallel as practicable. The inclination of any bar to the plane of the truss shall not exceed $\frac{1}{16}$ in. to the foot. The bars shall be packed close, held against lateral movement, and so arranged that those in the same panel will not be in contact.

76. Expansion

The design shall be such as to allow for the changes in length of the span, resulting from changes in temperature, at the rate of 1 in. in 100 ft. Provision shall be made for changes in length of the span resulting from live load stresses. In spans more than 300 ft long, allowance shall be made for the expansion in the floor.

77. End Bearings

In spans more than 70 ft long, there shall be hinged bearings at both ends and rollers or rockers at the expansion end. Shorter spans shall be designed to slide on bearings with smooth surfaces.

Bearings and ends of spans shall be secured against lateral and vertical movement.

End bearings on masonry preferably shall be raised above the bridge seat by metal pedestals or bolsters.

78. Shoes and Pedestals

Shoes and pedestals shall be designed on the assumption that the load is distributed uniformly over the entire bearing surface. They shall be made preferably of cast steel, with no part less than 1 in. in thickness. Built-up shoes and pedestals, if used, shall have material not less than ¾ in. thick in load-carrying portions, and may have plates and shapes welded together, provided the welding serves primarily to make the component parts function as a unit, with the vertical load being carried directly in contact bearing. Diaphragms shall be provided between web surfaces.

The difference in width or length between top and bottom bearing surfaces shall not exceed twice the vertical distance between them. For hinged bearings, the vertical distance shall be measured from the center line of pin.

79. Base and Masonry Plates

Base and masonry plates shall be designed on the assumption that the load is distributed uniformly over the entire bearing surface, except where modified by eccentricity caused by rocker travel.

The distance from bearing line of rocker or bearing line of end roller to edge of base plate shall not be greater than 2 times the thickness of the plate. The base plate shall not project more than its thickness beyond the end of rocker or roller.

For spans designed to slide on bearings with smooth surfaces, the distance from center line of bearing to edge of masonry plate, measured parallel with the track, shall not be more than 2 times the thickness of the plate plus 4 in.

80. Rockers or Rollers

Rockers shall be used in preference to rollers where conditions permit. The upper surface of rockers shall have a pin or cylindrical bearing. The lower surface shall be cylindrical, with its center of rotation preferably at the center line of rotation of the upper bearing surface. If the centers of rotation of the upper and lower bearing surfaces do not coincide, the center of rotation of the upper bearing surface shall lie below that of the lower bearing surface and the resulting longitudinal force shall be provided for. The length of rocker shall not be greater than the length of upper bearing surface plus the height of rocker. Each end face of the rocker shall make a right angle with its upper and lower bearing surfaces, and these vertical portions of the end face shall not be less than 1½ in. deep. No part of an end face of the rocker shall lie inside a straight line, which has a slope of not more than 1 horizontal to 1½ vertical and connects the bottoms of the lower and upper vertical faces. The rocker shall be geared to the base plate.

Rollers may be either cylindrical or segmental and shall not be less than 6 in. in diameter. They shall be secured to insure parallelism and geared to the upper and lower plates. The roller nest shall be so designed that the parts may be readily cleaned.

81. Inclined Bearings

For spans on an inclined grade and without hinged bearings, the masonry plates or the sole plates shall be beveled so that the masonry surfaces may be made level.

82. Anchor Bolts

Anchor bolts shall be not less than $1\frac{1}{4}$ in. in diameter. There shall be washers under the nuts. Anchor bolt holes in pedestals and sole plates shall be $\frac{3}{8}$ in. larger in diameter than the bolts, and at expansion points the holes in the sole plates shall be slotted.

Anchor bolts that do not take uplift shall be long enough to extend 12 in. into the masonry. Those that do take uplift shall be designed to engage a mass of masonry the weight of which is $1\frac{1}{2}$ times the uplift.

83. Camber

The camber of trusses shall be equal to the deflection produced by the dead load plus a load of 3000 lb per ft of track. The camber of plate girders more than 90 ft. in length shall be equal to the deflection produced by the dead load only. Plate girders 90 ft or less in length and rolled beams need not be cambered.

84. Screw Threads

Screw threads shall be American Standard, free fit, except that for pin ends of diameters greater than $1\frac{3}{8}$ in., the number of threads shall be 6 to an inch.

85. Pins

In pins more than 9 in. in diameter, there shall be a hole not less than 2 in. in diameter bored longitudinally on the center line.

The turned bodies of pins shall be long enough to extend at the ends $\frac{1}{4}$ in. beyond the outside faces of the parts connected. The pins shall be secured by chambered nuts or by solid nuts and washers. If the pins are bored, through rods with cap washers may be used. The screw ends shall be long enough to allow burring the threads.

Pin-connected members shall be secured in such a way as to limit lateral movement on the pin. Filler rings shall be made of metal not less than $\frac{1}{2}$ in. thick.

86. Name Plates

Attached to the end of each span, at a point convenient for inspection, there shall be a name plate showing in raised letters and figures the name of the fabricator and the year of construction.

C. ALLOY STEELS

Foreword

In issuing specifications for using alloy steels in railway bridges, it is desirable to present certain information, not appropriate as specification clauses but worthy of consideration when the use of an alloy steel is contemplated.

1. Nickel and silicon steels both have been used in important railway and highway bridges and have proved satisfactory. Other alloy steels have been used successfully.

2. Silicon steel can be obtained from practically all structural and plate mills, even in moderate tonnages if not ordered in too great a variety of sections. Nickel steel can be had by special arrangements with the mill and in larger tonnages; other alloys, only by negotiation with the mills.

3. It is customary to use carbon steel rivets in fabricating alloy steels, the rivet values being based on the carbon rivet. In rare instances alloy steel rivets have been used with alloy steels. High-strength structural rivet steel, covered by ASTM Specifications, designation 195, should be used with caution in members subject to wide variation or reversal of stress during passage of a train, until checked by the Research Council on Riveted and Bolted Structural Joints.

4. The modulus of elasticity is the same for the alloy steels as for carbon steel.

5. Short spans can be constructed most economically of carbon steel.

6. As span lengths increase and the influence of dead load becomes preponderant, a span of equal cost as between carbon steel and part alloy steel will be reached, but of two spans of equal cost in place, the heavier one (carbon steel) is the better purchase for reasons of rigidity and thickness of material (resistance to corrosion).

7. With increasing length of span, the economy in use of alloy steel will become pronounced and the features of rigidity and resistance to corrosion will be amply protected in an alloy steel design. For such spans, and unless there is extreme need for reducing the dead load, the floor systems should be of carbon steel.

8. Nickel and silicon steels are excellent bridge steels, but it must be remembered that in every high-strength steel, whether high-carbon or alloy, some desirable property must be encroached upon to secure the added strength. While such steels have ample plastic action between yield point and ultimate strength, it is not as great as for carbon steel. While they have excellent ductility and bending properties, those properties are not quite so good as in carbon steel. Therefore, when such steels are used, some effort should be made to compensate in the design and details for these differences. For that reason, the allowable unit stresses in these specifications are a somewhat smaller fraction of the specified yield point than those for carbon steel and slightly greater edge distances are desirable.

9. On account of the possibility of fatigue failure, the allowable unit stresses in Sec. C, Art. 1, shall not apply to members that will be subjected to a large number of stress cycles during the life of the bridge and having either reversal of stress or low dead load stress. For such members the allowable unit stresses shall be reduced, and in floor systems shall not exceed those specified in Sec. A, Art. 31.

10. In considering other alloy steels as they are made available, the tests should be made to develop the effect of rivet holes and riveting, before determining the al-

lowable unit stresses. No steel should be considered satisfactory for bridge construction if its yield point exceeds 70 per cent of the ultimate strength.

General

The term "alloy steel" means structural silicon steel or structural nickel steel. The following specifications shall apply to alloy steel parts of bridges except as provided otherwise herein.

Unit Stresses

1. Unit Stresses

Except as limited in Sec. C, Par. 9, of the Foreword, the allowable unit stresses to be used in proportioning the parts of a bridge shall be as follows:

	Lb per Sq In. Silicon Steel	*Lb per Sq In.* Nickel Steel
(a) Structural Steel.		
Axial tension, structural steel, net section...	24,000	30,000
Axial tension, eyebars....................		27,000
Tension in extreme fibers of rolled shapes, girders, and built sections, subject to bending..................................	24,000	30,000
Axial compression, gross section:		
For stiffeners of plate girders............	24,000	30,000
For compression members centrally loaded and with values of l/r not greater than 130 for silicon steel or 120 for nickel steel:		
Riveted ends......................	$20{,}000 - 0.46\frac{l^2}{r^2}$	$24{,}000 - 0.66\frac{l^2}{r^2}$
Pin ends..........................	$20{,}000 - 0.61\frac{l^2}{r^2}$	$24{,}000 - 0.86\frac{l^2}{r^2}$

l = length of member, in inches.
r = least radius of gyration of member, in inches.

For compression members of known eccentricity, see Appendix A.

	Lb per Sq In. Silicon Steel	*Lb per Sq In.* Nickel Steel
Compression in extreme fibers of rolled shapes, girders, and built sections, subject to bending, gross section (for values of l/b not greater than 40)......................	$24{,}000 - 6.67\frac{l^2}{b^2}$	$30{,}000 - 8\frac{1}{3}\frac{l^2}{b^2}$

l = length, in inches, of unsupported flange between lateral connections or knee braces.
b = flange width, in inches.

Diagonal tension in webs of girders and rolled beams at sections where maximum shear and bending occur simultaneously........	24,000	27,000
Stress in extreme fibers of pins.............	36,000	44,000
Shear in plate girder webs, gross section......	14,000	17,500
Shear in pins............................	18,000	22,000
Bearing on pins..........................	32,000	40,000
Bearing on milled stiffeners and other steel parts in contact........................	36,000	44,000
Bearing between rockers and rocker pins....	16,000	18,000

Bearing on expansion rollers and rockers:

	Lb per Lin In.
For diameters up to 25 in.............	$\dfrac{p - 13{,}000}{20{,}000} 600d$
For diameters from 25 in. to 125 in.....	$\dfrac{p - 13{,}000}{20{,}000} 3000 \sqrt{d}$

d = diameter of roller or rocker, in inches.
p = yield point in tension of the steel in the roller or the base, whichever is least.

2. Thickness of Material

Metal, except for fillers, shall be not less than $\frac{3}{8}$ in. thick. Silicon steel plates shall be not more than $1\frac{1}{8}$ in. thick and nickel steel plates not more than 1 in.

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 $\frac{1}{25}$ of the unsupported distance between the nearest rivet lines. The thickness of the cover plates or webs connecting the segments shall be not less than $\frac{1}{32}$ of the unsupported distance between the nearest lines of their connecting rivets or the roots of their rolled flanges.

The thickness of the webs of plate girders shall be not less than $\frac{1}{45}$ for silicon steel and $\frac{1}{30}$ for nickel steel, of the clear distance between flange angles or side plates.

If the depth of the web between flanges or side plates of a plate girder exceeds 50 times its thickness, it shall be stiffened by pairs of angles riveted to the web in accordance with the requirements of Sec. A, Art. 67.

3. Sizes of Rivets

The diameter of rivets in main members preferably shall not be less than 1 in.

4. Lacing

In compression members, the shearing force normal to the member in the plane of the lacing shall be that obtained by the following formula:

$$V = \frac{P}{100}\left(\frac{100}{\frac{l}{r} + 10} + \frac{\frac{l}{r}}{60}\right)$$

where V = normal shearing force.
P = allowable compressive axial load on member.
l = length of member, in inches.
r = radius of gyration of section about the axis perpendicular to plane of lacing, in inches.

To the shear so determined shall be added any shear due to the weight of the member or other forces, and the lacing proportioned for the combined shear.

5. Rivets

Silicon steel and nickel steel shall not be used for rivets. Rivets preferably shall be made of the rivet steel specified in Sec. B, Art. 1. Other steels may be used for rivets and bolts if specified by the engineer.

6. Alloy and Structural Steels Combined

In case the deformation of alloy steel elements of members or spans induces stresses in other elements exceeding the allowable stresses for structural steel, such other elements also shall be made of alloy steel. Otherwise, bracing and secondary truss members generally shall be made of structural steel. Lacing, stay plates, diaphragms, stiffening angles, and other detail material except gussets and splice plates shall be made of structural steel. For alloy steel elements, the workmanship shall be that specified for alloy steel.

7. Secondary Stresses

The design and details shall be such that secondary stresses will be as small as practicable. Secondary stresses due to truss distortion or floorbeam deflection usually need not be considered in any member, the width of which, measured parallel to the plane of distortion, is less than $\frac{1}{10}$ of its length. If the secondary stress exceeds 5600 psi for tension members and 4200 psi for compression members, the excess shall be treated as a primary stress.

APPENDIX A

Formulas For Compression Members

The permissible unit stresses in axial compression for centrally loaded members given by the parabolic formulas in Sec. A, Art. 31, and Sec. B, Art. 8, agree so closely with those obtained by the secant formula that they may be used without substantial error for slenderness ratios within the limits shown.

The corresponding secant formulas, which should be used when the slenderness ratio exceeds those limits, are:

For riveted ends. $$p = \frac{\frac{y}{f}}{1 + 0.25 \operatorname{Sec} \frac{0.75\,l}{2r}\sqrt{\frac{fp}{E}}} \qquad (1)$$

For pin ends.
$$p = \frac{\dfrac{y}{f}}{1 + 0.25 \sec \dfrac{0.875\, l}{2r}\sqrt{\dfrac{fp}{E}}} \tag{2}$$

For compression members with known eccentricity of loading, for all slenderness ratios, the formulas are:

For riveted ends.
$$p = \frac{\dfrac{y}{f}}{1 + \left(\dfrac{e_1c_1}{r_1^2} + 0.25\right)\sec \dfrac{0.75l}{2r_1}\sqrt{\dfrac{fp}{E}} + \dfrac{e_2c_2}{r_2^2}\sec \dfrac{0.75l}{2r_2}\sqrt{\dfrac{fp}{E}}} \tag{3}$$

For pin ends.
$$p = \frac{\dfrac{y}{f}}{1 + \left(\dfrac{e_1c_1}{r_1^2} + 0.25\right)\sec \dfrac{0.875l}{2r_1}\sqrt{\dfrac{fp}{E}} + \dfrac{e_2c_2}{r_2^2}\sec \dfrac{0.875l}{2r_2}\sqrt{\dfrac{fp}{E}}} \tag{4}$$

p = allowable average compressive unit stress.
e_1 = known eccentricity of the applied load in the direction of the smaller value of r (r_1).
e_2 = known eccentricity of the applied load in the direction of the greater value of r (r_2).
c_1 = distance from neutral axis to extreme fiber in the direction of r_1.
c_2 = distance from neutral axis to extreme fiber in the direction of r_2.
l = length of member.
$r = r_1$ = least radius of gyration of the member.
r_2 = radius of gyration of the member in a direction at right angles with that of r_1.
Values of e, c, l, and r are in inches.
E = modulus of elasticity = 29,400,000.
y = yield point in tension:

33,000 for structural steel
45,000 for silicon steel
55,000 for nickel steel

f = factor of safety based on yield point.

The coefficient 0.25 in the denominators of the formulas provides for inherent crookedness and unknown eccentricity.

The factors of safety to be used in the formulas are computed as follows:

From the parabolic formulas in Sec. A, Art. 31, and Sec. C, Art. 8, for $\dfrac{l}{r} = 0$

p = 15,000 for structural steel
= 20,000 for silicon steel
= 24,000 for nickel steel

From formulas (1) and (2), for $\dfrac{l}{r} = 0$

$$p = \frac{y}{f} \div 1.25, \text{ from which } f = \frac{y}{1.25p}$$

Using the foregoing values of y and p, the following values of f are derived:

$$\begin{aligned} f &= 1.76 \text{ for structural steel} \\ &= 1.80 \text{ for silicon steel} \\ &= 1.83\tfrac{1}{3} \text{ for nickel steel} \end{aligned}$$

APPENDIX C

AMERICAN WELDING SOCIETY

STANDARD CODE FOR ARC AND GAS WELDING IN BUILDING CONSTRUCTION, 1946

Sections 1 and 2 and Appendix C

STANDARD SPECIFICATIONS FOR WELDED HIGHWAY AND RAILWAY BRIDGES, 1947

Sections 1 and 2 and Appendix A

STANDARD CODE FOR ARC AND GAS WELDING IN BUILDING CONSTRUCTION

SECTION 1—GENERAL PROVISIONS

101. Application of Welding

(*a*) Arc and gas welding may be employed, either alone or in combination with riveting,* bolting, or other connecting means permitted under the Building code,† for connecting to one another, or assembling the component parts of, steel beams, girders, lintels, trusses, columns, and other structural steel members of buildings, or for connecting steel to wrought-iron members of existing buildings, provided that such work be designed and executed in accordance with the provisions of this Code.

(*b*) The provisions of Section 2, Part II—Permissible Unit Stresses, and Part III—Details of Welded Joints, shall not apply to structures or parts of structures which will be subject to reversal of load or substantially complete release of load (other than wind load) many thousands of times.‡

102. Definitions

Welding terms used in this Code shall be interpreted in accordance with the Standard Definitions of the American Welding Society.

103. Plans, Drawings, and Loading and Stress Data

(*a*) Framing plans, showing the size and location of all principal members, shall be submitted for the approval of the Building Commissioner.§

(*b*) All framing shall comply with the requirements of the locally applicable building law or specifications or other construction regulations.

(*c*) Framing plans shall indicate the type or types of construction (as defined in Article 104) to be employed and, in the case of rigid frames, shall be supplemented

* See Article 202.

† Building Code, wherever the expression occurs in this Code, refers to building law or specifications or other construction regulations in conjunction with which this Code is applied. In the absence of any locally applicable building law or specifications or other construction regulations, it is recommended that the construction be required to comply with the Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings, Riveted, Bolted and Arc-Welded Construction, of the Amercian Institute of Steel Construction.

‡ It is recommended that such structures or parts of structures be designed in accordance with the American Welding Society's "Specifications for Welded Highway and Railway Bridges," wherein provision is made for fatigue effects not contemplated in this Code.

§ The term "Building Commissioner" is herein used in an inclusive sense, referring to the official or bureau, by whatever term locally designated, who is delegated to enforce the local building law or specifications or other construction regulations.

by necessary data as to the assumed loads and the shears, moments, and axial forces to be resisted by all members and their connecting welds.

(*d*) Detail drawings, giving complete information necessary for the fabrication and field assembly of the component parts, shall be prepared in advance of actual fabrication, but will not be required as a basis for the issuance of a building permit. Prints of the detail drawings or of designated portions thereof shall be submitted to the Building Commissioner upon his request, prior to their use in fabrication.

104. Types of Construction

(*a*) Three basic types of design and design assumption are permissible, under the respective conditions stated hereinafter; and each will govern in a specific manner the sizes of members and the types and strength of their connections.

Type 1, commonly designated as "rigid-frame" (continuous, restrained), assumes that the end connections of beams, girders, and all other members in the frame have sufficient rigidity to hold virtually unchanged the original angles between such members and the members to which they connect.

Type 2, commonly designated as "conventional" or "simple" framing (unrestrained, free-ended), assumes that the ends of beams and girders are connected for shear only and are free to rotate under load.

Type 3, commonly designated as "semi-rigid framing" (partially restrained), assumes that the connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the complete rigidity of Type 1 and the complete flexibility of Type 2.

(*b*) All welded connections shall be consistent in their design with the assumptions as to type of construction, as specified on the plans.

(*c*) Type 1 construction is unconditionally permitted under this Code. It is a necessary condition of this type that the calculated stresses and resulting strains in all members and their connections occur within the elastic range and that the stresses do not exceed those allowed in Article 204 of this Code and other applicable parts of the Building Code.

(*d*) Type 2 construction is permitted under this Code, subject to the stipulations of Paragraph (*e*) hereof wherever applicable. Beam-to-column connections with seats for the reactions and with top clip angles for lateral support only are classed under Type 2.

(*e*) In tier buildings, designed in general as Type 2 construction, in that the beam-to-column connections other than wind connections are flexible, the distribution of the wind moments, as between the several joints of the frame, may be made by recognized empirical methods provided that either:

(1) The wind connections designed to resist the assumed wind moments are adequate to resist the moments induced by the gravity loading and the wind loading, at the increased unit stresses permitted therefor, or,

(2) The wind connections designed to resist the assumed wind moments are so designed that larger moments, induced by the gravity loading under the actual condition of restraint, will be relieved by deformation of the connection material without serious overstress in the welds.

(*f*) Type 3 (semi-rigid) construction will be permitted only upon the filing with the Building Commissioner of satisfactory evidence that the connections to be used are capable of resisting definite moments without overstress of the welds. The proportioning of main members joined by such connections shall be predicated upon no greater degree of end restraint than the minimum known to be effected by the respective connections.

Types 2 and 3 construction will ordinarily necessitate some non-elastic but self-limiting deformation of a structural steel part, but under forces which do not overstress the welds.

105. Welding Symbols for Construction Drawings

Welding symbols used on framing plans and detail drawings shall be the American Welding Society Standard Symbols; or other adequate welding symbols may be used, provided a complete explanation thereof is shown on the plans or drawings.

106. Base Material

Structural steel shall conform to the Standard Specifications of the American Society for Testing Materials for Structural Steel for Bridges and Buildings, Serial Designation A-7, as amended to date.

107. Safety Precautions

(*a*) Operators of welding and cutting equipment shall be protected from the rays of the arc or flame by gloves and by helmets, hand shields, or goggles equipped with suitable filter lenses. Cover-glasses in helmets, shields, and goggles shall be replaced when they become sufficiently marred to impair the operator's vision.

(*b*) Closed spaces shall be ventilated properly while welding or cutting is being done therein.

(*c*) Care shall be taken to avoid risk of fire or explosion when welding or cutting near flammable or explosive materials.

(*d*) Suitable protection against the rays of the arc shall be maintained by the contractor where arc-welding operations might be viewed within harmful range by persons other than the welding operators and inspectors.

SECTION 2—DESIGN OF WELDED CONNECTIONS

Part I—General Requirements

201. Plans and Drawings

(*a*) Full and complete information regarding location, type, size, and extent of all welds shall be clearly shown on the plans or detail drawings. The plans or detail drawings shall clearly distinguish between shop and field welds.

(*b*) Note shall be made on the detail drawings of those joints or groups of joints in which it is especially important that the welding sequence and technique of welding be carefully controlled to minimize shrinkage stresses and distortion.

(*c*) Weld lengths called for on the drawings shall be the effective lengths.

202. Rivets and Bolts

(*a*) In new work, rivets or bolts in combination with welds shall not be considered as sharing the stress, and welds shall be provided to carry the entire stress for which the connection is designed.

(*b*) In making alterations to structures, existing rivets may be utilized for carrying stresses resulting from existing dead loads, and welding shall be provided to carry all additional stress.

PART II—PERMISSIBLE UNIT STRESSES

203. General

Unit stresses specified herein are for welded joints in which residual stress due to welding and restraint of parts during cooling have been properly controlled and kept as low as possible, by using a planned sequence and procedure of welding as prescribed in Article 403.

204. Unit Stresses in Welds

(*a*) Welded joints constructed with the base material and filler metal prescribed in this Code shall be proportioned so that the stresses caused therein by the loads specified in the Building Code shall not exceed the following values:

Kind of Stress	*Permissible Unit Stress (Pounds per Square Inch)*
Tension on section through throat of butt weld	Same as corresponding allowable stress for base metal
Compression on section through throat of butt weld	Same as corresponding allowable stress for base metal
Shear on section through throat of butt weld	Same as corresponding allowable stress for base metal
Shear on section through throat of fillet weld, or on faying surface area of plug or slot weld	13,600

(*b*) Fiber stresses due to bending shall not exceed the values prescribed above for tension and compression, respectively.

(*c*) Stress in a fillet weld shall be considered as shear on the throat, for any direction of the applied stress.

(*d*) Plug or slot welds shall not be ascribed any value in resistance to stress other than shear.

205. Combined Stresses

In the case of fillet welds subjected to both shear and bending stresses, the maximum unit stress resulting from the combination shall not exceed that allowed for shear in Article 204.

206. Increased Unit Stresses

When the Building Code permits the use of increased unit stresses for stresses due to combinations of loads, or to secondary or erection stresses, a corresponding increase shall be applied to the allowable unit stresses in welds.

207. Effective Areas of Weld Metal

(*a*) The effective area of butt and fillet welds shall be considered as the effective length of weld times the effective throat thickness.

(*b*) The effective shearing area of plug and slot welds shall be considered as the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

(*c*) The effective area of fillet welds in holes and slots shall be computed as specified in Article 207 (*a*) for fillet welds, using for the effective length the length of centerline of the weld through the center of the plane through the throat. However, in the case of overlapping fillets, the area thus computed shall not exceed the nominal cross-sectional area of the hole or slot.

(*d*) The effective length of a fillet weld shall be the over-all length of full size fillet, including returns.*

(*e*) The effective length of a butt weld shall be the width of the part joined, when the ends of the weld are made as specified in Article 406 (*e*). A transverse skewed butt weld shall not be assumed in computations to be longer than the width of the joint or piece perpendicular to the direction of stress.

(*f*) The effective throat thickness of a fillet weld shall be the shortest distance from the root to the face of the diagrammatic weld. (The effective throat thickness of an equal-leg 45° fillet weld is 0.707 times the normal leg size of the weld.)

(*g*) The effective throat thickness of a complete-penetration butt weld shall be the thickness of the thinner part joined.

(*h*) The effective throat thickness of an incomplete-penetration butt weld shall, for design purposes, be considered as reduced in accordance with the provisions of Article 210.

PART III—DETAILS OF WELDED JOINTS

208. Qualification of Joint Types

In accordance with the provisions of Article 601 (*a*), the types of joints described in Articles 209, 210, 211, and 212 may be used without the procedure qualification tests prescribed in Article 601 (*b*).

209. Complete-Penetration Butt Welds

(*a*) Complete-penetration butt welds shall be those having thorough fusion of weld and base metal throughout the entire depth of the joint. All other butt welds shall be considered incomplete-penetration welds.

(*b*) Complete-penetration butt welds in the following joints are exempt from procedure qualification tests:

* See Article 211 (*f*).

Square-edge butt joints in material not more than $\frac{5}{16}$ in. thick, welded in the flat position only, on both sides, with separation of joint edges not less than one-half the thickness.* Gouging or chipping out the back side of the initial layer before depositing welds on the other side, prescribed in Article 406 (*d*) for complete-penetration welds, may be omitted.

Single-vee butt joints with included angle not less than 60°, and single-bevel butt joints with included angle not less than 45°; welded both sides with root face not exceeding $\frac{1}{8}$ in. and root openings not less than $\frac{1}{16}$ in.†

Single-vee and single-bevel butt joints with included angle not less than 45°; welded one side onto a backing of the specified base material, without root face and with root opening not less than $\frac{3}{16}$ in.‡

Double-vee butt joints with included angle not less than 60°, and double-bevel butt joints with included angle not less than 45°; with root face not exceeding $\frac{1}{16}$ in. and root opening not less than $\frac{1}{8}$ in.§

Single-J butt joints welded both sides, and double-J butt joints; both having included angles not less than 45° if made in the horizontal position and not less than 25° if made in the other positions; with root radius not less than $\frac{1}{2}$ in., root face $\frac{1}{16}$ in. to $\frac{3}{16}$ in., and root opening not exceeding $\frac{1}{8}$ in.‖

Single-U butt joints welded both sides, and double-U butt joints; both with included angle not less than 45° if made in the horizontal position and not less than 20° if made in other positions; with root radius not less than $\frac{1}{4}$ in., root face $\frac{1}{16}$ in. to $\frac{3}{16}$ in., and root opening not exceeding $\frac{1}{8}$ in.**

Single-bevel, double-bevel, single-J, and double-J butt joints, having included angles, root faces, root radii, and root openings as described herein, may be used for joining a grooved part to an extended surface disposed approximately at right angles thereto. All such joints shall be welded from both sides except that the single-bevel and single-J joints may be welded from one side only if completely fused to a backing of the specified base material.

(*c*) For single-bevel, double-bevel, single-J, and double-J butt joints made in the horizontal position, the square, or unprepared, edge shall be on the bottom part joined.

210. Incomplete-Penetration Butt Welds

(*a*) Incomplete-penetration butt welds may be used in the following joints but only at points where their failure would not in any way endanger the structure. They shall be made as nearly complete penetration as the method of welding will permit; but, where they are used to carry calculated stress, the effective throat area shall be deemed to be no more than as hereinafter provided.

* For illustration see Fig. C1, Appendix C.

† For illustration see Figs. C2 and C3, Appendix C.

‡ For illustration see Figs. C4 and C5, Appendix C.

§ For illustration see Figs C6 and C7, Appendix C.

‖ For illustration see Figs. C8 and C9, Appendix C.

** For illustration see Figs. C10 and C11, Appendix C.

Square-edge butt joints in material not more than $\frac{1}{8}$ in. thick, welded from one side.* The effective throat thickness shall be deemed to be only one-half the thickness of the parts joined, unless the abutting edges are separated by more than one-half the thickness of the thinner part joined.

Square-edge butt joints in material not more than $\frac{1}{4}$ in. thick, welded from both sides, with the abutting edges separated not more than $\frac{1}{16}$ in.† Gouging or chipping out the back side of the initial layer before depositing welds on the other side may be omitted. The effective throat thickness shall be deemed to be three-fourths of the thickness of the thinner part joined.

Square-edge butt joints in material not more than $\frac{1}{4}$ in. thick, welded from one side, with abutting edges separated by not less than one-half the thickness of the thinner part joined.‡ The effective throat thickness shall be deemed to be only three-fourths the thickness of the thinner part joined.

Single-vee butt joints, welded from one side only with included angle of not less than 60°; root face not exceeding $\frac{1}{8}$ in.; and root opening not less than $\frac{3}{32}$ in.§ The effective throat thickness shall be deemed to be three-fourths the thickness of the thinner part joined.

Single-bevel butt joints, welded from one side only with included angle of not less than 45°; root face not exceeding $\frac{1}{8}$ in.; and root opening not less than $\frac{3}{32}$ in.‖ The effective throat thickness shall be deemed to be three-fourths the thickness of the inner part joined.

(*b*) Incomplete-penetration butt welds, made from one side, between square-edged parts, or in single-vee or single-bevel grooves, shall not be used to carry calculated tensile stress, even on the basis of the prescribed reduced effective throat thickness, unless the joint is restrained to prevent any rotation which might result from the eccentricity of the weld with respect to the welded parts.

211. Fillet Welds

(*a*) Fillet welds may be used without procedure qualification tests but shall conform to the requirements prescribed in this Article and Article 404.

(*b*) *Size*

The relation between weld size and the maximum thickness of material on which various sizes of strength fillet weld may be used shall be in accordance with the following table:

Size of Fillet Weld (Inches)	*Maximum Thickness of Parts (Inches)*
$\frac{3}{16}$	$\frac{1}{2}$
$\frac{1}{4}$	$\frac{3}{4}$
$\frac{5}{16}$	$1\frac{1}{4}$
$\frac{3}{8}$	2
$\frac{1}{2}$	6
$\frac{5}{8}$	Over 6

* For illustration see Fig. C12, Appendix C.

† For illustration see Fig. C13, Appendix C.

‡ For illustration see Fig. C14, Appendix C.

§ For illustration see Fig. C15, Appendix C.

‖ For illustration see Fig. C16, Appendix C.

The maximum nominal size of fillet weld applied to a nominally square edge of a plate or section shall be $\frac{1}{16}$ in. less than the nominal thickness of the edge, and the nominal size of fillet weld used along the toe of an angle or the edge of a rounded or sloping flange shall not exceed three-fourths the nominal thickness of the angle leg or three-fourths the nominal edge thickness of the sloping flange;* except that, when required by the design conditions and specially designated on the drawings, fillet welds equal in size to the edge of a plate or rolled section may be used, provided that the weld is built out in such a manner as to insure full throat thickness, full fusion area, and no injury to the base metal that will reduce its thickness adjacent to the weld.

(c) *Length and Spacing*

The minimum effective length of a strength fillet weld shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed one-fourth of its effective length. [See Article 207 (*d*).] If longitudinal fillet welds are used alone in end connections, the length of each fillet weld shall be not less than the perpendicular distance between them. The transverse spacing of longitudinal fillet welds used in end connections shall not exceed 8 in., unless end transverse welds or intermediate plug or slot welds are used.

(*d*) *Intermediate Fillet Welds*

Intermediate fillet welds may be used to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest practical size. The effective length of any segment of intermittent fillet welding shall be not less than four times the weld size with a minimum of $1\frac{1}{2}$ in. The clear spacing between the effective lengths of such segments at the edges of plates and the unsupported edges of rolled shapes carrying calculated stress shall not exceed the following number of times the thickness of the thinner part joined:

16 times for compression

24 times for tension

and shall in no case be more than 12 in. The effective length of longitudinal fillet welds at the ends of built-up members shall be not less than the width of the component part joined.

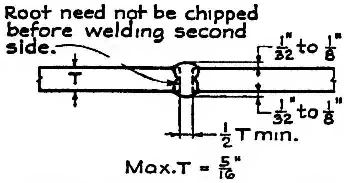
(*e*) *Lap Joints*

The minimum width of laps, on lap joints, shall be five times the thickness of the thinner part joined and not less than 1 in. Lap joints joining plates or bars subjected to axial stress shall be double fillet welded,† except where deflection of the

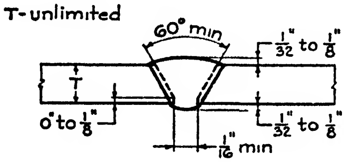
* For illustration see Fig. C17, Appendix C.
† For illustration see Fig. C18, Appendix C.

APPENDIX C (AWS)—WELDED JOINTS

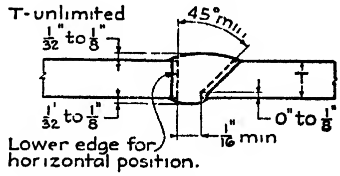
Covered by Articles 209, 210, and 211



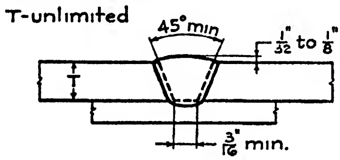
OPEN SQUARE-BUTT JOINT WELDED BOTH SIDES
FIG. C1



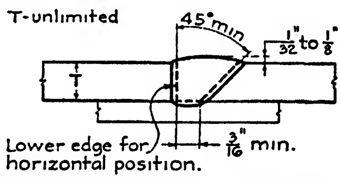
SINGLE-V BUTT JOINT WELDED BOTH SIDES
FIG. C2



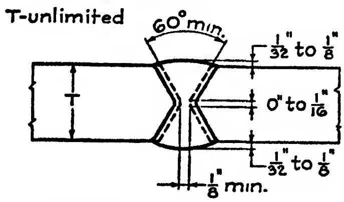
SINGLE BEVEL BUTT JOINT WELDED BOTH SIDES
FIG. C3



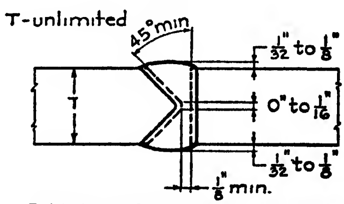
SINGLE-V BUTT JOINT, WELDED ONE SIDE ON BACKING STRUCTURE
FIG. C4



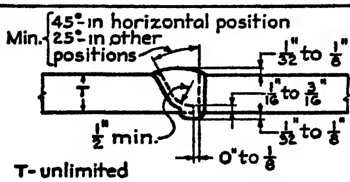
SINGLE BEVEL BUTT JOINT, WELDED ONE SIDE ON BACKING STRUCTURE
FIG. C5



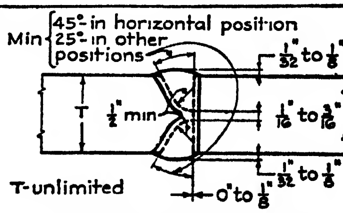
DOUBLE-V BUTT JOINT
FIG. C6



DOUBLE BEVEL BUTT JOINT
FIG. C7



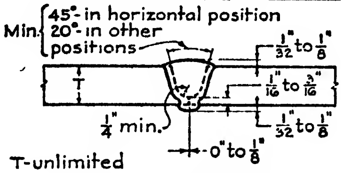
SINGLE-J BUTT JOINT WELDED BOTH SIDES
FIG. C8



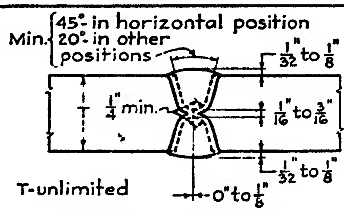
DOUBLE-J BUTT JOINT
FIG. C9

APPENDIX C (AWS)—WELDED JOINTS

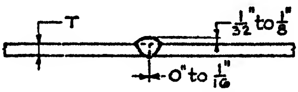
Covered by Articles 209, 210, and 211



SINGLE-U BUTT JOINT WELDED BOTH SIDES
FIG. C10

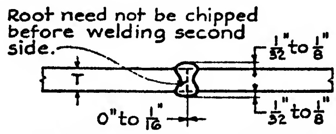


DOUBLE-U BUTT JOINT
FIG. C11



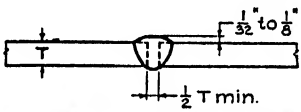
Effective Throat Thickness = 1/2 T
Max. T = 1/8"

SQUARE-BUTT JOINT WELDED ONE SIDE
FIG. C12



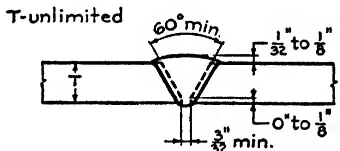
Effective Throat Thickness = 3/4 T
Max. T = 1/4"

SQUARE-BUTT JOINT WELDED BOTH SIDES
FIG. C13



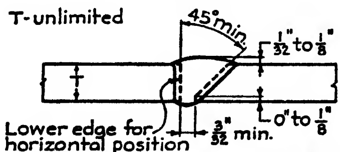
Effective Throat Thickness = 3/4 T
Max. T = 1/4"

OPEN SQUARE-BUTT JOINT WELDED ONE SIDE
FIG. C14



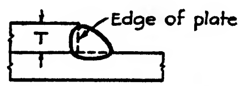
Effective Throat Thickness = 3/4 T

SINGLE-V BUTT JOINT WELDED ONE SIDE
FIG. C15

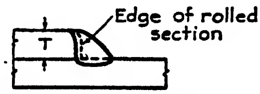


Effective Throat Thickness = 3/4 T

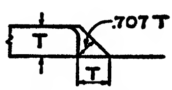
SINGLE BEVEL BUTT JOINT WELDED ONE SIDE
FIG. C16



Max. Effective Fillet = T - 1/16"



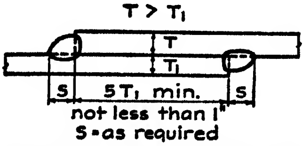
Max. Effective Fillet = 3/4 T



Max. Effective Fillet = T

EDGE FILLET WELDS
FIG. C17

T > T_1



DOUBLE FILLET WELDED LAP JOINT
FIG. C18

lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

(*f*) *End Returns* (*Boxing*)

Side or end fillet welds terminating at ends or sides, respectively, of parts or members shall, wherever practicable, be returned continuously around the corners for a distance not less than twice the nominal size of the weld. This provision shall apply to side and top fillet welds connecting brackets, beam seats, and similar connections, at the tension side of such connections, on the plane about which bending moments are computed. End returns shall be indicated on the design and detail drawings.

(*g*) Fillet welds in holes or slots may be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts. Such fillet welds may overlap, subject to the provisions of Article 207 (*c*).

212. Plug and Slot Welds

(*a*) Plug or slot welds may be used in plates not more than 1 in. thick, where subjected principally to shearing stresses or where needed to prevent buckling of lapped parts.

(*b*) The transverse spacing between slots shall not exceed 8 in. unless the design otherwise prevents excessive transverse bending in the connection.

(*c*) If the material is not over $\frac{5}{8}$ in. in thickness, the hole shall be filled with weld metal approximately flush with the surface of the part; if the material is over $\frac{5}{8}$ in. in thickness, the hole shall be filled with weld metal at least to $\frac{5}{8}$ in. in depth.

(*d*) Holes for plug welds shall be circular. The diameter of the hole shall be not less than the thickness of the part containing the hole plus $\frac{5}{16}$ in., rounded to the next greater odd sixteenth. It shall not be greater than 3 times the thickness of the weld metal.

(*e*) The width of slot shall be not less than the above-specified diameter for plug welds. The maximum length of slot shall not exceed 10 times the thickness of the part containing the slot.

(*f*) The ends of slots shall be semicircular or shall have the corners rounded to a radius not less than the thickness of the part containing the slot.

213. Eccentricity

In designing welded joints, adequate provision shall be made for bending stresses due to eccentricity, if any, in the disposition and section of base metal parts and in the location and types of welded joints.

214. Combinations of Welds

If two or more of the general types of weld (butt, fillet, plug, slot) are combined in a single joint, the effective capacity of each shall be separately computed with reference to the axis of the group, in order to determine the allowable capacity of the combination.

Part IV—Structural Details

215. Stitch Welds

If two or more plates or rolled shapes are used to build up a member, sufficient stitch welding (of the fillet, plug, or slot type) to make the parts act in unison shall be provided as follows, except where transfer of calculated stress between the parts joined requires closer spacing:

1. For plates, the longitudinal clear spacing between stitch welds shall not exceed the provisions of Article 211 (*d*), and the transverse spacing shall not exceed 32 times the thickness of the thinner plate joined.
2. For members composed of two or more rolled shapes, in contact one with another, the longitudinal spacing of stitch welds shall not exceed 24 inches or the limits prescribed in (3).
3. For members composed of rolled shapes, separated one from the other by a gusset plate, the component parts shall be stitched together at intervals such that the critical ratio, l/r, for each component between stitching shall not exceed three-fourths the critical ratio for the whole member.

216. Girders and Beams

Girders shall be proportioned by their moments of inertia. Butt-welded splices shall be made with complete-penetration butt welds. If the flanges are spliced, the splices shall either develop the full effective strength of the material or they shall develop the strength required by the total stresses, calculated on the assumption that all live and impact loads are increased 15 per cent, but in no case shall such strength developed be less than 50 per cent of the effective strength of the material spliced.

217. Connections of Tension and Compression Members in Trusses

Welded connections at ends of tension or compression members in trusses shall either develop the full effective strength of the material or they shall develop the strength required by the total stresses, calculated on the assumption that all live and impact loads are increased 15 per cent, but in no case shall such strength developed be less than 50 per cent of the effective strength of the material connected.

218. Milled Joints in Compression Members

(*a*) Where compression members are in full-milled bearing on base plates, and where full-milled tier-building columns are spliced, there shall be sufficient welding to hold all parts securely in place.

(*b*) Where other compression members are spliced by full-milled bearing, the splice material and its welding shall be arranged to hold all parts in line and shall be proportioned for 50 per cent of the computed stress.

(*c*) But all the foregoing joints shall be proportioned to resist any tension that would be developed by specified wind forces acting in conjunction with 75 per cent of the calculated dead load stress and no live load, if this condition of loading will produce more tension than with full dead load and live load applied.

219. Filler Plates

Fillers of less than $\frac{1}{4}$ in. thickness shall not be used to transfer stress but shall be trimmed flush with the welded edges of the stress-carrying element, and the sizes of the welds along the edges shall be increased over the required sizes by an amount equal to the thickness of the filler.

STANDARD SPECIFICATIONS FOR WELDED HIGHWAY AND RAILWAY BRIDGES

The following Sections, 1–8 inclusive, are mandatory upon all parties to any contract entered into as subject to the American Welding Society Specifications for Welded Highway and Railway Bridges, 1947.

SECTION 1. GENERAL PROVISIONS

101. Application

These Specifications are to be used in conjunction with the prescribed standard specification for the design and construction of highway or railroad bridges, as required.

102. Definitions

The welding terms used in these Specifications shall be interpreted according to the definitions given in Appendix B [AWS Standard Specifications for Welded Highway and Railway Bridges].

103. Welding

Metal-arc welding may be used in new work, in lieu of any riveting and bolting permitted by the standard specifications mentioned above; and may be used alone or in combination with riveting, bolting, or other means of fastening in the strengthening and repairing of old bridges provided that the base material shall conform to the provisions of Art. 105, and provided that all welds and welding shall conform, in design, execution, and results obtained, to the provisions of these Specifications.

104. Gas Cutting

Gas (flame) cutting may be substituted for shearing or planing in the preparation of ends and edges to be welded, provided that all processes pertaining thereto shall be executed, and results obtained, in accordance with these Specifications. This provision shall not be interpreted to amend the provisions of the applicable general specifications with respect to use of gas cutting for preparation of bearing or sliding surfaces or for other purposes prohibited by the general specifications.

105. Base Metal

(*a*) Steel base metal to be welded under these Specifications shall be, except as noted hereinafter in this Article, open-hearth or electric-furnace steel conforming to the latest revision of Specifications A7, Steel for Bridges and Buildings, of the American Society for Testing Materials.

(*b*) If steel base metal to be welded exceeds 1 in. in thickness, one of the following three additional stipulations shall apply, as determined in the inquiry and contract for the bridge or, if therein absent, by the contractor for fabrication:

1. The workmanship requirements of Art. 604 (*f*) shall be enforced.
2. A check test shall be made upon drillings from each part exceeding 1 in. in thickness; if such check test indicates that the carbon content plus one-sixth of the manganese content exceeds 0.40 per cent, the workmanship requirements of Art. 604 (*f*) shall be enforced.
3. The steel shall conform to the requirements of the latest amendment to Federal Specification, QQ-S-741, Grade A (Bridges), Type II (Welding Quality).

(*c*) These Specifications do not cover the welding of steel over 2 ½ in. thick.

(*d*) Wrought iron base metal shall conform to ASTM Standard Specifications as follows: for bars and shapes, A207; for plates, A42; for definitions of terms, A81.

SECTION 2. DESIGN OF NEW BRIDGES

Part I—General Requirements

201. Plans

Full and complete information regarding location, type, size, and extent of all welds shall be clearly shown on the plans. The plans shall clearly distinguish between shop and field welds.

Note shall be made on the drawings of those joints or groups of joints in which it is especially important that the welding sequence and technique of welding be carefully controlled to minimize shrinkage stresses and distortion, which sequence and technique shall be subject to the approval of the Engineer.

Weld lengths called for on the drawings shall be the required effective lengths.

202. Arrangement of Parts

Members and joints shall be so designed that the component parts may be readily assembled and securely held in correct position and alignment by means of clamps or other suitable devices, and with the welds so located as to be readily accessible for performing the welding operations.

Joints and splices shall be so located with relation to each other and with relation to attachments as to avoid large concentrations of welding.

203. Eccentricity

Eccentricity between intersecting parts and members, and of connections, shall be avoided insofar as practicable.

204. Ambiguity of Stresses

Structures shall be designed so as to avoid, as far as practicable, ambiguity in the determination of stresses.

205. Preference of Welding Positions

In the design of a member or joint, the position in which the welds are made shall have the following order of preference: butt welds—flat, overhead or vertical, horizontal; fillet welds—flat, horizontal, overhead or vertical.

206. Shop and Field Welds

In the design of members and joints, shop welds shall be used wherever possible and the use of field welds shall be kept to a minimum. Where practicable, field welded joints shall be located in zones of minimum stress.

207. Rivets and Bolts

In new work, rivets or bolts in combination with welds shall not be considered as sharing the stress, and welds shall be provided to carry the entire stress for which the connection is designed. Bolts or rivets used in assembly may be left in place if their removal is not specified.

Part II—Permissible Unit Stresses

208. General

Unit stresses specified herein are for welded joints in which shrinkage stresses due to welding and restraint of parts during cooling have been properly controlled and kept as low as possible by using a planned sequence and procedure of welding.

209. Normal Unit Stresses

(*a*) The normal unit stresses for base material shall, for the purpose of these Specifications, be the allowable unit stresses prescribed in the applicable general specifications except that for shear the normal allowable unit stress shall be 13,000 psi.

(*b*) The normal allowable unit stresses for butt welds shall be the same as prescribed for the base material parts which they join.

(*c*) The normal allowable shear unit stress for fillet, plug, and slot welds, computed in accordance with Arts. 211 to 214, shall not exceed 69 per cent of the normal allowable stress for the base material in tension.

(*d*) These allowable stresses are to apply on the effective areas of weld metal defined in Art. 213, and they also apply to tension, compression, or shear stresses that are due to the effects of bending.

(*e*) Normal unit stresses shall be used, except as noted in Art. 210 for proportioning the cross section of base material and the required effective area of welds to resist the maximum calculated stress.

(*f*) Where normal unit stresses are used and there is reversal of stress during the passage of live load, the maximum calculated stress shall be considered as the greatest stress of each kind increased by 50 per cent of the numerically smaller stress, and the base material and welds shall be proportioned to sustain either stress at the applicable normal unit stress. In highway bridges only 70 per cent of the dead-load stress shall be considered as counteracting live-load stress of the opposite sign.

TABLE 1. PERMISSIBLE UNIT STRESSES FOR WELDED

Loading Producing Maximum Stress			Base Metal in Axially Stressed Members Spliced or End-Connected by Fillet or Plug Welds		
	In Railway Bridges	In Highway Bridges	Form. No.	Max. Stress	Required Sectional Area
A	Short Critical Loading. (100 ft or less of loaded single track.)		1	Tension	$A = \frac{\text{Max.} - \frac{2}{3}\text{ Min.}}{7{,}500}$ but $\overline{\gtrless}$ Max./10,000 $\overline{\gtrless} \frac{\text{Min.} + \frac{1}{2}\text{ Min.}}{p}$
			2	Compression	$A = \frac{\text{Max.} - \frac{2}{3}\text{ Min.}}{7{,}500}$ but $\overline{\gtrless} \frac{\text{Max.} - \frac{1}{2}\text{ Min.}}{p}$ $\overline{\gtrless} \frac{\text{Max.}}{p}$
B	Longer Critical Loading. (More than 100 ft of loaded single track or any length of two or more tracks.) Use "A" loading if greater area is required thereby.	Short Critical Loading. (Not more than two panels or 60 ft of loaded single lane.)	3	Tension	$A = \frac{\text{Max.} - \frac{2}{3}\text{ Min.}}{10{,}500}$ but $\overline{\gtrless} \frac{\text{Max.}}{14{,}000}$ $\overline{\gtrless} \frac{\text{Min.} + \frac{1}{2}\text{ Min.}}{p}$
			4	Compression	$A = \frac{\text{Max.} - \frac{2}{3}\text{ Min.}}{10{,}500}$ but $\overline{\gtrless} \frac{\text{Max.} - \frac{1}{2}\text{ Min.}}{p}$ $\overline{\gtrless} \frac{\text{Max.}}{p}$
C		Longer Critical Loading. (More than two panels or 60 ft of loaded single lane or not more than two panels or 60 ft of loaded two lanes.) Use "B" loading if greater area is required thereby.	5	Tension	$A = \frac{\text{Max.} - \frac{2}{3}\text{ Min.}}{15{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{18{,}000}$ $\overline{\gtrless} \frac{\text{Min.} + \frac{1}{2}\text{ Min.}}{p}$
			6	Compression	$A = \frac{\text{Max.} - \frac{2}{3}\text{ Min.}}{15{,}000}$ but $\overline{\gtrless} \frac{\text{Max.} - \frac{1}{2}\text{ Min.}}{p}$ $\overline{\gtrless} \frac{\text{Max.}}{p}$
D		Loading in more than two panels or 60 ft of two or more lanes	Normal Unit Stresses (See Art. 209)		

A = required effective area; p = allowable unit compressive stress as prescribed by the general specifications for the type of member involved; the capacity of a full butt weld in compression being treated as identical with that of the connected material. "Max." and "Min." refer to total stresses. "Max." is the numerically greater stress.

HIGHWAY AND RAILWAY BRIDGES (SEE ART. 210)

Required Effective Weld Areas in All Members and Parts

Form. No.	Type of Weld	Max. Stress	Required Weld Area	Form. No.	Type of Weld	Max. Stress	Required Weld Area
7	Butt	Tension	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{13{,}500}$ but $\overline{\gtrless} \frac{\text{Max.}}{18{,}000}$	9	Butt	Shear	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{9{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{13{,}000}$
8	Butt	Com-pression	$A = \frac{\text{Max.}-\text{Min.}}{18{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{p}$	10	Fillet or Plug	Shear	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{7{,}200}$ but $\overline{\gtrless} \frac{\text{Max.}}{12{,}400}$
11	Butt	Tension	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{15{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{18{,}000}$	13	Butt	Shear	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{10{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{13{,}000}$
12	Butt	Com-pression	$A = \frac{\text{Max.}-\frac{8}{10}\text{ Min.}}{18{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{p}$	14	Fillet or Plug	Shear	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{10{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{12{,}400}$
15	Butt	Tension	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{18{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{18{,}000}$	17	Butt	Shear	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{12{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{13{,}000}$.
16	Butt	Com-pression	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{18{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{p}$	18	Fillet or Plug	Shear	$A = \frac{\text{Max.}-\frac{1}{2}\text{ Min.}}{12{,}000}$ but $\overline{\gtrless} \frac{\text{Max.}}{12{,}400}$

Normal Unit Stresses. (See Art. 209)

NOTE 1. See Art. 214(*a*) for enhanced allowable stresses for butt welds made smooth and flush.
NOTE 2. See Art. 214(*c*) for restriction on use of slot welds.

210. Design for Repeated Stress

The cross section of base material in axially stressed members spliced or connected by fillet or plug welds, and the required effective areas of all welds which are subject to conditions of loading producing reversal or fluctuation of stress many times repeated, shall be computed in accordance with the applicable formulas shown in Table 1 (p. 526-27).

In this table, "A" is the required effective area; the terms "Max." and "Min." refer to the maximum and minimum total stresses, respectively, computed in accordance with the requirements of the applicable general specifications.

"Max." refers to the numerically greater stress, of whichever sign, and is to be used in the design formulas of this Article as a plus quantity. "Min." refers to the numerically smaller stress; if it be of the same sign as "Max." it shall be used as a plus quantity, and if it be of opposite sign to "Max." it shall be used as a minus quantity in the design formulas.

NOTE: Formulas 1, 2, 7, 8, 9, and 10 apply only to railway bridges and are based on an occurrence of two million repetitions of the loading which would produce maximum stress in the member.

Formulas 3, 4, 11, 12, 13, and 14 apply to both railway and highway bridges and are based on an occurrence of 600,000 repetitions of the loading which would produce maximum stress in the member.

Formulas 5, 6, 15, 16, 17, and 18 apply only to highway bridges and are based on an occurrence of 100,000 repetitions of the loading which would produce maximum stress in the member. In any of the foregoing cases if traffic or loadometer surveys or other information indicate that a much greater number of repetitions of maximum load is expected during the life of the structure, the calculated areas of the members or parts affected should be multiplied by a "frequency factor."*

211. Combined Stresses

In the case of axial stress combined with bending, the allowable unit stress of each kind shall be governed by the requirements of Arts. 209 and 210, and the maximum combined unit stresses calculated therefrom shall be limited in accordance with the requirements of the applicable general specifications.

212. Increased Unit Stresses

When the applicable general specifications permit the use of increased unit stresses for stresses due to combinations of loads, or to secondary or erection stresses, a corresponding increase shall be applied to the allowable unit stresses in welds.

* Reports No. 3 (May 1943 *Welding Journal*) and 4 (January 1945 *Welding Journal*) of the Committee on Fatigue Testing (Structural), Welding Research Council, contain charts showing the approximate (inverse) relationship between numbers of repetitions endured and unit fatigue strength, for the more usual types of welded joints. The ratios therein suggested between the respective fatigue strengths at 2,000,000, 600,000 or 100,000 repetitions, and at the higher number of repetitions expected, will be a satisfactory "frequency factor" for the design method above recommended.

213. Effective Areas of Weld Metal

(*a*) The effective area of a butt or a fillet weld* shall be considered as the effective length of weld times the effective throat thickness.

(*b*) The effective shearing area of a plug or a slot weld shall be considered as the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

(*c*) The effective area of fillet welds in holes or slots shall be computed, as specified in Art. 213(*a*) for fillet welds, using for the effective length, the length of centerline of the weld through the center of the plane through the throat. However, in the case of overlapping fillets, the area thus computed shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

(*d*) The effective length of a fillet weld shall be the over-all length of full size fillet, including end returns.

(*e*) The effective length of a butt weld shall be the width of the part joined, assuming the ends of the weld are made as specified in Art. 606(*e*). A transverse skewed butt weld shall not be assumed in computations to be longer than the width of the joint or piece, perpendicular to the direction of stress.

(*f*) The effective throat thickness of a fillet weld shall be the shortest distance from the root to the face of the diagrammatic weld. (The effective throat thickness of an equal-leg 45° fillet weld is 0.707 times the nominal leg size of the weld.)

(*g*) The effective throat thickness of a butt weld shall be the thickness of the thinner part joined, with no allowance for reinforcement.

214. Use of Welds

(*a*) *Butt Welds.* The formulas for butt welds assume that there are no abrupt changes in the directions of lines of stress as they pass across the joint, that the reinforcement is very moderate and merges smoothly into the base metal, and that the parts are so arranged and held at the time of welding that the weld zone may contract without severe restraint.

Regardless of the range of stress and the number of repetitions of loading, butt welds in parts of equal thickness, which are particularly specified on the design drawings to be finished perfectly smooth and flush with the base material on all surfaces, may be computed for the maximum stress in compression, tension, or shear at the allowable normal unit stresses, as provided in the second paragraph of Art. 209. No weld thus specified shall have the excess weld metal removed by chipping, unless the chipping be followed by grinding to a smooth surface free from depressions and flush with the base material.

(*b*) *Fillet Welds.* Stress in a fillet weld shall be considered as shear on the throat, for any direction of the applied stress.

(*c*) *Plug and Slot Welds.* Neither plug nor slot welds shall be ascribed any value in resistance to calculated stresses other than shear. Further, slot welds shall not be considered to resist calculated shear, except where unit stresses for steady load are applicable. (This is not a restriction upon the use of fillet welds in slots to resist calculated shear.)

* Note differentiation between plug or slot welds and fillet welds in holes or slots. See Arts. 221 and 222.

Part III—Design of Welded Joints

215. Eccentricity of Connections

In designing welded joints, adequate provision shall be made for bending stresses due to eccentricity, if any, in the disposition and section of base metal parts and in the location and types of welded joints.

For members having symmetrical cross sections, the connection welds shall be arranged symmetrically about the axis of the member, or proper allowance shall be made for unsymmetrical distribution of stresses.

For members having unsymmetrical cross sections, such as angles, etc., the arrangement of connection welds shall be determined by approximately balancing the moments of their strength capacities about the gravity axis of the member.

216. Shrinkage

Joints shall be designed so as to minimize, as far as practicable, stresses due to the contraction of the weld metal and adjacent base metal upon cooling.

In the case of a member or part which must, in fabrication or erection, be welded at both ends into a rigid structure or assemblage, the design shall provide that the end connection last made shall be of such a type, or made in such a manner, as to avoid or minimize contraction in the direction of the length of such member or part. In such connections, for short members, no resistance to axial tension shall be ascribed to a weld lying transverse to the direction of the tension and made with the far end of the member fixed.

217. Combinations of Welds

If two or more of the general types of weld (butt, fillet, plug, slot) are combined in a single joint, the allowable capacity of each shall be separately computed with reference to the axis of the group, in order to determine the allowable capacity of the combination.

However, butt welds in parts or members, the design of which is governed by Art. 210, shall not be supplemented by splice straps attached by fillet, plug, or slot welds.

218. Prohibited Types of Joint

Prohibited types of joints are:

1. Butt joints not fully welded throughout their cross section.
2. Butt joints welded from one side only unless completely fused to a backing of the specified base material.
3. Butt joints made with intermittent welds.
4. Edge or corner joints made with bead welds unless used for sealing purposes only.

Typical sketches of some prohibited joints are shown in Figs. 2-*A*, 2-*D*, 2-*I*, 2-*N*, 2-*R*.

219. Qualification of Joint Types

(*a*) The types of joint described in Arts. 220, 221, and 222 may be used without performing the procedure qualification tests prescribed in Art. 702.

(*b*) Joints may deviate in form from the specific design requirements of Arts. 220, 221, and 222 only on condition that the Contractor shall at his own expense demonstrate the efficiency of his proposed joint or joints and the procedure for welding them, in the manner prescribed in Appendix D, Part I, Procedure Qualification [AWS Standard Specifications for Welded Highway and Railway Bridges], and shall thereby obtain the approval of the Engineer.

220. Details of Butt Weld Design

(*a*) Square-groove butt joints may be used to join material not thicker than ¼ in., and with separation not less than as shown in Figs. 2-*B* and 2-*C*.

(*b*) The following forms of grooved butt joints may be used for joining parts edge to edge:

Single vee, as shown in Figs. 2-*E*, 2-*F*.
Double vee, as shown in Figs. 2-*G*, 2-*H*.
*Single bevel, as shown in Fig. 2-*J* and (not shown) similar to Fig. 2-*K* (p. 533).
Double bevel (not shown) similar to Fig. 2-*L*.
*Single J, as shown in Fig. 2-*O*.
Double J, as shown in Figs. 2-*P*, 2-*Q*.
Single U, as shown in Fig. 2-*S*.
Double U, as shown in Figs. 2-*T*, 2-*U*.

(*c*) The following forms of grooved butt joints may be used for joining a grooved part to an extended surface at right angles therewith:

Single bevel, as shown in Fig. 2-*K* or (not shown) similar to Fig. 2-*J*.
Double bevel, as shown in Figs. 2-*L*, 2-*M*.
Single J (not shown) similar to Fig. 2-*O*.
Double J (not shown) similar to Figs. 2-*P*, 2-*Q*.

Grooving is not required, if the butting plate is not over ¼ in. in thickness, a root opening of one-half the thickness is provided, and the joint is welded from both sides. Compare Fig. 2-*C*.

(*d*) The forms for grooved butt joints for joining parts of unequal thickness are shown in Figs. 2-*V* to 2-*Z*, inclusive.

(*e*) The forms permitted under Paragraphs (*a*), (*b*), (*c*), and (*d*) hereof may be used only within the limits of maximum plate thickness and minimum included angles and with the root face and root opening dimensions, root radii, and requirements as to reinforcement or surface contour, shown in the respective figures and described in this clause (*e*), unless the Engineer approves deviations therefrom that may be proposed by the Contractor, which deviations shall be established as provided in Art. 219(*b*), before they are incorporated in the final shop details.

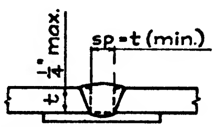
The minimum angles of bevel for various grooved joints are set forth in Table 2 (p. 534). (Note that the minimum included angle for J and U joints varies with the position of welding.)

* The J and U joints are, however, difficult to make properly in the horizontal position, in which case bevel or vee joints are to be preferred. Bevel joints for joining parts edge-to-edge shall preferably be used in only the flat and horizontal positions, and J grooves for joining such parts in only the flat position.

GENERAL NOTE: The height of reinforcement for butt welds shall not exceed 1/8 inch. See Art. 220 for root face dimensions and other details not shown in sketches.

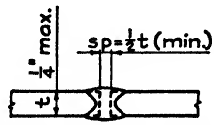
Welded from one side only.
Not permitted by specs.

FIG. 2-A



Welded from one side only, on backing material.

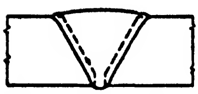
FIG. 2-B



Welded from both sides.

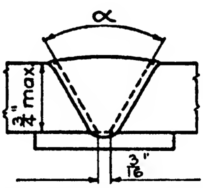
FIG. 2-C

SQUARE BUTT JOINTS



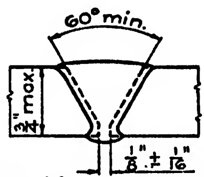
Welded from one side only.
Not permitted by specs.

FIG. 2-D



Welded from one side only, with backing material.
Included angle α-
60° for root opening shown
45° " " " 5/16"

FIG. 2-E



Welded from both sides.

FIG. 2-F

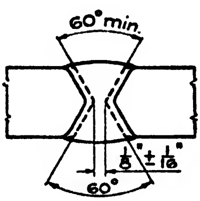


FIG. 2-G

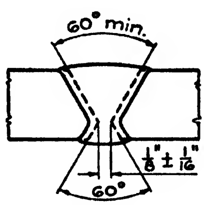
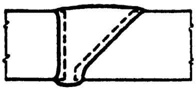


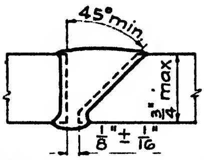
FIG. 2-H

VEE BUTT JOINTS



Welded from one side only.
Not permitted by specs.

FIG. 2-I



Welded from both sides.

FIG. 2-J

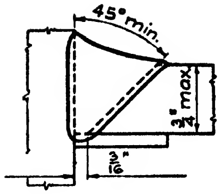


FIG. 2-K

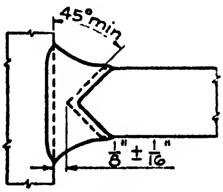


FIG. 2-L·

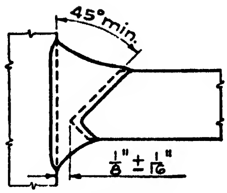
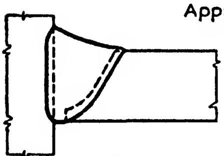


FIG. 2-M

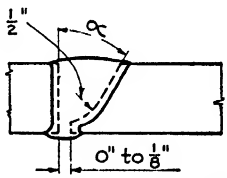
BEVEL JOINTS

Included angle α -
20° min. for flat and overhead welding.
45° min. for vertical and horizontal welding.



Welded from one side only.
Not permitted by specs.

FIG. 2-N



Welded from both sides.

FIG. 2-O

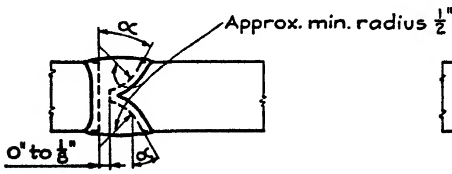


FIG. 2-P

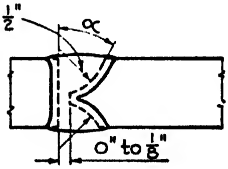
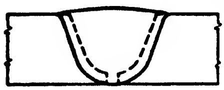


FIG. 2-Q

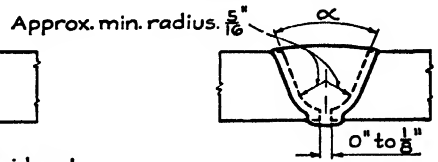
J JOINTS

Included angle α-
20° min. for flat and overhead welding.
45° min. for vertical and horizontal welding.



Welded from one side only.
Not permitted by specs.

FIG. 2-R



Welded from both sides.

FIG. 2-S

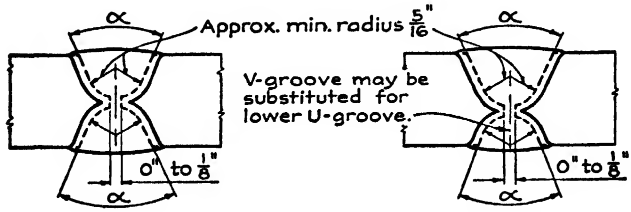


FIG. 2-T

FIG. 2-U

U-BUTT JOINTS

The width of root face, where used, shall be not more than 1/16 in. for parts less than 3/8 in., nor more than 1/8 in. for parts 3/8 in. or more in thickness.

TABLE 2

Type of Joint Preparation	Minimum Included Angle for Grooved Joints			
	All Positions	Flat and Overhead	Vertical	Horizontal
1. Single vee	See Figs. 2-*D*, 2-*E*, and 2-*F*			
2. Double vee	60°			
3. Single bevel	45°			
4. Double bevel	45°			
5. Single J		20°	45°	45°
6. Double J		20°	45°	45°
7. Single U		20°	45°	45°
8. Double U		20°	45°	45°

NOTE: See Art. 220 (*b*) for preference in use of bevel, vee, J, and U grooves for various positions.

The detail drawings shall indicate root edges or root faces and root openings complying with the approved welding procedure requirements, to assure thorough fusion of the base metal parts. (These dimensions vary with different types of joints, welding processes and procedures, and varying thicknesses of base metal. It may or may not be necessary to separate J- and U-joint edges prior to welding.)

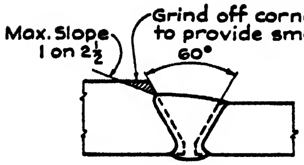


FIG. 2-V

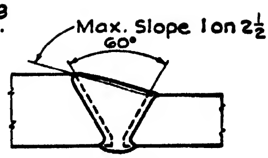


FIG. 2-W

(For lesser differences in thickness of parts joined.)

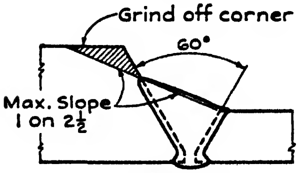


FIG. 2-X

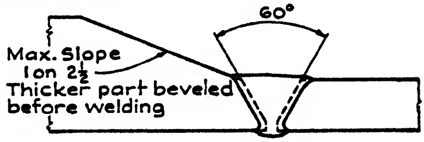


FIG. 2-Y

(For greater differences in thickness of parts joined.)

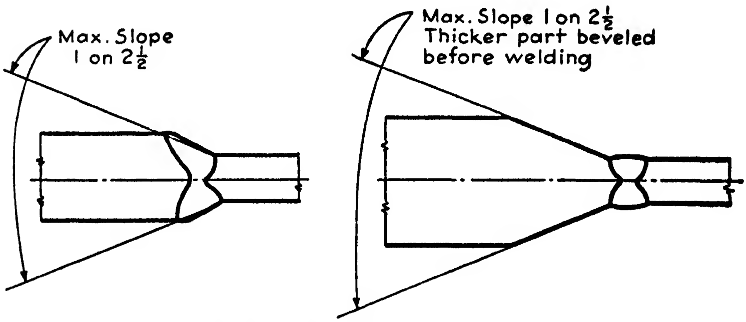


FIG. 2-Z

(For centerline alignment of parts)

Note: The details shown in Fig. 2-V, W, X, and Y for single V grooves may be adapted to grooves with backing material and to single U, J, or bevel grooves; also to double V, U, J, or bevel grooves joining concentric parts, or joining parts disposed as shown.

Butt Joints Between Parts of Different Thickness

The standard provisions of these Specifications for form and dimensions of grooved butt welds, and their root openings, are indicated in Fig. 2.

Butt welds shall be so proportioned that their surface contours will lie in gradual transition curves. For butt welded joints between base metal parts of unequal thicknesses, a transition shall be provided on a slope or bevel not greater than 1 in 2½ to join the offset surfaces. This transition may be provided by sloping the surface of the weld metal or by beveling the thicker part, or by a combination of these two methods. See Figs. 2-*V* to 2-*Z* for typical examples.

221. Details of Fillet Weld Design

(*a*) The relation between weld size and the maximum thickness of material on which various sizes of fillet weld may be used shall be in accordance with the following table:

Size of Fillet Weld (Inches)	*Maximum Thickness of Parts (Inches)*
3/16	1/2
1/4	3/4
5/16	1 1/4
3/8	2
1/2	6
5/8	Over 6

The maximum size of fillet weld that may be used along the edge of material 1/4 in. or more in thickness shall be 1/16 in. less than the thickness of the material. Fillet welds along the edge of material less than 1/4 in. in thickness may have their size equal to the thickness of the material. When required by the design conditions and specially designated on the drawings for material 1/4 in. or more in thickness, fillet welds equal in size to the thickness of the edge they connect may be used, provided that the weld is built out to obtain full throat thickness and full fusion area, and provided there is no injury to the base metal that reduces its thickness adjacent to the weld. (See Fig. 2-*AA*.)

(*b*) The minimum effective length of a fillet weld shall be four times its size and in no case less than 1 1/2 in.

(*c*) Fillet welds terminating at the corners of parts or members shall, wherever practicable, be returned continuously full size around the corners for a distance not less than twice the nominal size of the weld. End returns shall be indicated on the design and detail drawings.

(*d*) Intermittent fillet welds preferably shall not be used. They shall be permitted only where the required weld area is less than that of a continuous fillet weld of the minimum permissible size. If used on main members, they shall be chain intermittent welds. In all other cases, chain intermittent welding is preferable to staggered intermittent welding.

Spacing of intermittent fillet welds shall be measured between the centers of the weld segments. The clear spacing shall in no case be greater than that specified in Art. 228.

(*e*) Fillet welds in holes or slots may be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts. The fillet welds in a hole or slot may overlap, subject to the provisions of Art. 213(*c*).

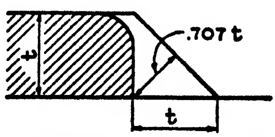


FIG. 2-AA—Nominal fillet weld throats along rolled edges.

(*f*) Seal welding shall preferably be accomplished by a continuous weld combining the functions of sealing and strength, changing section only as the required strength may necessitate.

222. Details of Plug and Slot Weld Design*

(*a*) Plug and slot welds may be used to transmit shear in a lap joint [except as noted in Art. 214(*c*)] or to prevent the buckling or separation of lapped parts. They shall conform in details to the requirements of this Art. 222 and the requirements of Art. 605(*j*) and 605(*k*).

(*b*) The diameter of the hole for a plug weld shall be not less than the thickness of the part containing it plus 5⁄16 in. preferably rounded to the next greater odd 1⁄16 in., nor shall it be greater than 2¼ times the thickness of the weld.

(*c*) The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

(*d*) The length of the slot for a slot weld shall not exceed ten times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus 5⁄16 in. preferably rounded to the next greater odd 1⁄16 in., nor shall it be greater than 2¼ times the thickness of the weld.

The ends of the slot shall be semicircular or shall have the corners rounded to a radius not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

(*e*) The minimum spacing of lines of slot welds in a direction transverse to their length shall be 4 times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be 2 times the length of the slot.

(*f*) The thickness of plug or slot welds in material ⅝ in. or less in thickness shall be equal to the thickness of the material. In material over ⅝ in. in thickness it shall be at least one-half the thickness of the material but not less than ⅝ in.

Part IV—Structural Details

223. General

Details shall in general be such as to minimize constraint against ductile behavior, to avoid undue concentration of welding, and to afford ample access for the placing of the weld metal.†

224. Non-Continuous Beams

The connections at the ends of non-continuous beams shall be designed with flexibility so as to avoid excessive secondary stresses due to bending. Seated connections with a flexible or guiding device to prevent end twisting are recommended.

* See Art. 221(*e*) for fillet welds in holes and slots.

† An example of detailing to avoid constraint is the clipping of stiffeners, brackets, and diaphragms instead of fitting them tightly into corners between flanges and webs of girders or beams.

225. Participation of Floor System

Floor system details shall be such as to minimize the participation of the floor system in chord or flange stresses.

226. Lap Joints

(*a*) The minimum overlap of parts at stress carrying lap joints shall be 5 times the thickness of the thinner part. Unless lateral deflection of the parts is prevented, they shall be connected by at least two transverse lines of fillet, plug, or slot welds or by two or more longitudinal fillet or slot welds.

(*b*) If longitudinal fillet welds are used alone in lap joints of end connections, the length of each fillet weld shall not be less than the perpendicular distance between them. The transverse spacing of the welds shall not exceed 16 times the thickness of the thinner part connected unless suitable provision is made (as by intermediate plug or slot welds) to prevent buckling or separation of the parts. The longitudinal fillet welds may be either at the edges of the member or in slots.

(*c*) When fillet welds in holes or slots are used, the clear distance from the edge of the hole or slot to the adjacent edge of the part containing it, measured perpendicular to the direction of stress, shall not be less than 5 times the thickness of the part or less than 2 times the width of the hole or slot. The strength of the part shall be determined from its critical net section.

227. Tee and Corner Joints

(*a*) Tee and corner joints whose parts intersect at an angle between 60° and 120° may be connected either by fillet welds or by butt welds. Fillet welds, where used, shall be provided on both sides of the joint unless suitable provision is made to prevent lateral deflection of the parts. Fillet welds shall not be used to connect joints

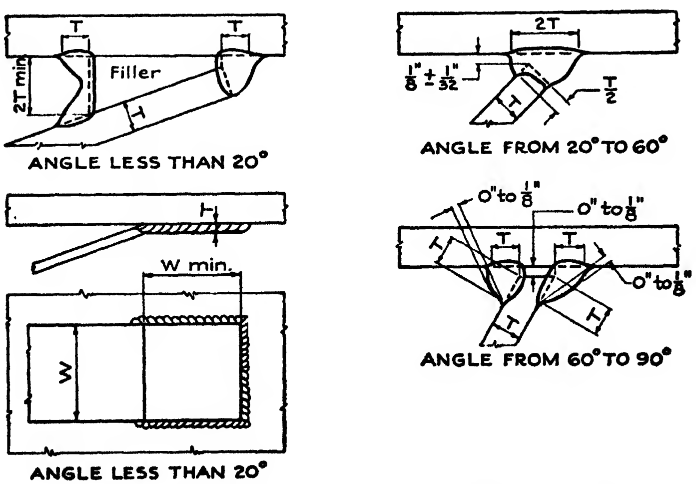


FIG. 2-BB—Joints between various parts at various angles.

whose angle of intersection is beyond the above limits; only butt welds, details of which are similar to those shown in Fig. 2-*BB*, or other suitable details shall be used.

(*b*) Tee and corner joints that are to be subjected to bending about an axis parallel to the joint shall have their welds arranged to avoid concentration of tensile stress at the root of any weld. In addition to any other necessary weld there shall be a continuous fillet weld in any re-entrant corner that is subjected to tensile stress. Butt welds with fillet welds, not larger than 3/8 in. along their faces to avoid abrupt changes in stress path, are preferred for such joints and shall be used when there is widely fluctuating repeated stress.

228. Stitch Welding

If two or more plates or rolled shapes are used to build up a member, sufficient stitch welding (fillet, plug, or slot welds) to make the parts act in unison shall be provided as follows unless calculated stress between the parts joined requires closer spacing:

1. At the ends of members there shall preferably be continuous longitudinal fillet welds at least as long as the width of the element or member being connected. Alternately there shall be plug or slot welds spaced at their minimum allowable spacing [see Arts. 222(*c*) and 222(*e*)] for a distance equal to 1½ times the width of the member.
2. The clear spacing in the direction of stress of stitch welds that connect plates to other plates or to shapes shall not exceed:
   Compression members..10 times the thickness of the thinner part but not more than 12 in.
   Tension members.......14 times the thickness of the thinner part but not more than 12 in.
   The spacing transverse to the direction of stress shall not exceed 24 times the thickness of the thinner part connected.
3. For members composed of two or more rolled shapes, in contact one with another, the longitudinal spacing of stitch welds shall not exceed 24 in. or the limits prescribed in (4).
4. For members composed of rolled shapes, separated one from the other by the thickness of a gusset plate, the component parts shall be stitched together at intervals such that the critical ratio, l/r, for each component between stitching shall not exceed three-fourths the critical ratio for the whole member.

In all cases sufficient weld strength shall be provided to transmit the shearing stresses between the component parts that are caused by flexure due to long-column action, applied bending moments, and any beam reactions or other loads that tend to stress the parts unequally.

229. Girders and Beams

(*a*) Girders shall be proportioned by their moments of inertia.

(*b*) Splices in beams or girders shall either develop the full effective strength of the material or they shall develop the strength required by the total stresses, calculated on the assumption that all live and impact loads are increased 15 per cent, but in no case shall the strength developed be less than 50 per cent of the effective strength of the material spliced, nor shall butt joints be only partially welded.

(*a*) Stiffeners may be either shapes, bars, or plates welded to the web by continuous or intermittent welds designed to transmit the stresses. Ends of stiffeners and other attachments may be welded to flanges only at points where the flanges carry compressive stress or where tensile stress does not exceed 75 per cent of the maximum allowable stress permitted by the applicable general specification.

(*d*) The number of flange plates shall preferably be limited to one for each flange. The thickness of this plate may be stepped down as required, the abutting ends of the different thicknesses being butt welded. The unsupported projection of flange plates shall not exceed 12 times their thickness.

(*e*) The maximum unit stress in the tension flange of a beam or girder at the point of termination of a welded cover or splice plate shall not exceed that which would be permitted for a comparable range between maximum and minimum stress by Formula 1, 3, or 5, whichever is applicable, of Table 1, unless details are used which have been proved by tests to be satisfactory without the use of such reduction in allowable stress.

230. Connections or Splices—Tension and Compression Members

Welded end connections or splices of tension or compression members shall, except as noted in Art. 231, either develop the full effective strength of the material, or they shall develop the strength required by the total stresses, calculated on the assumption that all live and impact loads are increased 15 per cent, but in no case shall the strength developed be less than 50 per cent of the effective strength of the material connected, nor shall butt joints be only partially welded.

231. Connections or Splices—Compression Members with Milled Joints

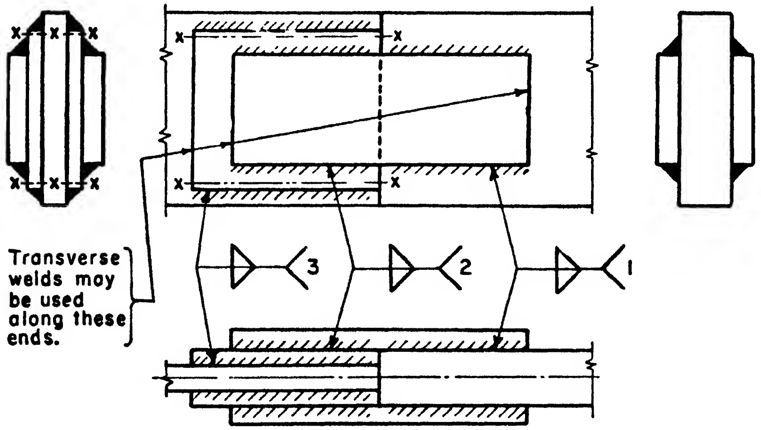
Where compression members are spliced with full milled bearing, unless otherwise stipulated by the applicable general specifications, the splice material and its welding shall be arranged to hold all parts in alignment and shall be proportioned to carry 50 per cent of the computed stress in the member. Where compression members are in full milled bearing on base plates, there shall be sufficient welding to hold all parts securely in place.

232. Attachments to Tension Members

Brackets, clips, gussets, stiffeners, and other detail material shall preferably not be welded to members or parts subjected to a wide range of tensile stress. In the case of railway bridges, if such attachments are welded to members subject to reversal, the allowable unit stress in the member or part to which the attachment is made shall not exceed three-fourths of the unit stress permitted by the applicable general specifications.

233. Fillers in Splices

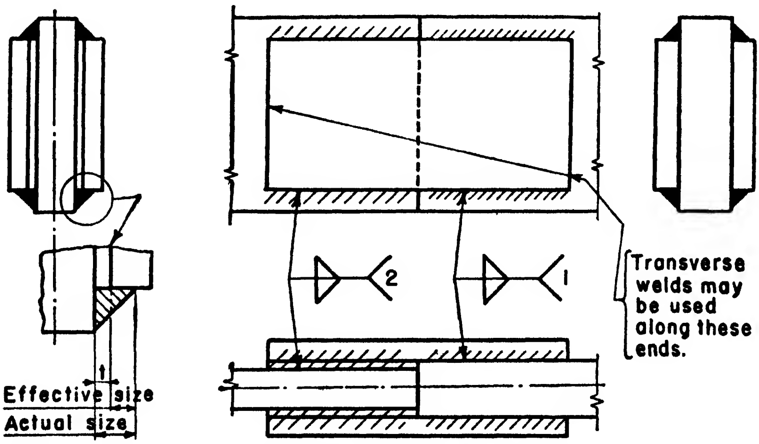
(*a*) Fillers shall be used where parts of different thickness or members with offset surfaces are connected with splice plates.



EFFECTIVE AREA OF WELDS (2) SHALL EQUAL THAT OF WELDS (1). THE LENGTH OF WELDS (2) SHALL BE SUFFICIENT TO AVOID OVERSTRESSING THE FILLER IN SHEAR ALONG PLANES X-X

EFFECTIVE AREA OF WELDS (3) SHALL AT LEAST EQUAL THAT OF WELDS (1) AND THERE SHALL BE NO OVERSTRESS OF THE ENDS OF WELDS (3) RESULTING FROM THE ECCENTRICITY OF THE FORCES ACTING ON THE FILLER.

FIG. 2-CC—FILLER 1/4 INCH OR MORE IN THICKNESS



EFFECTIVE AREA OF WELD (2) SHALL EQUAL THAT OF WELD (1) BUT ITS SIZE SHALL BE ITS EFFECTIVE SIZE PLUS THE THICKNESS OF THE FILLER, t.

FIG. 2-DD—FILLER LESS THAN 1/4-INCH THICK

(*b*) Any filler ¼ in. or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate stress, applied at the surface of the filler as an eccentric load. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate stress and shall be long enough to avoid overstressing the filler along the toe of the weld. [See Fig. 2-*CC*.]

(*c*) Any filler less than ¼ in. thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plate stress plus the thickness of the filler plate. [See Fig. 2-*DD*.]

APPENDIX A (AWS)

UNIT STRESSES FOR WELDED DESIGN

1. As stated in the Foreword to these Specifications, the unit-stress formulas of Arts. 209 and 210 are based upon repeated load, or "fatigue," experiments with structural steel and with welded joints; particularly upon those of the Committee on Fatigue Testing (Structural) of the Welding Research Council of the Engineering Foundation, References 1 to 5 in the Bibilography at the end of this Appendix.

2. Standard specifications for riveted bridges* recognize the effect of fatigue in cases of reversal of stress, the practical effect of their prescription being as follows:

	Allowable Unit Stress	
	Tension on Main Material	Shear on Rivets
Static, or variable but non-reversing stress	18,000	13,500
Zero-to-maximum	18,000	13,500
Full reversal	12,000	6,750

which may be represented graphically as shown in Fig. A-1.

(For a discussion of Figs. A–1 to A–6 in this Appendix and of their association with the unit-stress formulas of Arts. 209 and 210, see Appendix 3 of Reference 2. The definitions of scale shown in Fig. A–1 are understood to apply to all.)

The design rule thus represented is correct only as to its general intent. Assuming that it is correct for some particular number of stress cycles to be endured, it then might be unsafe for members subjected to many more cycles and wasteful for members subjected to fewer cycles. Furthermore, it implies that fatigue failure of a joint will not occur below the static yield point unless the minimum stress in the habitual cycle is of opposite sign to the maximum, which is not true.

It would therefore appear that these standard clauses should in due course be modified and, in particular, that they should be differentiated for railway and highway bridges, respectively, with respect to the probable number of cycles of maximum stress.

The formulas of Arts. 209 and 210 do not, therefore, imply that the situation as regards failure in fatigue is more serious or more complex in welded than in riveted

* Specifications for Steel Railway Bridges, AREA 1948, Art. 215. Specifications for Highway Bridges, AASHO 1949, Art. 3.6.5.

joints. They represent an effort, for the sake of safety with economy, to get closer to experimental facts than the writers of the standard specifications for riveted bridges have as yet done.

3. It has always been intended in these Specifications, as indicated by the wording of Art. 209, that main material requirements should be governed by the applicable general specification, except as differences may be introduced by the welding.

In butt-welded tension members the main material may or may not be increased because of the area required in the butt welds to meet fatigue conditions (as later discussed in Art. 6); in riveted tension members it is always increased, to compensate for rivet holes. Therefore, with relatively few exceptions, less area of main material will be required for butt-welded than for riveted tension members.

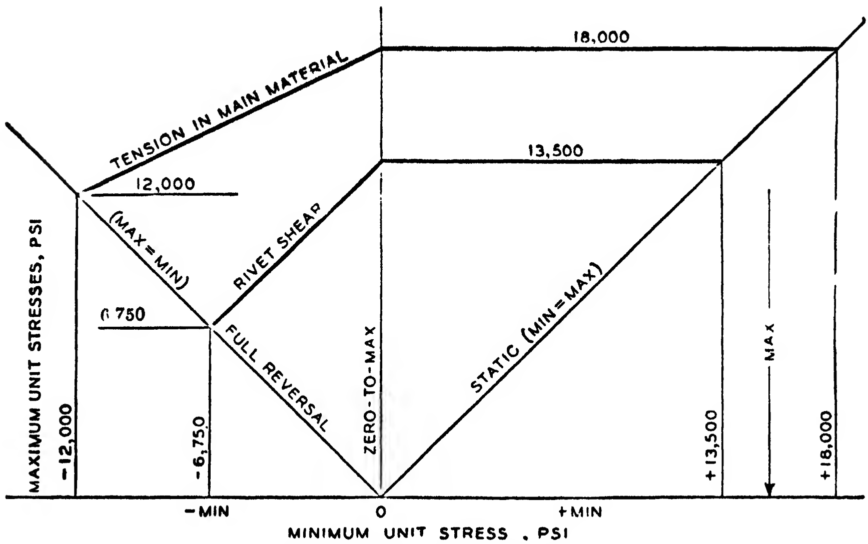


Fig. A-1—Current allowable unit stresses for riveted bridges.*

On the other hand, it having been found that material connected by fillet welding almost always fails, under repeated stress, in the main material adjacent to the fillets, at reduced unit stresses, and that this condition cannot be improved by additional welding or by any other practical treatment that has been demonstrated thus far, it has been necessary to set members so connected apart as a special case and to give them somewhat detailed treatment. A similar treatment has been prescribed for members or structural elements to which attachments have been made by fillet welding.

4. This special treatment is expressed in Formulas 1 to 6, inclusive, of Table 1, Art. 210. The underlying data are taken from Reference 5, Series D, in which fatigue tests were made on plates connected with fillet welds as indicated in Fig. A-2*a* (p. 544), the specimens being so designed as to result in failure of the plates.

* Specifications for Steel Railway Bridges, AREA 1948, Art. 215. Specifications for Highway Bridges, AASHO 1949, Art. 3.6.5.

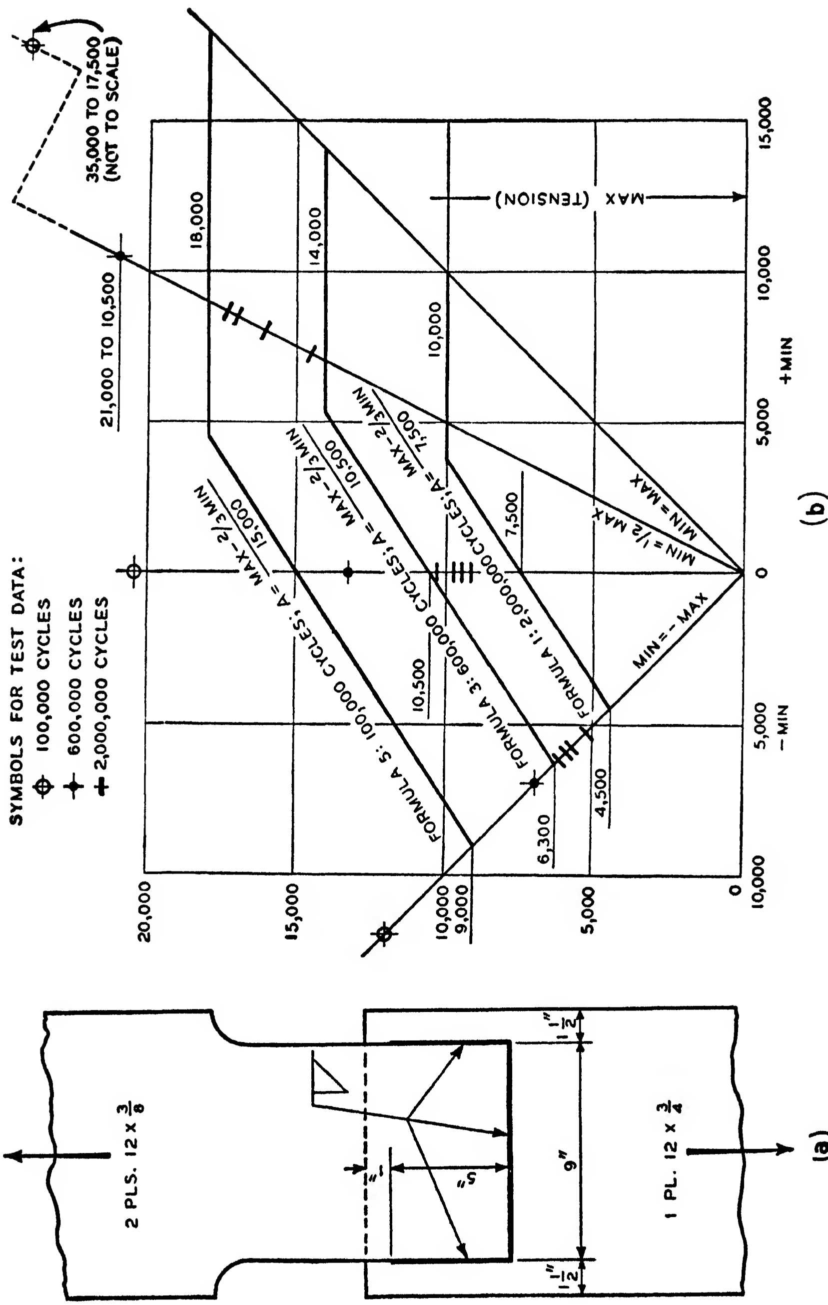
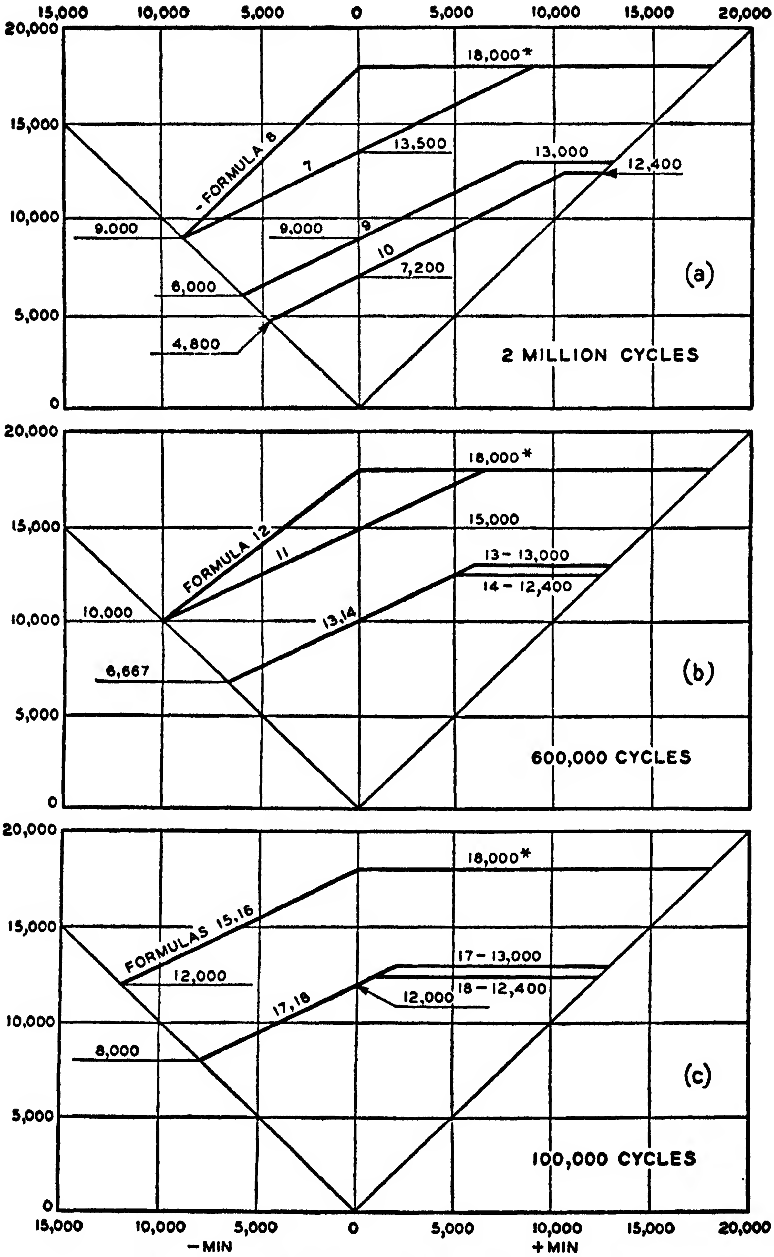


Fig. A-2—Allowable unit stress for main material in tension members connected by fillet welds.



* PLOTTED FOR THE MAXIMUM VALUE ASCRIBED TO THE VARIABLE "p" IN MOST SPECIFICATIONS FOR BRIDGES

Fig. A-3—Allowable unit stresses for weld metal.

This type of connection was found to be as efficient as any of several tried, for the development of the main material by fillet welding.

In Fig. A–2*b* (p. 544), the fatigue strengths calculated from the specimens of Series D are plotted in terms of pounds per square inch of tension on the main material. Three testing cycles are represented, being from left to right in the figure, (1) full reversal, (2) zero-to-tension, and (3) tension-to-tension one-half as great. Individual specimens are plotted (short dashes) for calculated fatigue strength at 2,000,000 cycles, and the weakest specimen in each group is also plotted as recalculated for fatigue strength of 600,000 cycles (filled dots) and at 100,000 cycles (open circles).

Formula 5 is plotted as the uppermost line of Fig. A–2*b* and expresses the allowable unit stress on those tension members in highway bridges which probably will not receive over 100,000 critical stress cycles Min.-to-Max. The degree of protection of members in this class may be estimated by comparing the uppermost line with the open circles.

Formula 3 is plotted as the next lower line and expresses the allowable unit stress on those tension members in highway bridges which may receive as many as 600,000 critical stress cycles Min.-to-Max. It is believed that this takes care of those members fully stressed by one or two vehicles in one lane. The line expressing this formula may be compared with the filled dots.

This formula applies also to the parts of railroad bridges which are less likely to be frequently loaded to the assumed maximum.

Formula 1 is plotted as the lowest line and expresses the allowable unit stress on these tension members in railway bridges which may receive 2,000,000 critical stress cycles Min.-to-Max. The degree of protection may be estimated by comparing this line with the test results expressed as dashes.

To avoid confusion, Formulas 6, 4, and 2 of Art. 210 have not been plotted in Fig. A–2*b*. Applying as they do to compression members, they would each plot as a sheaf of lines dependent upon the variation of the ratio l/r. In general they would plot below the three lines shown.

5. The formulas for unit stress on weld metal are plotted in Fig. A–3 for members subject to 2,000,000, 600,000, and 100,000 cycles respectively.

This parallel representation to the same scale shows the decreasing severity of the formulas for the three cases, based upon the decreasing number of probable stress cycles, Min.-to-Max.

6. The three formulas 7, 11, and 15 for butt welds in members subjected to stress cycles in which Max. is tension, are grouped in Fig. A–4 for comparison with the test data.

The test data plotted on Fig. A–4 are taken from Figs. 3, 4, and 5 of Reference 3. Figs. 3 and 4 thereof show the results of individual tests, calculated to 100,000 and 2,000,000 cycles, and also the values considered "dependable." By the aid of Fig. 5 (of Reference 3) "dependable" values can be interpolated at 600,000 cycles.

In Fig. A–4 the "dependable" fatigue strengths at 2,000,000 cycles are plotted as dashes; a comparison thereof with the lowest line, Formula 7, indicates the degree of protection for butt welds in members subject to 2,000,000 cycles of maximum stress. The dependable fatigue strengths at 600,000 cycles are plotted as filled dots, and may be compared with the middle line, Formula 11. The dependable fatigue strengths at 100,000 cycles are plotted as open circles and may be compared with the upper line, Formula 15.

The margins thus indicated, for wide variations from Min. to Max., are obviously less than the customary 33/18 for static loading. This, the Committee believes, is as it should be; as the margin against an event that will certainly happen once, or a few times, must be greater than for a succession of events which may never accumulate.

It will be noted that Formulas 7, 11, and 15 all permit 18,000 psi on butt welds in tension, for static stress and for three different ranges of tensile Min. to tensile Max.

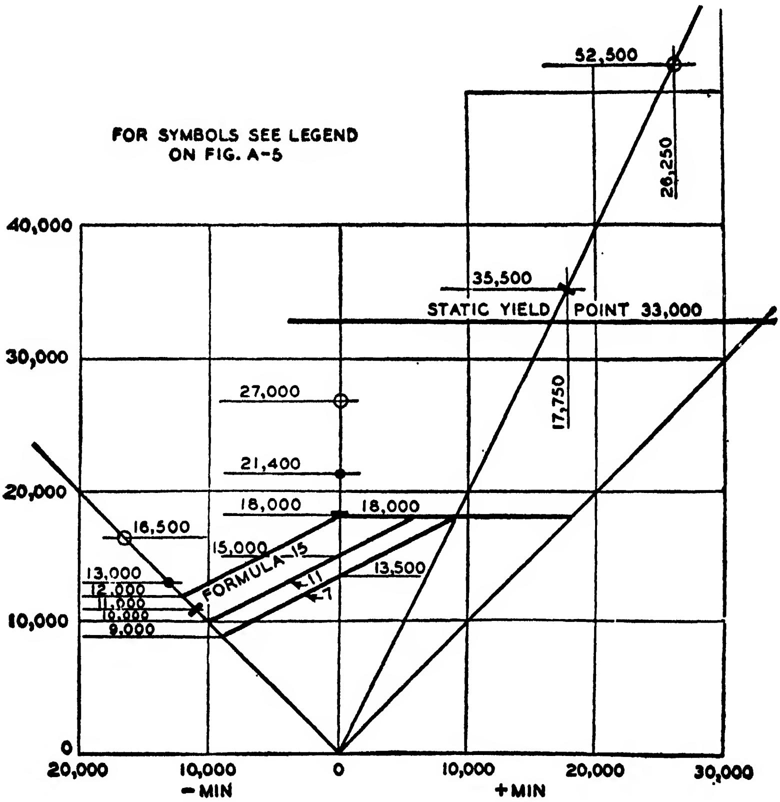


FIG. A-4—Butt welds in tension members.

This is justified by the test data shown and should be of considerable economic value in permitting full development of the base metal area in cases of the applicable stress ranges. Inasmuch, however, as the allowance for main material in tension, by the applicable general specifications, follows the line of Formula 15, whereas Formulas 11 and 7 lie below this for wide variations of stress, it follows that in railway bridges, and in the "short-loading" members of highway bridges, if the maximum stress is tensile and the range from Max. to Min. is considerable, the main material cannot be fully developed and will have to be increased, until its unit stress conforms to Formula 7, or Formula 11, respectively.

In flexural members, this requires the location of splices to be removed from points of maximum stress. In the case of a girder whose tension flange varies in thickness throughout its length, this means that the thicker flange plate nearer the point of maximum moment must be extended further than would otherwise be necessary, before it is butt welded to the end of the adjacent thinner flange plate. Unless and until some practical means is developed for increasing locally the base metal cross section of an axially stressed member adjacent to a weld, without sacrificing fatigue resistance, the cross section of the member must be increased throughout its entire length.

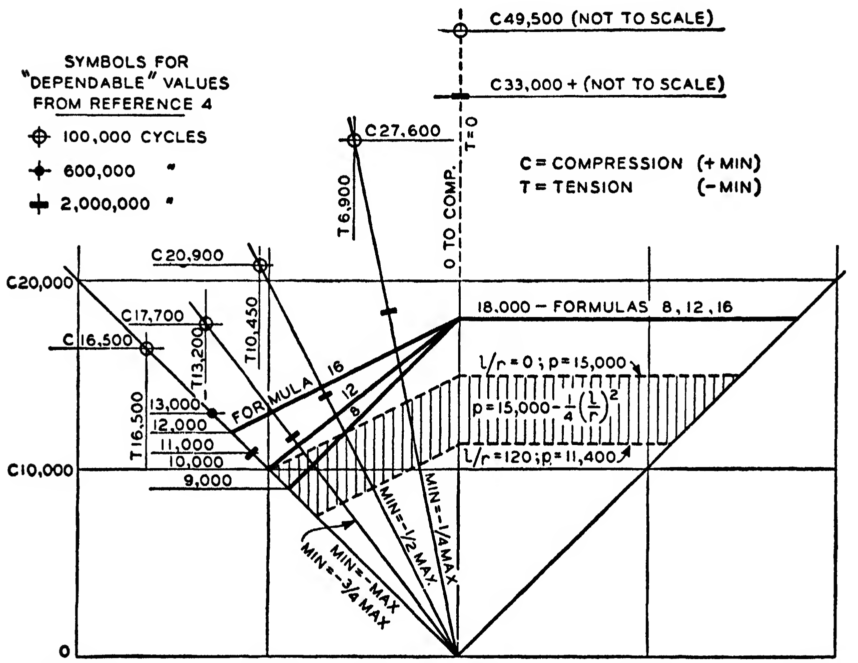


FIG. A-5—Butt welds in compression members.

This requirement cannot be avoided by adding fillet-welded splice plates, which under the stress conditions cited will weaken, rather than strengthen, the joint.

7. The three formulas 8, 12, and 16 for butt welds in members subjected to stress cycles in which Max. is compression are grouped in Fig. A-5 for comparison with the test data.

The "dependable" values for full reversal, plotted on the left-hand 45° line, are identical with those in Fig. A-4, since the cycles "compression-to-an-equal-tension" and "tension-to-an-equal-compression" are identical.

The remaining test data plotted in Fig. A-5 are from Reference 4, Table 4.

The cross-hatched area on Fig. A-5 covers the allowable unit stress, by the AREA and AASHO Specifications, on columns with length-ratios from 0 to 120. It is seen that only in railway bridges, and then only for short columns subject to almost complete reversal, will the allowable stress on the butt weld be insufficient to fully develop the main material.

8. The research cited in Art. 1 has not developed any data in direct support of Formulas 9, 13, and 17 for butt welds subjected to shear of cyclical variation. The Committee assumes that for wide variations it is conservative to allow two-thirds of the allowance for the same Min.-to-Max. ratio in tension. For narrow variations, approaching a static load, the Committee considered the figure of 11,000 psi given

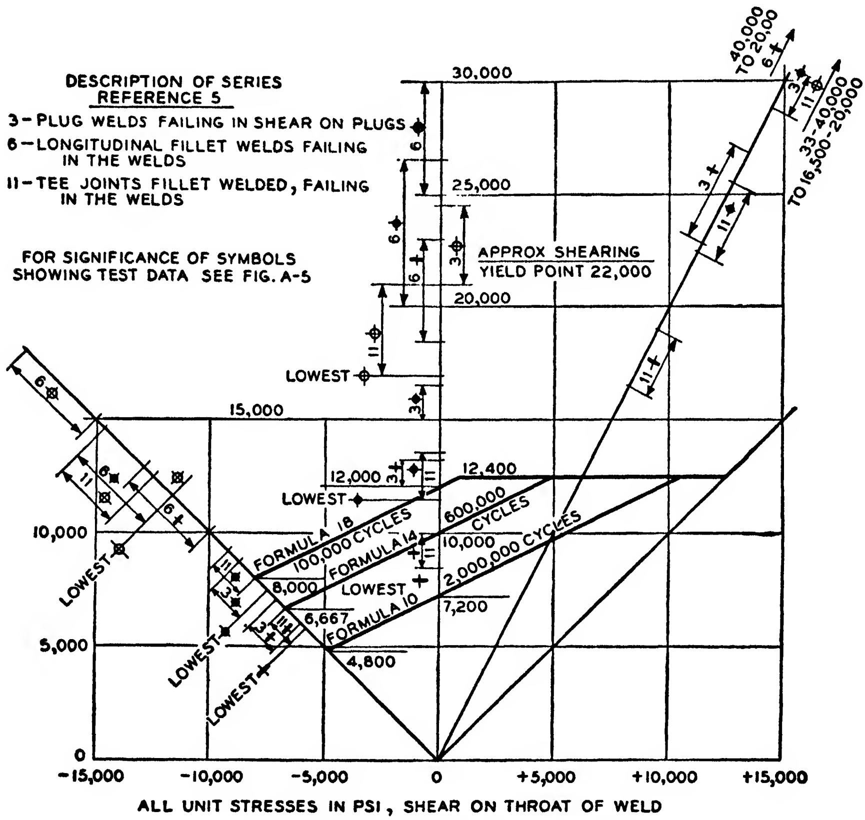


FIG. A-6—Fillet and plug welds in shear.

in standard specifications for allowance on gross area of girder webs, from which as a matter of fact a number of rivet holes would be deducted; it therefore decided to allow 13,000 psi on butt welds in shear and assumes that this may be carried back to the connected material inasmuch as in welded construction the full area of the connected material is available.

9. The three formulas 10, 14, and 18 for fillet and plug welds subjected to variable shearing stress are grouped in Fig. A-6 for comparison with the data.

The test data plotted on Fig. A-6 are derived and adapted from Reference 5. As the plotting of individual tests would overcrowd the figure, dimension lines are drawn to show the spread between the highest and lowest strength calculated for each Series, on each stress cycle. The Series numbers from Reference 5 and a brief description of each Series are shown on the figure.

As with previous figures, the lowest line, Formula 10, may be compared with the lowest set of data, that for 2,000,000 cycles, and so on up. In these three formulas the allowable fillet weld shear stress for near-static conditions is 12,400 psi. This bears the same relation to the 18,000 in tension found in bridge specifications as the familiar 13,600 bears to 20,000 on tension in specifications for buildings. Also, it gives a convenient figure of 1,100 lb per in., per ⅛ in. of size of fillet weld.

The work of Welding Research Council has indicated that the fatigue strength of slot welds is too low to justify the recognition of their use under the conditions here discussed.

10. It is not probable that any method of design that recognizes the phenomenon of fatigue can be set up to introduce less complication in the arithmetical work than the formulas contained in these Specifications. As, however, this type of specification is unusual in this country, certain typical examples will be worked out in what follows, to demonstrate the use of the formulas.

Ex. 1. Compression Member Connected by Butt Welding—Main Material.
Max. $= D + L + I = -640$ (kips). Min. $= D = +220$ (kips).
$l/r = 20$. $p = 15{,}000 - \frac{1}{4}(20)^2 = 14{,}900$.

$$A = \frac{640 + 110}{14.9} = 50.3 \text{ sq in. (by the AREA Spec.)}$$

$$= \frac{706 + 110}{14.9} = 54.7 \text{ sq in. (by the AASHO Spec., using } 0.70D \text{ in rev.)}$$

Ex. 2. Butt Weld for Splice or Connection of above Member.

a. Ry. Bridge, "short loading"; Formula 8. $\frac{640 + 220}{18} = 47.7$ sq in.

b. $\left\{\begin{array}{l}\text{Ry. Bridge, "long loading";}\\ \text{Hwy. Bridge, "short loading";}\end{array}\right\}$ Formula 12. $\frac{640 + 176}{18} = 45.3$ sq in.

c. Hwy Bridge, "long loading"; Formula 16. $\frac{640 + 110}{18} = 41.7$ sq in.

Therefore a full butt weld will, in all three cases, develop the main material of this member as designed by the general specifications. This circumstance is predictable without computations, from Fig. 5 and the discussion thereon.

Ex. 3. Tension Member Connected by Butt Welding—Main Material.
Max. $= D + L + I = +640$. Min. $= D = -220$.

$$A = \frac{640 + 110}{18} = 41.7 \text{ sq in. (by the AREA Spec.)}$$

$$= \frac{706 + 110}{18} = 45.3 \text{ sq in. (by the AASHO Spec.)}$$

Ex. 4. Butt Weld for Splice or Connection of above Member.

a. Ry. Bridge, "short loading"; Formula 7. $\frac{640 + 110}{13.5} = 55.6$ sq in.

b. $\left\{\begin{array}{l}\text{Ry. Bridge, "long loading";}\\ \text{Hwy. Bridge, "short loading";}\end{array}\right\}$ Formula 11. $\frac{640 + 110}{15.0} = 50.0$ sq in.

c. Hwy. Bridge, "long loading"; Formula 15. $\frac{640 + 110}{18.0} = 41.7$ sq in.

Therefore a full butt weld will, in Case *c*, develop the main material of this member as designed, but in Cases *a* and *b* the main section must be increased to equal the area of the required weld.

Ex. 5. Tension Member Connected by Fillet Welding—Main Material.
Max. $= D + L + I = +600$. $D = +180$.

a. Ry. Bridge, "short loading";

Formula 1. $\frac{600 - 120}{7.5} = 64.0$, or $\frac{600}{10} = 60.0$ sq in.

b. $\left\{\begin{array}{l}\text{Ry. Bridge, "long loading"}\\ \text{Hwy. Bridge, "short loading"}\end{array}\right\}$

Formula 3. $\frac{600 - 120}{10.5} = 45.7$, or $\frac{600}{14} = 42.8$ sq in.

c. Hwy. Bridge, "long loading";

Formula 5. $\frac{600 - 120}{15.0} = 32.0$, or $\frac{600}{18} = 36.4$ sq in.

Ex. 6. Fillet Weld for Connection of above Member

a. Ry Bridge;

Formula 10. $\frac{600 - 90}{7.2} = 70.8$, or $\frac{600}{12.4} = 48.5$ sq in.

b. $\left\{\begin{array}{l}\text{Ry. Bridge, "long loading";}\\ \text{Hwy. Bridge, "short loading"}\end{array}\right.$

Formula 14. $\frac{600 - 90}{10.0} = 51.0$, or $\frac{600}{12.4} = 48.5$ sq in.

c. Hwy. Bridge, "long loading";

Formula 18. $\frac{600 - 90}{12.0} = 42.5$, or $\frac{600}{12.4} = 48.5$ sq in.

E.g., for *a*, ½″ fillet = 0.354 throat × 200 lineal in. = 70.8 sq in.
for *b*, ⅜″ fillet = 0.265 throat × 192 lineal in. = 50.9 sq in.
for *c*, ⅜″ fillet = 0.265 throat × 184 lineal in. = 48.8 sq in.

Ex. 7. Member Governed by a Special Stress Combination. (Art. 212.)

$$\begin{aligned}\text{Max.} &= D + L + I = +420, \text{ or } = D + L + I + W + T = +600.\\ \text{Min.} &= D - L - I = -80, \text{ or } = D - L - I - W - T = -250.\end{aligned}$$

Assuming, as is usual, that the second combination takes a 25 per cent increase in allowable unit stress, the denominator of whatever formula is applicable is to be multiplied by 5/4, *e.g.*, if the applicable Formula is No. 7,

$$A = \frac{420 + 40}{13.5} = 34.1, \text{ or } \frac{600 + 130}{13.5} \times \frac{4}{5} = 43.2 \quad \text{Use } 43.2 \text{ sq in.}$$

Ex. 8. Connection of Girder Flange Plate to Web Plate.

Max. shear $= D + L + I = 220$ (kips). Min. $= D = 40$.
Web plate (say) $40 \times \frac{1}{2}$.

Assume Hwy. Bridge "short loading," Formula 14.

$$\left.\begin{aligned}&\frac{220 - 20}{10.0} = 20.0, \text{ or } \frac{220}{12.4} = 17.8. \text{ Use former*}\\ &\frac{20.0}{40} = 0.50 \text{ sq in.; } 2 \times \tfrac{3}{8} \times 0.707 = 0.53\end{aligned}\right\} \begin{aligned}&\text{Assuming that shear per lin in.}\\ &= \frac{\text{Total Shear}}{\text{Depth between welds}}\end{aligned}$$

Use two $\frac{3}{8}$-in. continuous fillet welds.

Ex. 9. Design of Fillet-Welded Compression Member Subject to Reversal of Axial Stress and to Non-Reversing Bending. Consider only Hwy. Bridge, "short loading."

Axial, Max. $= D + L + I = -90$ (kips). Min. $= 0.7D - L - I = +60$.
B. M. Max. $=$ Min. $= D = 10$ kip ft.

Assume a make-up, calculate l/r and I/c.

$$\text{Say}\begin{cases} l/r = 60;\ p = 15{,}000 - \tfrac{1}{4}(60)^2 = 14{,}100.\\ I/c = 100;\ \dfrac{10 \times 12}{100} = 1{,}200 \text{ psi bending.}\end{cases}$$

$$\text{Formula 4: Req. } A = \frac{90 + 40}{10.5 - 1.2} = 14.0 \text{ or } \left(\frac{90 + 30}{14.1 - 1.2} = 9.3\right) \text{ sq in.}$$

Revise the make-up if necessary to obtain the area of 14.0 and I/c of 100.

$$\text{Actual extreme fiber stress Max.} = \frac{90}{14.0} + 1.2 = 7{,}600 \text{ psi.}$$

* A draftsman will soon recognize that if Min. is less than about one-half (0.53) Max., the first of the two cases governs, and vice versa. Thus the double calculation is virtually never necessary to an experienced man. A similar criterion is easily found for each of the formulas.

If the bending moment is inconsiderable at the point of connection by fillet welding, then

$$\text{Req. } A = \frac{90 + 40}{10.5} = 12.4 \text{ or } \left(\frac{90 + 30}{14.1} = 8.5\right) \text{ sq in.}$$

$$\text{Actual extreme fiber stress Max.} = \frac{90}{12.4} + 1.2 = 8{,}450 \text{ psi.}$$

REFERENCES—BIBLIOGRAPHY

References in this Appendix are to the following Reports of the Committee on Fatigue Testing (Structural) of the Welding Research Council of the Engineering Foundation, and will be found in the issues noted of the Welding Research Supplement to the *Welding Journal* of the AMERICAN WELDING SOCIETY.

Reference 1. Effect of Periods of Rest on the Fatigue Strength of Welded Joints. (September 1946.)

Reference 2. Calculation and Graphical Representation of the Fatigue Strength of Structural Joints. (February 1942.)

Reference 3. Fatigue Strength of Butt Welds in Ordinary Bridge Steel. (May 1943.)

Reference 4. Do., Maximum Stress Compressive. (January 1945.)

Reference 5. Fatigue Strength of Fillet, Plug and Slot Welds in Ordinary Bridge Steel. (July 1945.)

These reports in turn contain references to more complete data to be found in various Bulletins of the Engineering Experiment Station, University of Illinois.

A digest and bibliography covering the foregoing and other weld fatigue research reports, many of which apply to structural joints, will be found in the following Supplements to the *Welding Journal*, AMERICAN WELDING SOCIETY.

January 1937: "Fatigue Strength of Welded Joints—A Review of the Literature to October 1, 1936," by W. Spraragen and G. E. Clausen.

September 1942: "Fatigue Strength of Welded Joints—A Review of the Literature from October 1936 to September 1941," by W. Spraragen and D. Rosenthal.

APPENDIX D

AMERICAN WELDING SOCIETY

STANDARD WELDING SYMBOLS, 1947

Pages 1 to 26

STANDARD WELDING SYMBOLS

SECTION I—BASIC SYMBOLS

101. Arc and Gas Weld Symbols

Arc and gas weld symbols shall be as shown in Fig. 1.

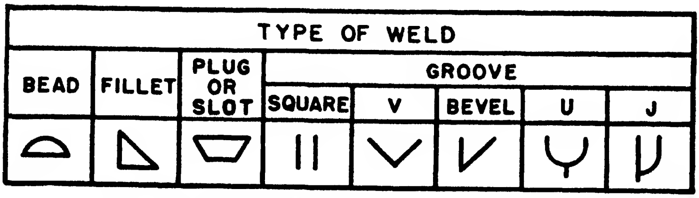


Fig. 1—Basic Arc and Gas Weld Symbols

102. Resistance Weld Symbols

Resistance weld symbols shall be as shown in Fig. 2.

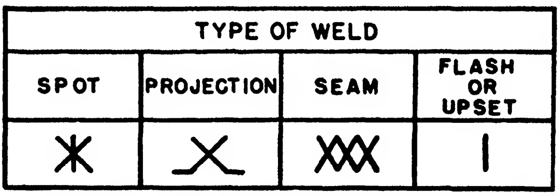


Fig. 2—Basic Resistance Weld Symbols

103. Brazing, Forge, Thermit, Induction and Flow Welding Symbols

Brazing, forge, thermit, induction and flow welding shall be indicated by using a process or specification reference in the tail of the welding symbol. (See Article 305 and Fig. 40.)

104. Supplementary Symbols

Supplementary symbols to be used in connection with weld symbols shall be as shown in Fig. 3.

WELD ALL AROUND	FIELD WELD	CONTOUR	
		FLUSH	CONVEX
○	●	—	⌒

Fig. 3—Supplementary Symbols

105. Elements of a Welding Symbol

This standard makes a distinction between the terms *weld symbol* and *welding symbol*. The *weld symbol* is the ideograph used to indicate the desired type of weld. The assembled *welding symbol* consists of the following eight elements, or such of these elements as are necessary:

Reference line.
Arrow.
Basic weld symbols.
Dimensions and other data.
Supplementary symbols.
Finish symbols.
Tail.
Specification, process, or other references.

106. Standard Location of Elements of a Welding Symbol

The elements of a welding symbol shall have standard locations with respect to each other as shown in Fig. 4.

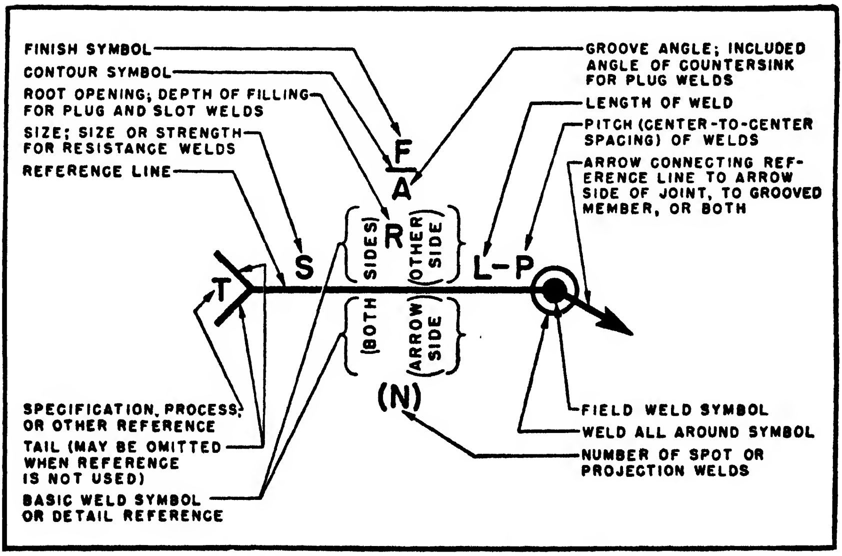


Fig. 4—Standard Location of Elements of a Welding Symbol

SECTION II—BASIC TYPES OF JOINTS AND WELDS

201. Basic Types of Joints

The basic types of joints are shown in Fig. 5 and the types of welds by which the members are frequently joined are listed thereon.

202. Basic Types of Welds

The basic types of welds indicated by the basic weld symbols are illustrated in Figs. 6 to 12 and 15 to 19, inclusive.

SECTION III—GENERAL PROVISIONS

301. Location Significance of Arrow

(*a*) In the case of groove, fillet, and flash or upset welding symbols, the arrow shall connect the welding symbol reference line to one side of the joint, and this side shall be considered the *arrow side* of the joint. The side opposite the arrow side of the joint shall be considered the *other side* of the joint. (See Figs. 6 to 12 inclusive and Fig. 19.)

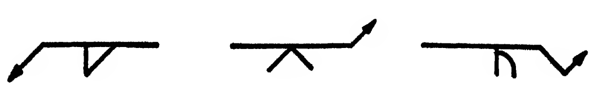
(*b*) In the case of plug, slot, spot, seam and projection welding symbols, the arrow shall connect the welding symbol reference line to the outer surface of one of the members of the joint at the center line of the desired weld. The member to which the arrow points shall be considered the *arrow-side* member. The other member of the joint shall be considered the *other-side* member. (See Figs. 15 to 18, inclusive.)

(*c*) When a joint is depicted by a single line on the drawing and the arrow of a welding symbol is directed to this line, the arrow side of the joint shall be considered as the near side of the joint in accordance with the usual conventions of drafting. (See Figs. 6 to 12, inclusive and Fig. 19.)

(*d*) When a joint is depicted as an area parallel to the plane of projection in a drawing and the arrow of a welding symbol is directed to that area, the *arrow-side* member of the joint shall be considered as the near member of the joint in accordance with the usual conventions of drafting. (See Figs. 15 to 18, inclusive.)

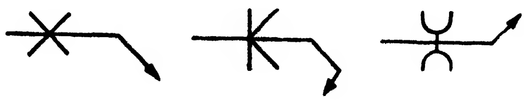
302. Location of Weld with Respect to Joint

(*a*) Welds on the arrow side of the joint shall be shown by placing the weld symbol on the side of the reference line toward the reader, thus: (See also Figs. 6*A*, 8*A*, 9*A*, 10*A*, 11*A*, 12*A*, 15*A*, 16*A* and 18*A*.)

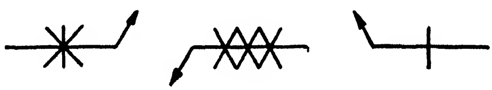


(*b*) Welds on the other side of the joint shall be shown by placing the weld symbol on the side of the reference line away from the reader, thus: (See also Figs. 6*B*, 8*B*, 9*B*, 10*B*, 11*B*, 12*B*, 15*B*, 16*B* and 18*B*.)

(*c*) Welds on both sides of the joint shall be shown by placing weld symbols on both sides of the reference line, toward and away from the reader, thus: (See also Figs. 7, 8*C*, 9*C*, 10*C*, 11*C* and 12*C*.)



(*d*) Spot, seam, flash and upset weld symbols have no arrow-side or other-side significance in themselves, although supplementary symbols used in conjunction therewith may have such significance. (See Articles 301, 906, 1007 and 1202.) Spot, seam, flash and upset weld symbols shall be centered on the reference line, thus: (See also Figs. 17 and 19.)



303. Method of Drawing Symbols

Symbols may be drawn mechanically or freehand, as desired.

304. Use of Inch, Degree and Pound Marks

Inch, degree and pound marks may be used on welding symbols or not, as desired, except that inch marks shall be used for indicating the diameter of spot and projection welds and the width of seam welds, when such welds are specified by linear dimension.

305. Location of Specification, Process or Other References

When a specification, process or other references is used with a welding symbol, the reference shall be placed in the tail, thus:

306. Use of Symbols Without References

When desired, symbols may be used without specification, process or other references in the following instances:

(a) When a note such as the following appears on the drawing: "Unless otherwise designated, all welds are to be made in accordance with Specification No. —."

(b) When the welding procedure to be used is prescribed elsewhere.

307. Use of General Notes

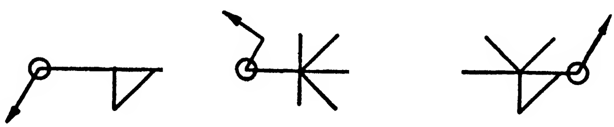
When desired, general notes such as the following may be placed on a drawing to provide detailed information pertaining to the predominating welds, and this information need not be repeated on the symbols.

"Unless otherwise indicated, all fillet welds are $^5/_{16}$ inch size."

"Unless otherwise indicated, root openings for all groove welds are $^3/_{16}$ inch.

308. Use of Weld-All-Around Symbol

Welds extending completely around a joint shall be indicated by means of the weld-all-around symbol, thus:



309. Use of Field Weld Symbol

Field welds (welds not made in a shop or at the place of initial construction) shall be indicated by means of the field weld symbol, thus:

310. Extent of Welding Denoted by Symbols

Symbols apply between abrupt changes in the direction of the welding or to the extent of hatching or dimension lines, except when the weld-all-around symbol is used. (See Figs. 23, 24, 35*A*, 36*C* and 38*A*.)

311. Weld Proportions

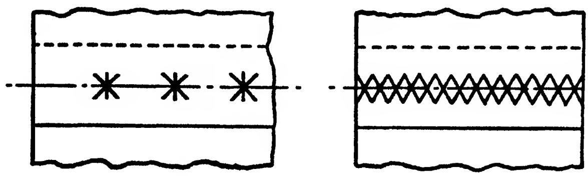
All welds shall be continuous and of user's standard proportions unless otherwise indicated.

312. Finishing of Welds

Finishing of welds, other than cleaning, shall be indicated by suitable contour and finish symbols. (See Arts. 409, 505, 603, 706, 804 and 1202.)

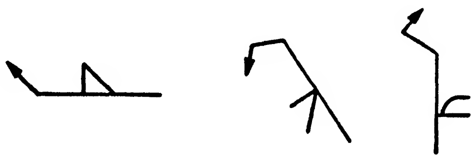
313. Location of Weld Symbols

(*a*) Weld symbols, except spot and seam, shall be shown only on the welding symbol reference line and not on the lines of the drawing.
(*b*) Spot and seam weld symbols may be placed directly on drawings at the locations of the desired welds, thus: (See also Fig. 17.)



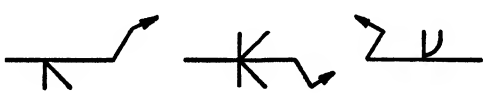
314. Construction of Fillet and Bevel- and J-Groove Welding Symbols

Fillet and bevel- and J-groove weld symbols shall be shown with the perpendicular leg *always* to the left, thus: (See also Figs. 6, 7, 9 and 11.)



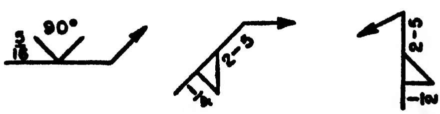
315. Use of Break in Arrow of Bevel- and J-Groove Welding Symbols

When a bevel- or J-groove weld symbol is used, the arrow shall point with a definite break toward the member which is to be chamfered, thus: (See also Figs. 9 and 11.) (In cases where the member to be chamfered is obvious, the break in the arrow may be omitted.)



316. Reading of Information on Welding Symbols

Information on welding symbols shall be placed to read from left to right along the reference line in accordance with the usual conventions of drafting, thus:

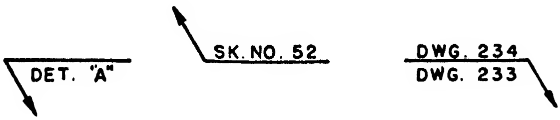


317. Combined Weld Symbols

For joints having more than one weld, a symbol shall be shown for each weld, thus: (See also Figs. 20 and 27.)

318. Designation of Special Types of Welds

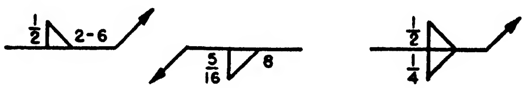
When the basic weld symbols are inadequate to indicate the desired weld, the weld shall be shown by a cross section, detail or other data, with a reference thereto on the welding symbol, observing the usual location significance, thus:



SECTION IV—FILLET WELDS

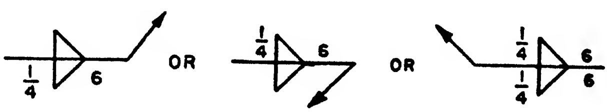
401. General

(*a*) Dimensions of fillet welds shall be shown on the same side of the reference line as the weld symbol, thus: (See also Figs. 21, 22 and 23.)

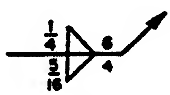


(*b*) When no general note governing the dimensions of fillet welds appears on the drawing, the dimensions of fillet welds on both sides of the joint shall be shown as follows:

(1) When both welds have the same dimensions, one or both may be dimensioned, thus:

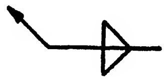


(2) When the welds differ in dimensions, both shall be dimensioned. thus:

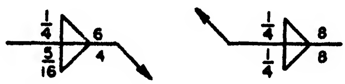


(c) When there appears on the drawing a general note governing the dimensions of fillet welds, such as "All fillet welds $^5/_{16}$ in. size unless otherwise noted," the dimensions of fillet welds on both sides of the joint shall be indicated as follows:

(1) When both welds have dimensions governed by the note, neither need be dimensioned, thus:

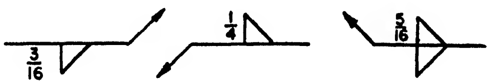


(2) When the dimensions of one or both welds differ from the dimensions given in the general note, both welds shall be dimensioned, thus:

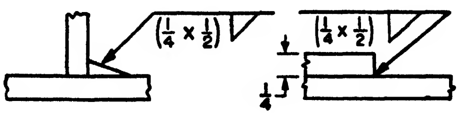


402. Size of Fillet Welds

(a) The size of a fillet weld shall be shown to the left of the weld symbol, thus: (See also Figs. 21*A*, 21*B*, 21*C* and 21*D*.)

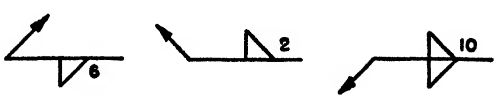


(b) The size of a fillet weld with unequal legs, shall be shown in parentheses to the left of the weld symbol, as shown below. Weld orientation is not shown by the symbol and shall be shown on the drawing when necessary. (See also Fig. 21*D*.)



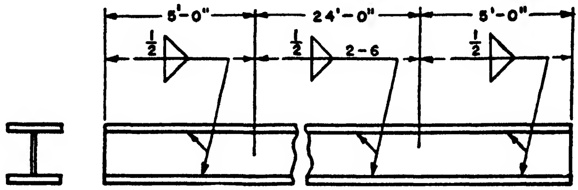
403. Length of Fillet Welds

(a) The length of a fillet weld, when indicated on the welding symbol, shall be shown to the right of the weld symbol, thus: (See also Fig. 21*F*.)



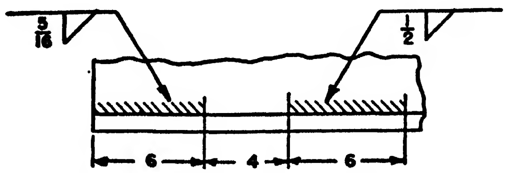
(*b*) When fillet welding extends for the full distance between abrupt changes in the direction of the welding (see Article 310), no length dimension need be shown on the welding symbol. (See Fig. 21*E*.)

(*c*) Specific lengths of fillet welding may be indicated by symbols in conjunction with dimension lines, thus: (See also Figs. 23*A* and 23*B*.)



404. Extent of Fillet Welding

(*a*) When it is desired to show the extent of fillet welding graphically, one type of hatching with definite end lines shall be used, thus:

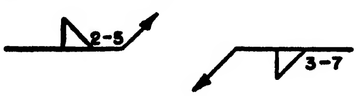


(*b*) Fillet welding extending beyond abrupt changes in the direction of the welding shall be indicated by means of additional arrows pointing to each section of the joint to be welded, as shown in Fig. 24*A*, except when the weld-all-around symbol is used.

405. Dimensioning of Intermittent Fillet Welding

(*a*) The pitch (center-to-center spacing) of intermittent fillet welding shall be shown as the distance between centers of increments on *one* side of the joint. (See Fig. 22.)

(*b*) The pitch (center-to-center spacing) of intermittent fillet welding shall be shown to the right of the length dimension, thus: (See also Fig. 22.)



(*c*) Chain intermittent fillet welding shall be shown thus: (See also Fig. 22*B*.)

(*d*) Staggered intermittent fillet welding shall be shown thus: (See also Fig. 22*C*.)

406. Termination of Intermittent Fillet Welding

(*a*) When intermittent fillet welding is used by itself, the symbol indicates that increments shall be located at the ends of the dimensioned length. (See Fig. 22.)

(*b*) When intermittent fillet welding is used between continuous fillet welding the symbol indicates that spaces equal to the pitch minus the length of one increment shall be left at the ends of the dimensioned length. (See Fig. 23*A*.)

407. Combination of Intermittent and Continuous Fillet Welding

Separate symbols shall be used for intermittent and continuous fillet welding when the two are used in combination. (See Fig. 23*A*.)

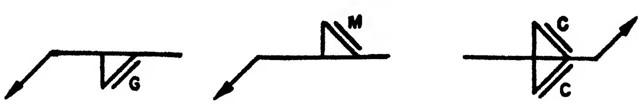
408. Fillet Welds in Holes and Slots

Fillet welds in holes and slots shall be shown by means of fillet weld symbols.

409. Surface Contour of Fillet Welds

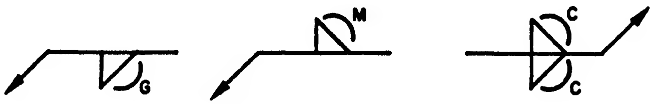
(*a*) Fillet welds that are to be welded approximately flat-faced without recourse to any method of finishing shall be shown by adding the flush-contour symbol to the weld symbol, observing the usual location significance, thus:

(*b*) Fillet welds that are to be made flat-faced by mechanical means, shall be shown by adding both the flush-contour symbol and the user's standard finish symbol* to the weld symbol, observing the usual location significance, thus:



* Finish symbols used herein indicate the method of finishing ("C" = chipping; "G" = grinding; "M" = machining) and not the degree of finish.

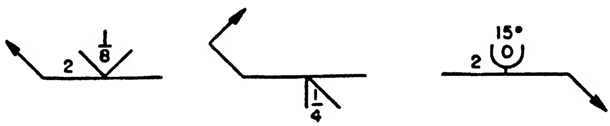
(*c*) Fillet welds that are to be mechanically finished to a convex contour, shall be shown by adding both the convex-contour symbol and the user's standard finish symbol* to the weld symbol, observing the usual location significance, thus:



SECTION V—GROOVE WELDS

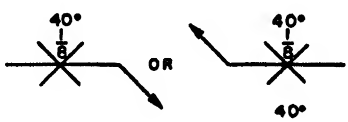
501. General

(*a*) Dimensions of groove welds shall be shown on the same side of the reference line as the weld symbol, thus: (See also Fig. 31*B*.)

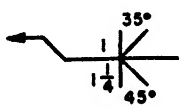


(*b*) When no general note governing the dimensions of groove welds appears on the drawing, the dimensions of double-groove welds shall be shown as follows:

(1) When both welds have the same dimensions, one or both may be dimensioned, thus:



(2) When the welds differ in dimensions, both shall be dimensioned, thus:

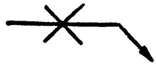


(*c*) When there appears on the drawing a general note governing the dimensions of groove welds, such as "All V-groove welds shall have a 60° groove angle unless otherwise noted," the dimensions of double-groove welds shall be indicated as follows:

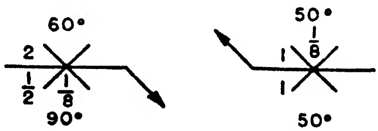
(1) When both welds have dimensions governed by the note, neither

* Finish symbols used herein indicate the method of finishing ("C" = chipping; "G" = grinding; "M" = machining) and not the degree of finish.

need be dimensioned, thus:

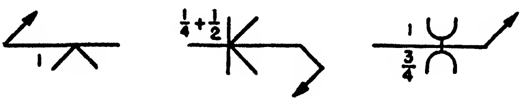


(2) When the dimensions of one or both welds differ from the dimensions given in the general note, both welds shall be dimensioned, thus:



502. Size of Groove Welds

(*a*) The size of groove welds shall be shown to the left of the weld symbol, thus: (See also Figs. 25, 26 and 27.)

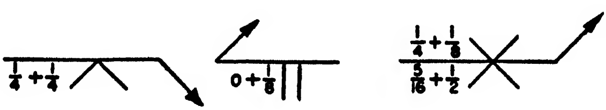


(*b*) The size of groove welds with no specified root penetration shall be shown as follows:

(1) The size of single-groove and symmetrical double-groove welds which extend completely through the member or members being joined, need not be shown on the welding symbol. (See Figs. 25*D* and 25*E*.)

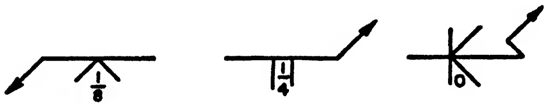
(2) The size of groove welds which extend only partly through the member or members being joined, shall be shown on the welding symbol. (See Figs. 25*A*, 25*C* and 25*F*.)

(*c*) The size of groove welds with specified root penetration shall be indicated by showing both the depth of chamfering and the root penetration, separated by a plus mark and placed to the left of the weld symbol. The depth of chamfering and the root penetration shall read in that order from left to right along the reference line, thus: (See also Figs. 26 and 27.)

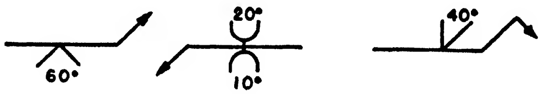


503. Groove Dimensions

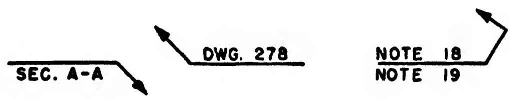
(*a*) Root opening of groove welds shall be the user's standard unless otherwise indicated. Root opening of groove welds, when not the user's standard, shall be shown inside the weld symbol, thus: (See also Fig. 28.)



(*b*) Groove angle of groove welds shall be the user's standard, unless otherwise indicated. Groove angle of groove welds, when not the user's standard, shall be shown thus: (See also Fig. 29.)



(*c*) Groove radii and root faces of U- and J-groove welds shall be the user's standard unless otherwise indicated. When groove radii and root faces of U- and J-groove welds are not the user's standard, the weld shall be shown by a cross section, detail or other data, with a reference thereto on the welding symbol, observing the usual location significance, thus:

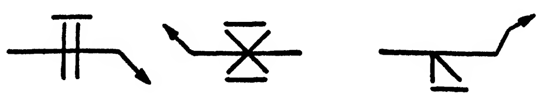


504. Designation of Back and Backing Welds

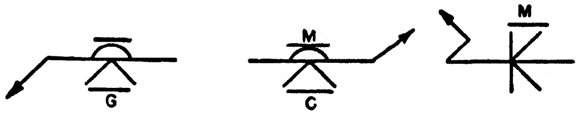
Bead-type back and backing welds of single-groove welds shall be shown by means of the bead weld symbol. (See Art. 602.)

505. Surface Contour of Groove Welds

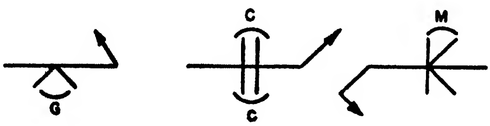
(*a*) Groove welds that are to be welded approximately flush without recourse to any method of finishing shall be shown by adding the flush-contour symbol to the weld symbol, observing the usual location significance, thus: (See also Fig. 30*A*.)



(*b*) Groove welds that are to be made flush by mechanical means shall be shown by adding both the flush-contour symbol and the user's standard finish symbol* to the weld symbol, observing the usual location significance, thus: (See also Fig. 30*B*.)



(*c*) Groove welds that are to be mechanically finished to a convex contour shall be shown by adding both the convex-contour symbol and the user's standard finish symbol* to the weld symbol, observing the usual location significance, thus: (See also Fig. 30*C*.)



SECTION VI—BEAD WELDS

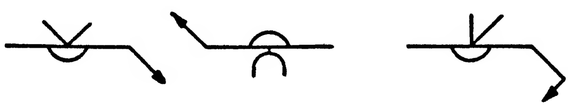
601. General

(*a*) The single bead weld symbol shall be used to indicate bead-type back or backing welds of single-groove welds. (See Fig. 13.)

(*b*) The dual bead weld symbol shall be used to indicate surfaces built up by welding. (See Fig. 14.)

602. Use of Bead Weld Symbol to Indicate Bead-Type Back or Backing Welds

(*a*) Bead welds used as back or backing welds of single-groove welds, shall be shown by placing a single bead weld symbol on the side of the reference line opposite the groove weld symbol, thus: (See also Figs. 13, 20*A* and 20*B*.)

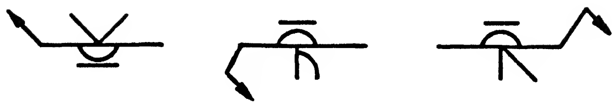


(*b*) Dimensions of bead welds used as back or backing welds shall not be shown on the welding symbol. If it is desired to specify these dimensions, they shall be shown on the drawing.

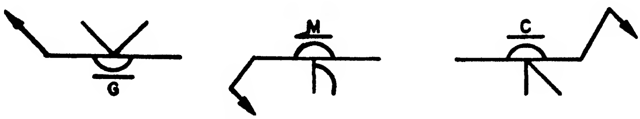
603. Surface Contour of Back or Backing Welds

(*a*) Back or backing welds that are to be welded approximately flush without recourse to any method of finishing shall be shown by adding the flush-contour symbol to the bead weld symbol, thus:

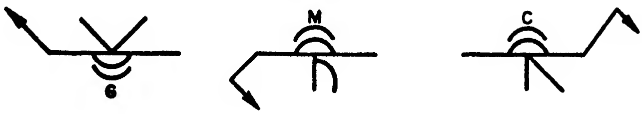
* Finish symbols used herein indicate the method of finishing ("*C*" = chipping; "*G*" = grinding: "*M*" = machining) and not the degree of finish.



(*b*) Back or backing welds that are to be made flush by mechanical means shall be shown by adding both the flush-contour symbol and the user's standard finish symbol* to the bead weld symbol, thus:



(*c*) Back or backing welds that are to be mechanically finished to a convex contour shall be shown by adding both the convex-contour symbol and the user's standard finish symbol* to the bead weld symbol, thus:



604. Use of Bead Weld Symbol to Indicate Surfaces Built Up by Welding

(*a*) Surfaces built up by welding, whether by single- or multiple-pass bead welds, shall be shown by the dual bead weld symbol, thus: (See also Fig. 14.)

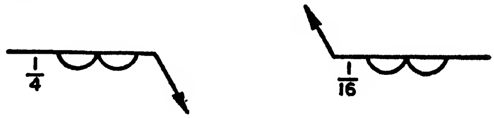
(*b*) The dual bead weld symbol does not indicate the welding of a joint, and hence has no arrow- or other-side significance. This symbol shall be drawn on the side of the reference line toward the reader and the arrow shall point clearly to the surface on which the weld is to be deposited. (See Fig. 14.)

(*c*) Dimensions used in conjunction with the dual bead weld symbol shall be shown on the same side of the reference line as the weld symbol, thus: (See also Fig. 14.)

* Finish symbols used herein indicate the method of finishing ("C" = chipping; "G" = grinding; "M" = machining) and not the degree of finish.

605. Size (Height) of Surfaces Built Up by Welding

(*a*) The size of a surface built up by welding shall be indicated by showing the minimum height of the weld deposit to the left of the weld symbol, thus: (See also Fig. 14*A*.)



(*b*) When no specific height of weld deposit is desired, no size dimension need be shown on the welding symbol.

606. Extent, Location and Orientation of Surfaces Built Up by Welding

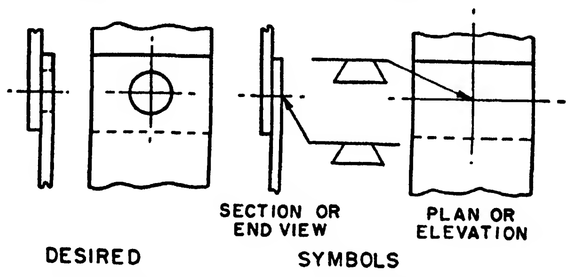
(*a*) When the entire area of a plane or curved surface is to be built up by welding, no dimension other than size (height of deposit) need be shown on the welding symbol. (See Fig. 14*C*.)

(*b*) When a portion of the area of a plane or curved surface is to be built up by welding, the extent, location and orientation of the area to be built up shall be indicated on the drawing. (See Fig. 14*D*.)

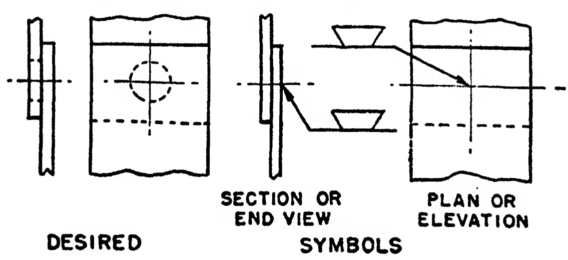
SECTION VII—PLUG WELDS

701. General

(*a*) Holes in the arrow-side member of a joint for plug welding shall be indicated by placing the weld symbol on the side of the reference line toward the reader, thus: (See also Fig. 15A.)



(*b*) Holes in the other-side member of a joint for plug welding shall be indicated by placing the weld symbol on the side of the reference line away from the reader, thus: (See also Fig. 15B.)



(*c*) Dimensions of plug welds shall be shown on the same side of the reference line as the weld symbol, thus: (See also Fig. 32.)

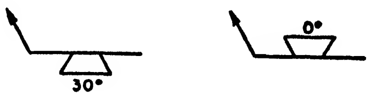
(*d*) The plug weld symbol shall not be used to designate fillet welds in holes. (See Article 408.)

702. Size of Plug Welds

The size of a plug weld shall be shown to the left of the weld symbol thus: (See also Fig. 32*A*.)

703. Angle of Countersink

(*a*) Included angle of countersink of plug welds shall be the user's standard unless otherwise indicated. Included angle of countersink, when not the user's standard, shall be shown thus: (See also Fig. 32*B*.)



704. Depth of Filling

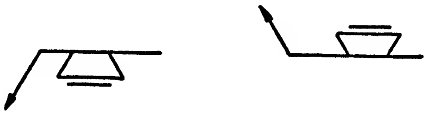
(*a*) Depth of filling of plug welds shall be complete unless otherwise indicated. When the depth of filling is less than complete, the depth of filling, in inches, shall be shown inside the weld symbol, thus: (See also Fig. 32*C*.)

705. Spacing of Plug Welds

Pitch (center-to-center spacing) of plug welds shall be shown to the right of the weld symbol, thus: (See also Fig. 32*D*.)

706. Surface Contour of Plug Welds

(*a*) Plug welds that are to be welded approximately flush without recourse to any method of finishing shall be shown by adding the flush-contour symbol to the weld symbol, thus:

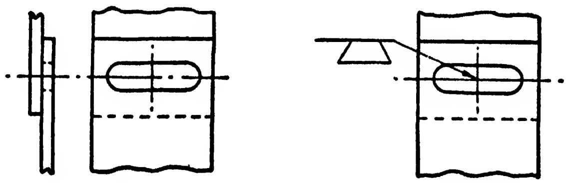


(*b*) Plug welds that are to be made flush by mechanical means shall be shown by adding both the flush-contour symbol and the user's standard finish symbol* to the weld symbol, thus:

SECTION VIII—SLOT WELDS

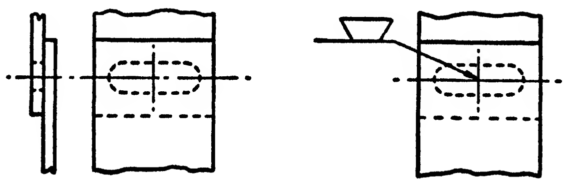
801. General

(*a*) Slots in the arrow-side member of a joint for slot welding shall be indicated by placing the weld symbol on the side of the reference line toward the reader, thus: (See also Fig. 16*A*.)



DESIRED SYMBOL

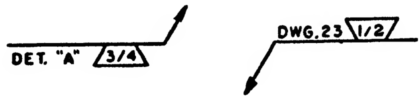
(*b*) Slots in the other-side member of a joint for slot welding shall be indicated by placing the weld symbol on the side of the reference line away from the reader, thus: (See also Fig. 16*B*.)



DESIRED SYMBOL

* Finish symbols used herein indicate the method of finishing ("C" = chipping; "G" = grinding; "M" = machining) and not the degree of finish.

(*c*) Dimensions of slot welds shall be shown on the same side of the reference line as the weld symbol, thus: (See also Fig. 33.)



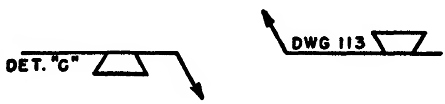
(*d*) The slot weld symbol shall not be used to designate fillet welds in slots. (See Article 408.)

802. Depth of Filling

(*a*) Depth of filling of slot welds shall be complete unless otherwise indicated. When the depth of filling is less than complete, the depth of filling, in inches, shall be shown inside the weld symbol, thus: (See also Fig. 33*B*.)

803. Details of Slot Welds

Length, width, spacing, included angle of countersink, orientation and location of slot welds cannot be shown on the welding symbol. These data shall be shown on the drawing or by a detail with a reference thereto on the welding symbol, observing the usual location significance thus: (See also Fig. 33.)



804. Surface Contour of Slot Welds

(*a*) Slot welds that are to be welded approximately flush without recourse to any method of finishing shall be shown by adding the flush-contour symbol to the weld symbol, thus:

(*b*) Slot welds that are to be made flush by mechanical means shall be shown by adding both the flush-contour symbol and the user's standard finish symbol* to the weld symbol, thus:

* Finish symbols used herein indicate the method of finishing ("*C*" = chipping; "*G*" = grinding; "*M*" = machining) and not the degree of finish.

SECTION IX—SPOT WELDS

901. General

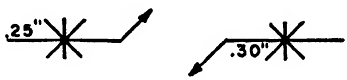
(*a*) Spot weld symbols have no arrow- or other-side significance in themselves, although supplementary symbols used in conjunction therewith may have such significance (See Article 906.) Spot weld symbols shall be centered on the reference line. (See Fig. 17*A*.)

(*b*) Dimensions of spot welds may be shown on either side of the reference line.

902. Size of Spot Welds

Spot welds shall be dimensioned by either size or strength, as follows:

(1) The size of spot welds shall be designated as the diameter of the weld expressed decimally in hundredths of an inch, and shall be shown, with inch marks, to the left of the weld symbol, thus: (See also Fig. 34*A*.)



(2) The strength of spot welds shall be designated as the minimum acceptable shear strength in pounds per spot, and shall be shown to the left of the weld symbol, thus: (See also Fig. 34*B*.)

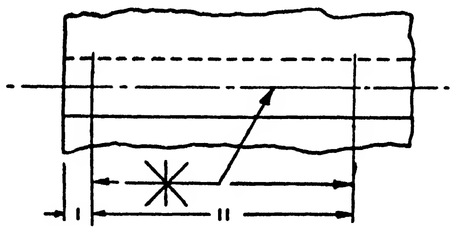
903. Spacing of Spot Welds

(*a*) The pitch (center-to-center spacing) of spot welds shall be shown to the right of the weld symbol, thus: (See also Fig. 34*C*.)

(*b*) When spot weld symbols are shown directly on the drawing, spacing shall be shown by dimensions. (See Fig. 34*D*.)

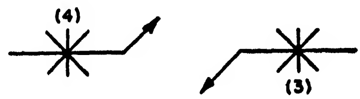
904. Extent of Spot Welding

When spot welding extends less than the distance between abrupt changes in the direction of the welding, or less than the full length of the joint (see Article 310), the extent shall be dimensioned, thus: (See also Fig. 35*A*.)



905. Number of Spot Welds

When a definite number of spot welds is desired in a certain joint, the number shall be shown in parentheses either above or below the weld symbol, thus: (See also Fig. 35*B*.)



906. Flush Spot-Welded Joints

When the exposed surface of one member of a spot-welded joint is to be flush, that surface shall be indicated by adding the flush-contour symbol to the weld symbol, observing the usual location significance, thus:

SECTION X—SEAM WELDS

1001. General

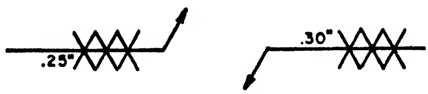
(*a*) Seam weld symbols have no arrow- or other-side significance in themselves, although supplementary symbols used in conjunction therewith may have such significance. (See Article 1007.) Seam weld symbols shall be centered on the reference line. (See Fig. 17 *B*.)

(*b*) Dimensions of seam welds may be shown on either side of the reference line.

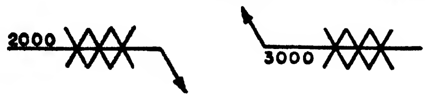
1002. Size of Seam Welds

Seam welds shall be dimensioned by either size or strength as follows:

(1) The size of seam welds shall be designated as the width of the weld expressed decimally in hundredths of an inch, and shall be shown, with inch marks, to the left of the weld symbol, thus: (See also Fig. 36*A*.)

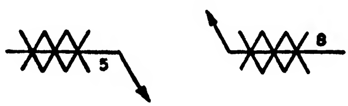


(2) The strength of seam welds shall be designated as the minimum acceptable shear strength in pounds per linear inch, and shall be shown to the left of the weld symbol, thus: (See also Fig. 36*B*.)



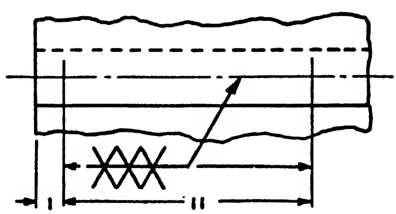
1003. Length of Seam Welds

(*a*) The length of a seam weld, when indicated on the welding symbol, shall be shown to the right of the weld symbol, thus: (See also Fig. 36*A*.)



(*b*) When seam welding extends for the full distance between abrupt changes in the direction of the welding (see Article 310), no length dimension need be shown on the welding symbol.

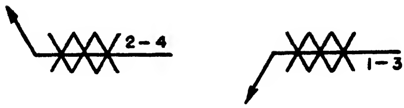
(*c*) When seam welding extends less than the distance between abrupt changes in the direction of the welding, or less than the full length of the joint (see Article 310), the extent shall be dimensioned, thus: (See also Fig. 36*C*.)



1004. Dimensioning of Intermittent Seam Welding

(*a*) The pitch (center-to-center spacing) of intermittent seam welding shall be shown as the distance between centers of the weld increments.

(*b*) The pitch (center-to-center spacing) of intermittent seam welding shall be shown to the right of the length dimension, thus: (See also Fig. 36*A*.)



1005. Termination of Intermittent Seam Welding

(*a*) When intermittent seam welding is used by itself, the symbol indicates that increments shall be located at the ends of the dimensioned length. (See Fig. 36*A*.)

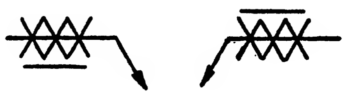
(*b*) When intermittent seam welding is used between continuous seam welding, the symbol indicates that spaces equal to the pitch minus the length of one increment shall be left at the ends of the dimensioned length.

1006. Combination of Intermittent and Continuous Seam Welding

Separate symbols shall be used for intermittent and continuous seam welding when the two are used in combination.

1007. Flush Seam-Welded Joints

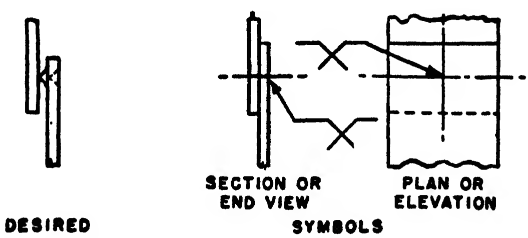
When the exposed surface of one member of a seam-welded joint is to be flush, that surface shall be indicated by adding the flush-contour symbol to the weld symbol, observing the usual location significance, thus:



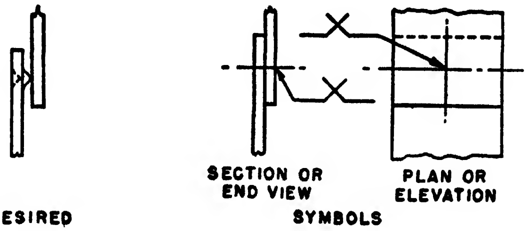
SECTION XI—PROJECTION WELDS

1101. General

(*a*) Embossments on the arrow-side member of a joint for projection welding shall be indicated by placing the weld symbol on the side of the reference line toward the reader, thus: (See also Fig. 18*A*.)

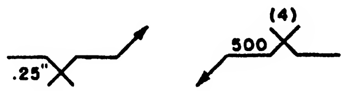


(*b*) Embossments on the other-side member of a joint for projection welding shall be indicated by placing the weld symbol on the side of the reference line away from the reader, thus: (See also Fig. 18*B*.)



(*c*) Proportions of projections shall be shown by a detail or other suitable means.

(*d*) Dimensions of projection welds shall be shown on the same side of the reference line as the weld symbol, thus: (See also Fig. 38*C*.)



1102. Size of Projection Welds

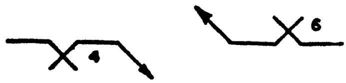
Projection welds shall be dimensioned by either size or strength, as follows:

(1) The size of projection welds shall be designated as the diameter of the weld expressed decimally in hundredths of an inch, and shall be shown, with inch marks, to the left of the weld symbol, thus: (See also Fig. 37*A*.)

(2) The strength of projection welds shall be designated as the minimum acceptable shear strength in pounds per weld, and shall be shown to the left of the weld symbol, thus: (See also Fig. 37*B*.)

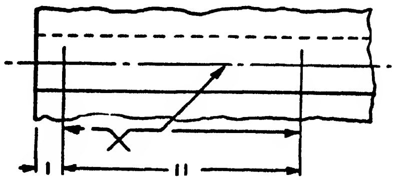
1103. Spacing of Projection Welds

The pitch (center-to-center spacing) of projection welds shall be shown to the right of the weld symbol, thus: (See also Fig. 37*C*.)



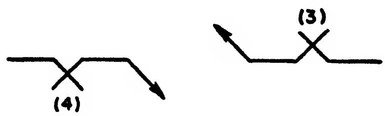
1104. Extent of Projection Welding

When projection welding extends less than the distance between abrupt changes in the direction of the welding, or less than the full length of the joint (see Article 310), the extent shall be dimensioned, thus: (See also Fig. 38*A*.)



1105. Number of Projection Welds

When a definite number of projection welds is desired in a certain joint, the number shall be shown in parentheses, thus: (See also Fig. 38*B*.)



1106. Flush Projection-Welded Joints

When the exposed surface of one member of a projection welded joint is to be made flush, that surface shall be indicated by adding the flush-contour symbol to the weld symbol, observing the usual location significance, thus:

SECTION XII—FLASH AND UPSET WELDS

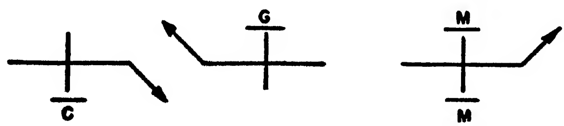
1201. General

(*a*) Flash and upset weld symbols have no arrow-side or other-side significance in themselves although supplementary symbols used in conjunction therewith may have such significance (see Article 1202). Flash or upset weld symbols shall be centered in the reference line. (See Fig. 19.)

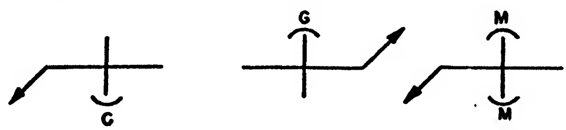
(*b*) Dimensions of flash and upset welds shall not be shown on the welding symbol.

1202. Surface Contour of Flash and Upset Welds

(*a*) Flash and upset welds that are to be made flush by mechanical means shall be shown by adding both the flush-contour symbol and the user's standard finish symbol* to the weld symbol, observing the usual location significance, thus: (See also Fig. 39*A*.)



(*b*) Flash and upset welds that are to be mechanically finished to a convex contour shall be shown by adding both the convex-contour symbol and the user's standard finish symbol* to the weld symbol, observing the usual location significance, thus: (See also Fig. 39*B*.)



* Finish symbols used herein indicate the method of finishing ("*C*" = chipping "*G*" = grinding; "*M*" = machining) and not the degree of finish.

APPENDIX E

SPECIFICATIONS FOR STRUCTURES OF ALUMINUM ALLOY 2014-T6

Pages 1-32 of Paper 971, Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Third Progress Report of the Committee of the Structural Division on Design in Lightweight Structural Alloys.

Foreword

These specifications are a revised version of "Specifications for Heavy Duty Structures of High-Strength Aluminum Alloy," published as *Proceedings*, ASCE, Separate No. 22, June, 1950, and as Paper No. 2532, *Transactions*, ASCE, Vol. 117, 1952, p. 1253. Publication of these specifications as a progress report of the Committee on Design in Lightweight Structural Alloys has been approved by the Executive Committee of the Structural Division.

Synopsis

These specifications cover allowable stresses, design rules, and fabrication procedures for riveted heavy duty structures built of the high-strength aluminum alloy known commercially as 2014-T6 (formerly 14S-T6). The basic allowable tensile working stress is 22 kips per sq in. based on a minimum yield strength of 53 kips per sq in. and a minimum tensile strength of 60 kips per sq in.

PART I. GENERAL

Introduction

These specifications cover the allowable stresses, the design rules, and the fabrication procedures for structures built of the high-strength aluminum alloy most commonly used for heavy duty structural purposes. In the preparation of these specifications the Committee has made use of the available theoretical and experimental work relating to this subject and especially to previous specifications by O. H. Ammann, Shortridge Hardesty, and the late Leon S. Moisseiff,[1] Members, ASCE.

These specifications are confined to allowable stresses, design rules, and fabrication. No attempt has been made to cover the loading, erection, inspection, or nontechnical provisions included in many specifications, since such provisions are fairly well established in current good structural practice. Furthermore, no attempt has been made to include design rules which cover every detail of construction but rather those which are different from steel practice or which are needed for the sake of completeness. It is intended, of course, that structures built under these specifications will be designed, constructed, and erected by following the current good practice already well established for steel structures, except as modified herein.

It is believed that the designer can make more efficient use of a set of specifications if he knows the basis for its various provisions. For this reason, Part III, Explanation of Specifications, has been added. Part III contains background information and references concerning those paragraphs of the specifications for which some explanation seems required.

When the abbreviation "kip" is used in these specifications it denotes "kilo"-pounds, or "thousands of pounds."

[1] "Design Specifications for Bridges and Structures of Aluminum Alloy 27S-T," by Leon S. Moisseiff, Aluminum Co. of America, Pittsburgh, Pa., 1940.

Material

The principal material considered in these specifications is a high-strength aluminum alloy having the following nominal chemical composition:

Composition	*Percentage by Weight*
Copper	4.4
Silicon	0.8
Manganese	0.8
Magnesium	0.4
Aluminum	93.6
Total	100.0

This material is covered by the American Society for Testing Materials (ASTM) Specifications Nos. B221-54T(CS41A), B235-54T(CS41A), B211-54T(CS41A), B247-54T(CS41A), and B209-54T (Clad CS41A). It is produced by several manufacturers under the commercial designation 2014-T6 and is available in the form of shapes, tubes, rods, bars, and forgings. It is also produced in the form of sheet and plate covered on both surfaces with an integral coating, or "cladding," of a corrosion-resistant aluminum alloy. In the latter form it is identified commercially by the designation Alclad 2014-T6 (formerly Alclad 14S-T6 and R-301). All these products are given a solution heat treatment and a precipitation heat treatment before being shipped.

The specified minimum tensile properties of this material are not the same in all the various products (plate, shapes, etc.). The specified minimum tensile strengths vary from 60 kips per sq in. to 68 kips per sq in., and the specified minimum yield strengths vary from 53 kips per sq in. to 58 kips per sq in. The following are the lowest of the various specified minimum properties and have been used as a basis for the selection of allowable stresses in these specifications (in kips per square inch):

Description	*Stress*
Tensile strength	60
Yield strength (offset 0.2%)	53

In addition to the specified minimum tensile properties, the engineer will be interested in some of the other mechanical properties not covered by specifications. The following are typical mechanical properties of this alloy and may be considered applicable to "nonclad" products, such as shapes, and to "clad" plate:

Shear strength, in kips per square inch	41
Modulus of elasticity in tension and compression, in kips per square inch	10,600
Modulus of elasticity in shear, in kips per square inch	4,000
Poisson's ratio	1/3
Coefficient of expansion per degree Fahrenheit	0.000012
Weight, in pounds per cubic inch	0.101

(The foregoing value of shear strength is typical as determined with steel shearing tools. The value determined by torsion tests is greater.)

Fig. 1 shows tensile and compressive stress-strain curves and the compressive tangent modulus curve for 2014-T6 material having the minimum properties listed in the second paragraph of this section.

Alloy 2014-T6 is the one principally considered in the preparation of these specifications and the one to which the allowable stresses for parts other than rivets and bolts apply. However, these specifications may be applied to structures built of other suitable aluminum alloys, provided such alloys meet the specified strengths and elongations listed in the ASTM specifications mentioned in the first paragraph of this section.

Rivets used in fabricating structures designed in accordance with these specifications shall be of aluminum alloy and may be either cold driven or hot driven. The alloys used are indicated in Table 1. Supplementary information on riveting was published[2] by E. C. Hartmann, M. ASCE, G. O. Hoglund, and M. A. Miller in 1944.

Permanent bolts used in structures designed in accordance with these specifica-

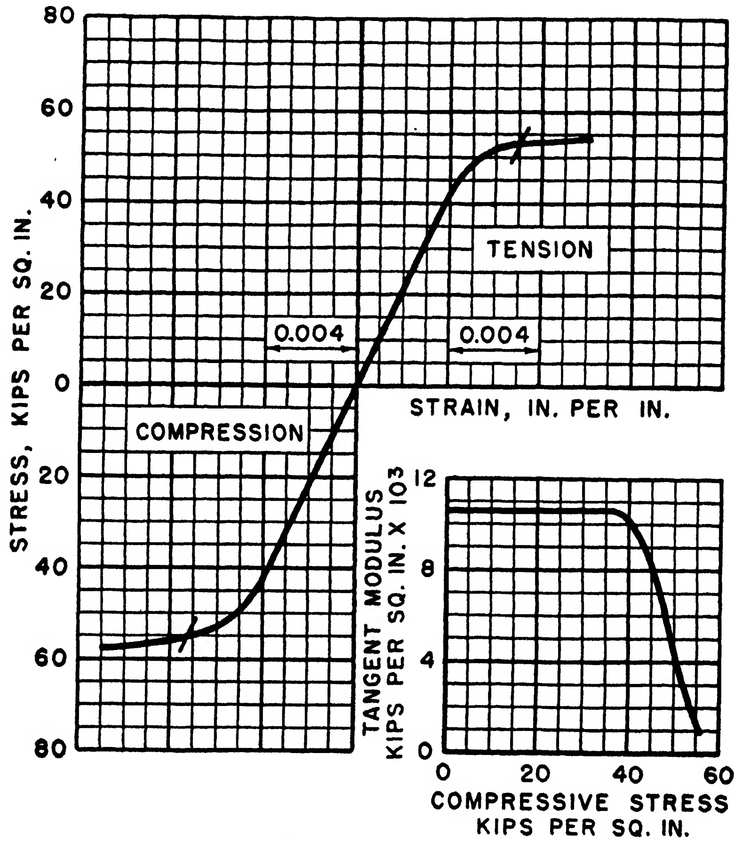


FIG. 1. Stress-strain and tangent-modulus curves.

[2] "Joining Aluminum Alloys," by E. C. Hartmann, G. O. Hoglund, and M. A. Miller, *Steel*, August 7, 1944, p. 84.

tions shall be of the aluminum alloy known commercially as 2024-T4. Such bolts have an expected minimum ultimate shear strength of 37 kips per sq in.

TABLE 1. ALLOYS TO BE USED FOR RIVETS

Designation before driving	Driving procedure	Designation after driving	Typical shear strength[a]
2117-T4	Cold, as received	2117-T3	33
6061-T4	Hot, 990° F to 1,050° F	6061-T43	24

[a]Typical ultimate shear strength of the driven rivet, in kips per square inch.

The materials covered by these specifications are heat treated for maximum strength. They cannot be welded without a considerable loss in strength. Structures designed under these specifications shall be assembled by riveting or bolting.

PART II. SPECIFICATIONS

Section A. Summary of Allowable Stresses

The allowable stresses to be used in proportioning the parts of a structure shall be as follows:

Specification	*Description*	*Stress in kips per square inch*
A-1	Axial tension, net section (see Specification II-4)	22
A-2	Tension in extreme fibers of shapes, girders, and built-up members subject to bending, net section (see Specification H-4)	22
A-3	Axial compression (see Section B)	..
A-4	Compression in extreme fibers of shapes, girders, and built-up members subject to bending (see Section C)	..
A-5	Compression in plates, legs, and webs (see Section D)	..
A-6	Stress in extreme fibers of pins	34
A-7	Shear in plates and webs (see Section E)	..
A-8	Shear in aluminum alloy 2117-T3 rivets, cold driven (see Tables 4 and 5)	10
A-9	Shear in aluminum alloy 6061-T43 rivets, driven at temperatures of from 990° F to 1,050° F (see Tables 4 and 6)	8
A-10	Shear in turned bolts of aluminum alloy 2014-T4 in reamed holes (see Table 4)	12
A-11	Shear in pins	16
A-12	Bearing on pins	30
A-13	Bearing on hot-driven or cold-driven rivets, milled stiffeners, turned bolts in reamed holes, and other parts in fixed contact (see Section G)	36

Section B. Column Design

B-1. Allowable Compressive Stress in Columns. The allowable compressive stress on the gross section of axially loaded columns shall be determined from the curves in Fig. 2. Let k be a factor describing end restraint. Ordinarily the curve

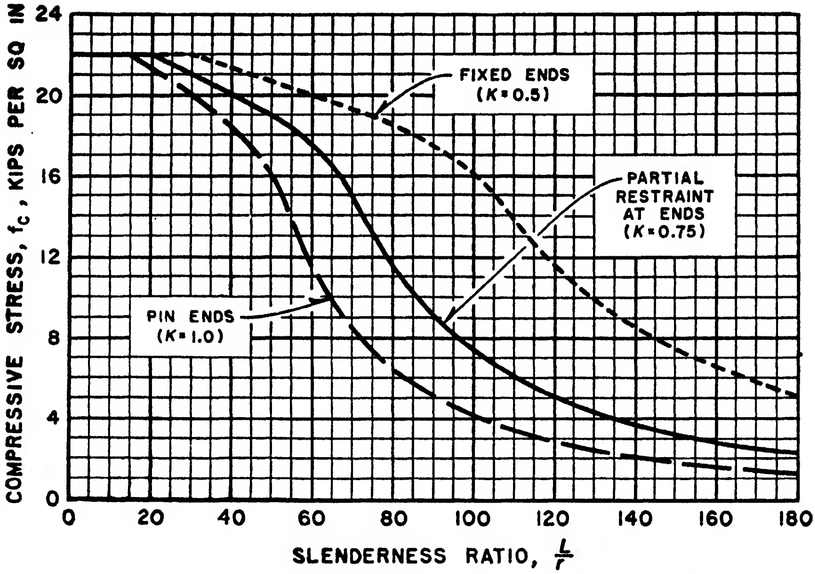


FIG. 2. Allowable compressive stresses for axially loaded columns (gross section).

for partial restraint (k = 0.75) shall be used. The curves for pin-ended and fixed-ended columns, also shown in Fig. 2, may be used as a guide in the selection of allowable compressive stresses for those cases in which the degree of end restraint is known to be different from that represented by k = 0.75. It is important, however, that no allowable stresses higher than those given for the case of k = 0.75 be used in actual design unless a detailed analysis of the structure demonstrates convincingly that a value of k smaller than 0.75 is justified for the member in question.

Columns having cross sections involving webs and outstanding legs of such proportions that local buckling may control the design shall be checked by the method outlined in Section D.

B-2. Columns with Slenderness Ratio Exceeding 120. Because long columns are relatively flexible, they may be appreciably weakened by the presence of lateral loads that would have little effect on the column strength of stiffer members. For this reason, columns with slenderness ratios greater than 120 should not be used unless special care is taken to insure that the effect of any lateral loads to which the member may be subjected, such as wind, dead load, or the weight of workmen and equipment, are taken into account by using the provision for combined compression and bending in Specification B-7.

B-3. Connections. Compression members shall be so designed that the main elements of the section will be connected directly to the gusset plates, pins, or other members.

B-4. Compression Splices. Members designed for compression, if faced for bearing, shall be spliced on four sides sufficiently to hold the abutting parts true to place.

The splice shall be as near a panel point as practicable and shall be designed to transmit at least one half of the stress through the splice material. Members not faced for bearing shall be fully spliced for the computed stress. In either case, adequate provision shall be made for transmitting shear.

B-5. Stay Plates. On the open sides of compression members, the flanges shall be connected by lacing bars, and there shall be stay plates as near each end as practicable. There shall be stay plates at intermediate points where the lacing is interrupted. The length of the end stay plates shall not be less than one and one-fourth times the distance between rivet lines. The thickness of stay plates shall not be less than one-fortieth of the distance between rivet lines.

B-6. Diagonal Lacing. The slenderness ratio of the part of the flange between the lacing bar connections shall be not more than two-thirds of the slenderness ratio of the member.

B-7. Combined Compression and Bending. The allowable stress in a member that carries bending moment in addition to uniform compression (as, for example, an eccentrically loaded column) shall be determined from the following formula:

$$f_b = f_B\left(1 - \frac{P/A}{f_C}\right)\left(1 - \frac{P/A}{f_{CE}}\right) \tag{1}$$

in which (in kips per sq in.):

f_b is the maximum bending stress (compression) that may be permitted at or near the center of the unsupported length, in addition to uniform compression, P/A;

P/A is the average compressive stress on the gross cross section, A, of the member, produced by a column load, P;

f_B is the allowable compressive working stress for the member considered as a beam;

f_C is the allowable working stress for the member considered as an axially loaded column; and

$$f_{CE} = \frac{74{,}000}{(L/r)^2} \tag{2}$$

in which L/r is the slenderness ratio for the member considered as a column tending to fail in the plane of the bending forces.

B-8. Transverse Shear in Columns. In designing lacing or shear webs for columns, the maximum shear on the column shall be computed from the formula:

$$V = P\,\frac{4.5\,r^2\,(f_B - P/A)}{f_C\,L\,c} + V_t \tag{3a}$$

but shall not be taken less than

$$V = 0.02\,P + V_t \tag{3b}$$

in which V is the maximum shear on any transverse section of a column in the outer eighth of the length at each end, in the direction of assumed bending, in kips;

r is the radius of gyration, in inches;

f_C is the allowable compressive stress taken from Fig. 2, in kips per square inch;

L is the length of the member, in inches;

c is the distance from the centroidal axis to the extreme fiber, in inches; and

V_t is the shear due to any transverse loads on a column, in kips.

The values of f_B, f_C, L, r, and c must be consistent with the direction of bending assumed.

Section C. Allowable Compressive Stresses in Flanges of Beams and Girders

C-1. The allowable compressive stress in the extreme fiber (gross section) of single-web rolled shapes, extruded shapes, girders, and built-up sections, subject to bending, shall be determined from the curve in Fig. 3. The terms used in Fig. 3 are defined as follows:

L is the laterally unsupported length of beam (clear distance between supports at which the beam is prevented from lateral displacement), or, in the case of a cantilever beam with one end free, L is four-thirds of the laterally unsupported length, in inches.

S_c is the section modulus for the beam about the axis normal to the web (compression side), in inches;

$$B = I_1 d \sqrt{11.7 + \frac{J}{I_1}\left(\frac{L}{d}\right)^2}$$

I_1 is the moment of inertia for the beam about the axis parallel to the web, in inches to the fourth power;

J is the torsion factor, in inches to the fourth power; and

d is the depth of beam, in inches.

In the case of beams having top and bottom flanges of different lateral stiffness, I_1 should be calculated as if both flanges were the same as the compression flange. Values of the torsion factor J are published for many standard shapes.[3] Values of J for plates and shapes not published may be calculated by assuming the section to be composed of rectangles and taking the sum of the terms $bt^3/3$ for each rectangle, in which b equals the length and t, the thickness of the rectangle, both in inches. The value of J for a built-up member is the sum of the individual values of J of the sections of which it is composed.

The allowable stresses from Fig. 3 provide a safe margin against the lateral buckling type of failure. The outstanding compression flanges of the beams and girders should be checked for local buckling by the method outlined in Section D.

Table 2 lists values of allowable stress determined from Fig. 3 and Section D for various laterally unsupported lengths of a number of standard I-beams and H-beams. Table 3 lists similar values for standard channels.

Because of their tube-like cross section, double-web box girders are very stiff in

[3] "Alcoa Structural Handbook," Aluminum Co. of America, Pittsburgh, Pa., 1955, pp. 204–226.

torsion compared with single-web girders of comparable size, and hence, lateral buckling failures such as are considered in Fig. 3 do not occur in such girders. For double-web box girders it is necessary only to check for local buckling of the flanges by the method outlined in Section D.

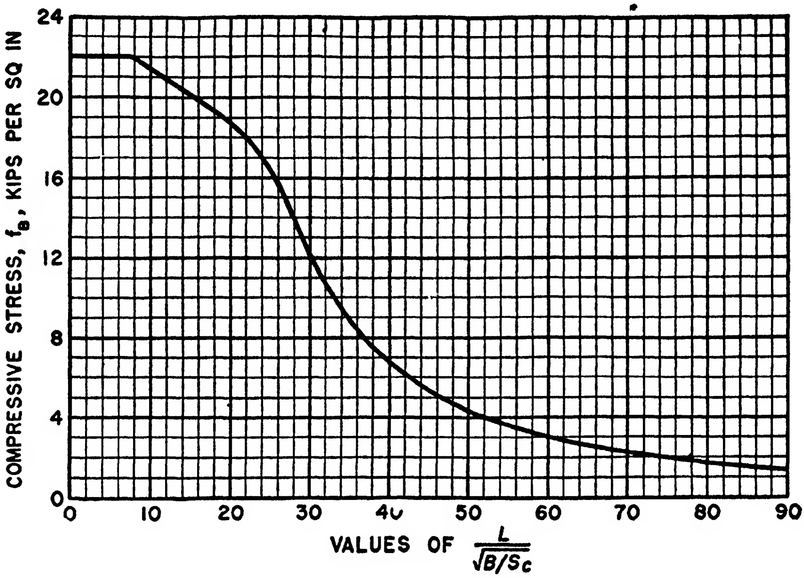


FIG. 3. Allowable compressive stresses in beam and girder flanges (gross section).

Section D. Allowable Compressive Stress for Plates, Legs, and Webs

D-1. For struts consisting of a single angle or a T-section, the compressive stress on the gross area shall not exceed the values given by the curves in Fig. 4 or Fig. 2, whichever is smaller.

D-2. For compression members other than those consisting of a single angle or a T-section, the following procedure shall be followed to provide a suitable margin of safety against the weakening effects of local buckling of flat plates, legs, and webs:

a. Compute the compressive stress f_c on the flat plate, leg, or web in question, based on the design loads and the gross area, without regard to local buckling. This stress must be within allowable limits as defined in Sections B and C.

b. Find the limiting value of b/t corresponding to the stress, f_c, by the use of Fig. 5 or Fig. 6. If the flat plate, leg, or web has a ratio of unsupported width to thickness not exceeding this limiting value, local buckling is not a problem and the full gross area of the plate, leg, or web may be considered effective.

c. If the flat plate, leg, or web has a ratio of unsupported width to thickness greater than the limiting b/t-ratio found in step b, only a part of its unsupported width shall be included in computing its effective area. The part of the unsupported width of any individual flat plate, leg, or web which may be considered effective shall be found as given in Eq. 4.

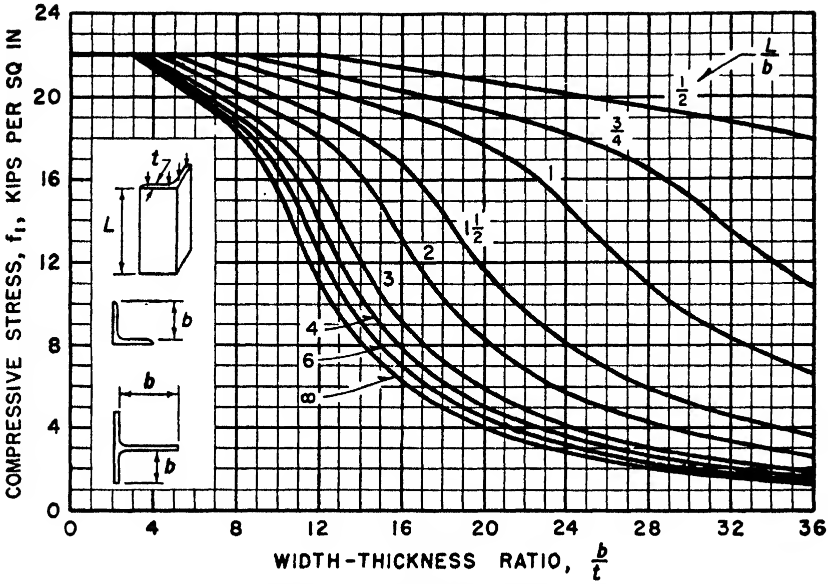


FIG. 4. Allowable compressive stresses in outstanding legs of single-angle and T-section struts (gross section).

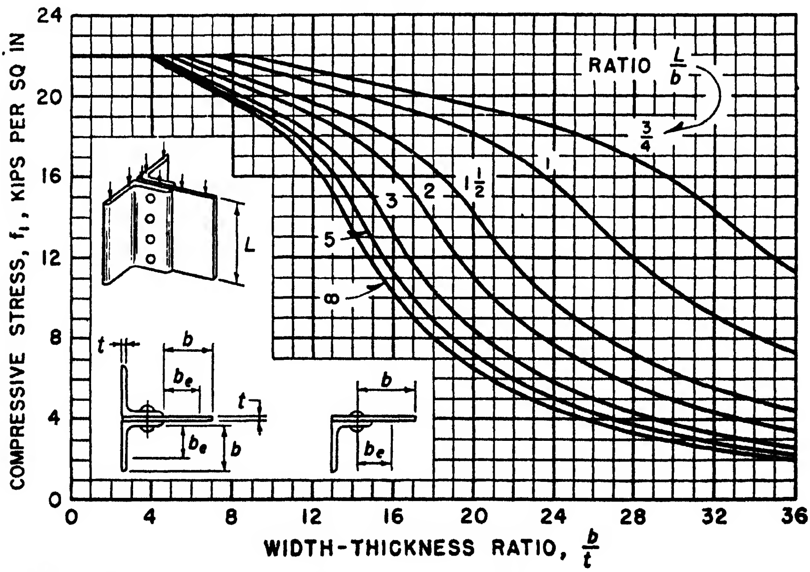


FIG. 5. Chart for determining effective width for outstanding legs of angles built into other parts and for plates built in along one edge.

TABLE 2. ALLOWABLE COMPRESSIVE STRESS IN BEAM FLANGES FOR VARIOUS VALUES OF LATERALLY UNSUPPORTED LENGTH OF COMPRESSION FLANGE, L, IN INCHES

Procedure.—Maximum allowable bending moments are found by multiplying the allowable compressive stresses (in kips per square inch) by the gross section modulus of the beam. The stress on the net section of the tension flange must also be kept within allowable limits.

Depth (in.)	Weight (lb per ft)	Section Modulus (in.³)	Values of L									
			16	32	64	96	128	160	192	256	352	480
			(a) I-Beams									
2	0.78	0.481	20.6[a]	18.8	10.2	6.1	4.4	3.5	2.9	2.1	1.6	..
2	1.43	0.782	21.6	19.9	17.0	12.2	9.1	7.2	6.0	4.5	3.3	2.4
2.5	1.80	1.162	21.8	20.3	18.3	15.6	12.0	9.6	8.0	6.0	4.4	3.2
3	1.96	1.68	21.4	19.8	15.8	10.2	7.5	5.9	5.0	3.7	2.7	2.0
3	2.59	1.95	21.7	20.3	17.9	14.1	10.4	8.3	6.9	5.2	3.8	2.8
4	2.64	3.03	21.7	20.0	15.7	9.7	7.1	5.5	4.6	3.4	2.5	1.8
4	3.63	3.59	21.8[a]	20.4	17.8	13.6	10.0	7.9	6.5	4.9	3.5	2.6
5	3.43	4.90	21.8[a]	20.2	16.2	9.9	7.1	5.5	4.5	3.4	2.4	1.8
5	5.10	6.09	21.9[a]	20.6	18.2	14.7	10.8	8.5	7.0	5.2	3.8	2.8
6	4.30	7.36	21.8[a]	20.5	16.9	10.5	7.3	5.6	4.6	3.4	2.4	1.8
6	5.96	8.77	21.8[a]	20.7	18.0	13.6	9.8	7.6	6.3	4.6	3.4	2.5
7	5.27	10.48	21.8[a]	20.8	17.5	11.2	7.7	5.9	4.7	3.4	2.5	1.8
7	6.92	12.12	21.8[a]	20.9	18.2	13.4	9.4	7.3	5.9	4.4	3.1	2.3
8	6.35	14.39	21.8[a]	21.0	18.0	12.2	8.2	6.1	5.0	3.6	2.5	1.8
8	8.81	17.18	21.8[a]	21.1	18.7	15.0	10.5	8.0	6.6	4.8	3.4	2.5
9	7.51	19.09	21.8[a]	21.2	18.5	13.2	8.8	6.5	5.2	3.7	2.6	1.9
9	10.37	22.75	21.8[a]	21.2	18.9	15.6	10.9	8.3	6.7	4.9	3.5	2.5
10	8.76	24.68	21.8[a]	21.3	18.8	14.5	9.4	6.9	5.5	3.9	2.7	2.0
10	12.10	29.41	21.8[a]	21.4	19.2	16.3	11.5	8.7	7.0	5.1	3.6	2.6
12	10.99	36.35	21.8[a]	21.5	19.1	15.2	9.8	7.1	5.6	3.9	2.7	1.9
12	17.28	50.81	22.0	21.7	19.8	17.6	14.0	10.4	8.4	6.0	4.2	3.1
			(b) H-Beams									
4	4.71	5.36	21.7[a]	21.4[a]	19.6	17.7	15.1	11.8	9.7	7.2	5.2	3.8
5	6.45	9.53	21.5[a]	21.5[a]	20.1	18.5	16.4	13.1	10.6	7.7	5.5	4.0
6	7.81	14.69	21.3[a]	21.3[a]	20.5	19.0	17.2	14.4	11.4	8.0	5.6	4.0
6	9.14	15.81	21.3[a]	21.3[a]	20.6	19.2	17.7	15.6	12.7	9.0	6.3	4.6
8	11.19	28.23	20.7[a]	20.7[a]	20.4[a]	19.7[a]	18.6	16.7	13.7	9.1	6.0	4.2
8	12.95	30.23	20.7[a]	20.7[a]	20.4[a]	19.8[a]	18.8	17.2	14.8	10.0	6.7	4.7

[a]These values are governed by local buckling (see Section D). All other values are determined from Fig. 3, Section C.

$$b_e = b\frac{f_l}{f_c} \tag{4}$$

in which:

b_e is that part of the unsupported width considered effective, in inches;
b is the unsupported width, in inches;
f_c is the compressive stress based on gross area from step (a), in kips per square inch; and
f_l is the stress found from Fig. 5 or Fig. 6 corresponding to the b/t-value for the plate, leg, or web in question, in kips per square inch.

TABLE 3. ALLOWABLE COMPRESSIVE STRESS IN CHANNEL FLANGES FOR VARIOUS VALUES OF LATERALLY UNSUPPORTED LENGTH OF COMPRESSION FLANGE, L, IN INCHES

Procedure.—Maximum allowable bending moments are found by multiplying the allowable compressive stresses (in kips per square inch) by the gross section modulus of the beam. The stress on the net section of the tension flange must also be kept within allowable limits.

Depth (in.)	Weight (lb per ft)	Section Modulus (in.^{3})	Values of L									
			16	32	64	96	128	160	192	256	352	480
3	1.42	1.10	21.2	19.3	13.2	8.4	6.2	4.9	4.1	3.1	2.2	1.6
3	2.07	1.38	21.6	20.0	17.4	13.2	9.9	7.8	6.5	4.8	3.5	2.6
4	1.85	1.92	21.4	19.2	12.3	7.6	5.5	4.4	3.6	2.7	2.0	1.4
4	2.50	2.29	21.6[a]	19.7	15.7	10.2	7.4	5.9	4.9	3.6	2.6	2.0
5	2.32	3.00	21.6[a]	19.4	12.3	7.4	5.3	4.2	3.4	2.8	1.8	1.3
5	3.97	4.17	21.7[a]	20.2	17.4	12.9	9.5	7.5	6.2	4.6	3.4	2.5
6	2.83	4.37	21.5[a]	19.7	12.9	7.4	5.2	4.1	3.3	2.4	1.8	1.3
6	4.48	5.80	21.7[a]	20.2	16.7	11.1	8.0	6.3	5.2	3.9	2.8	2.0
7	3.38	6.08	21.6[a]	20.0	14.0	7.8	5.4	4.1	3.4	2.5	1.8	1.3
7	5.96	8.64	21.7[a]	20.4	17.5	12.5	9.0	7.0	5.8	4.3	3.1	2.2
8	4.25	8.46	21.7[a]	20.2	15.3	8.4	5.7	4.3	3.5	2.6	1.8	1.3
8	6.79	11.34	21.7[a]	20.5	17.5	12.2	8.7	6.7	5.5	4.1	2.9	2.1
9	4.60	10.60	21.6[a]	20.4	16.0	8.8	5.8	4.4	3.5	2.5	1.8	1.3
9	8.65	15.75	21.7[a]	20.8	18.1	13.7	9.7	7.4	6.2	4.5	3.2	2.4
10	5.28	13.47	21.6[a]	20.6	16.6	9.3	6.1	4.5	3.6	2.5	1.8	1.3
10	10.37	20.69	21.7[a]	21.0	18.5	14.8	10.4	8.0	6.5	4.8	3.4	2.5
12	7.41	21.97	21.6[a]	21.0	17.9	11.3	7.2	5.2	4.1	2.9	2.0	1.4
12	12.10	29.94	21.7[a]	21.1	18.6	14.6	9.9	7.5	6.0	4.4	3.1	2.2

[a]These values are governed by local buckling (see Section D). All other values are determined from Fig. 3, Section C.

d. Compute the compressive stress on the effective area. In the case of an axially loaded column this is simply the axial load divided by the total effective area, which, in turn, is simply the sum of the effective areas of the component parts. In the case of a beam or girder the compressive stress on the effective area shall be determined as follows: Compute the compressive extreme fiber stress f_c for the gross section of the

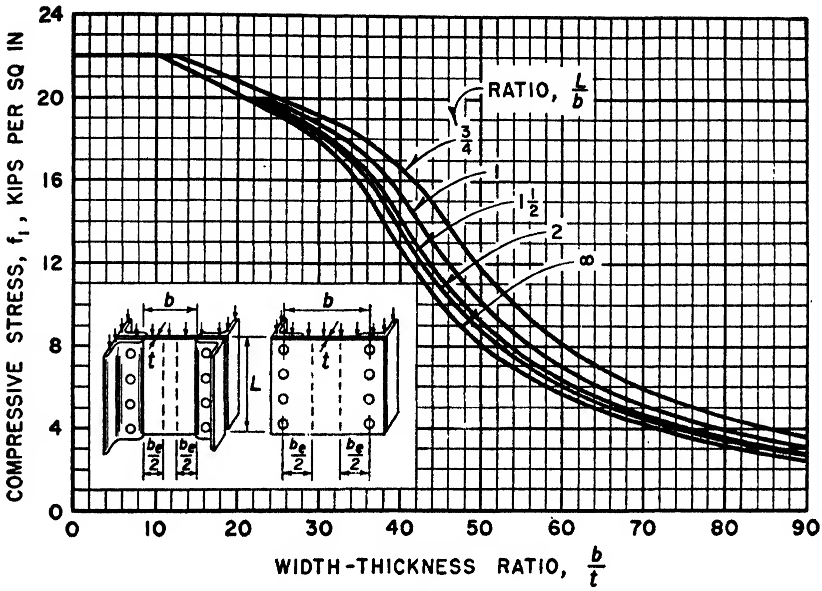


FIG. 6. Chart for determining effective width for flat plates built in along two edges.

beam or girder and then multiply this value by the ratio of the gross compression flange area to the effective compression flange area, including in each flange area not only the flange proper but also one-sixth of the area of the web.

e. The compressive stress on the effective area computed in accordance with step *d* shall not exceed allowable limits as defined in Sections B and C for the gross area.

f. Steps *c*, *d* and *e* provide a suitable factor of safety against collapse of the member as a whole, but do not necessarily provide complete protection against local buckling of individual flat surfaces at the design load. Where local buckling at the design load cannot be tolerated because of appearance, or for other reasons, the computed compressive stress on the gross area, f_c, shall not exceed $1.5f_l$, where f_l is the compressive stress given in Fig. 5 or Fig. 6 for the b/t ratio in question. Regardless of whether or not this limitation is placed on f_c, the member must still be checked by the method outlined in steps *a* through *e* to insure an adequate factor of safety against collapse.

Section E. Allowable Shear Stresses in Plates and Webs

E-1. The allowable shear stress on flat webs shall not exceed the values given by the curves in Fig. 7. The values in Fig. 7 apply to the gross area of the web, but the shear on the net area shall not exceed 15 kips per sq in.

Section F. Plate Girder Design

F-1. Proportioning Plate Girders. Plate girders shall be proportioned by the moment of inertia method, with the gross section used to determine the moment of inertia.

The stress on the net area of the tension flange shall be found by multiplying the stress on the gross section by the ratio of the gross area of the tension flange to the net area. In determining this ratio the tension flange shall be considered to consist of the flange angles and cover plates plus one-sixth of the web.

F-2. Allowable Flange Stress. The allowable compressive stress in the extreme fiber of plate girders shall be determined as outlined in Sections C and D. The numerical value of the term $\sqrt{B/S_c}$, used in Fig. 3, is rarely less than one-half of the width, in inches, of the compression flange for a plate girder. This fact is useful in preliminary design.

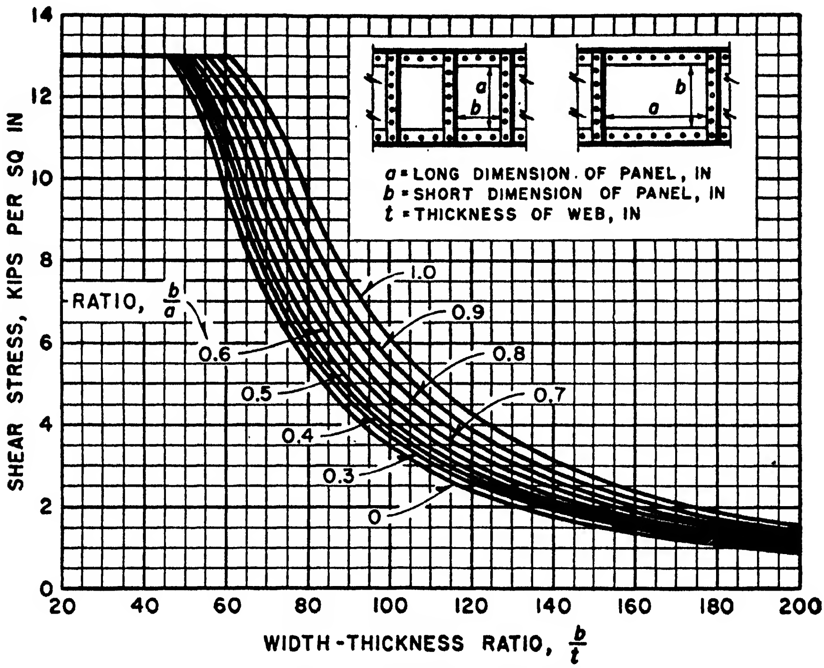


FIG. 7. Allowable shear stresses on webs; partial restraint assumed at edges of rectangular panels (gross section).

F-3. Flange Cover Plates. Cover plates shall extend far enough to allow at least two extra rivets at each end of the plate beyond the theoretical end, and the spacing of the rivets in the remainder of the plate shall be such as to develop the required strength of the plate at any section.

F-4. Flange Rivets. The flanges of plate girders shall be connected to the web with enough rivets to transmit the longitudinal shear at any point together with any load that is applied directly on the flange.

F-5. Flange Splices. It is preferable that flange angles be spliced with angles and that no two members be spliced at the same cross section.

F-6. Allowable Web Stresses. The allowable shear stress in the webs of plate girders shall not exceed the values given by the curves in Fig. 7. The longitudinal compressive stress in webs of plate girders at the toe of the compression flange shall not exceed the values given by the curves in Fig. 8.

F-7. Web Splices. It is preferable that splices in the webs of plate girders be made with splice plates on both sides of the web.

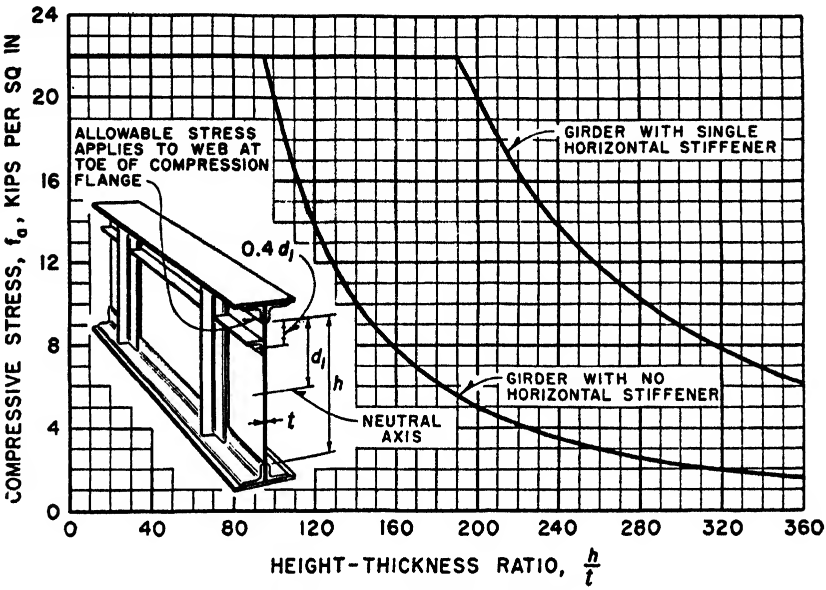


FIG. 8. Allowable longitudinal compressive stresses for webs of girders.

F-8. Spacing of Vertical Stiffeners to Resist Shear Buckling. The distance, s, between vertical stiffeners shall not exceed the values given by the solid curves in Fig. 9, which are replots of the curves in Fig. 7. The maximum value of the ratio of stiffener spacing to height of web, s/h, in Fig. 9 shall be determined from the ratio of clear height to thickness, h/t, and the computed shear stress on the girder web. Where a stiffener is composed of a pair of members, one on each side of the web, the distance s shall be the clear distance between the stiffeners. Where a stiffener is composed of a member on one side of the web only, the distance s shall be the distance between rivet lines. In determining the spacing of vertical stiffeners to resist shear buckling in panels containing a horizontal stiffener located as shown in Fig. 8, the distance h in Fig. 9 may be taken as 90% of the clear height between flanges.

F-9. Size of Vertical Stiffeners to Resist Shear Buckling. Stiffeners applied to plate girder webs to resist shear buckling shall have a moment of inertia not less than the values given by the dotted curves in Fig. 9. The minimum value of the ratio of the stiffener moment of inertia to the fourth power of the web thickness, I_s/t^4, in Fig. 9, shall be determined from the ratio of height of web to thickness of web, h/t, and the computed shear stress on the girder web.

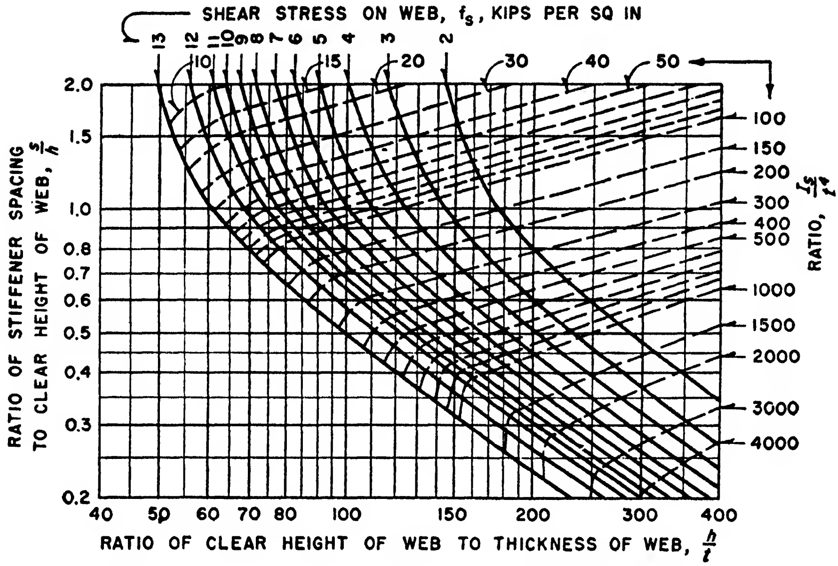


FIG. 9. Spacing and moment of inertia of vertical stiffeners to resist shear buckling on webs of plate girders.

For a stiffener composed of members of equal size on both sides of the web, the moment of inertia shall be taken about the center line of the web. For a stiffener composed of a member on one side only, the moment of inertia shall be taken about the face of the web in contact with the stiffener. In determining moment of inertia of stiffeners, the term h shall always be taken as the full clear height between flanges, regardless of whether or not a horizontal stiffener is present.

F–10. Vertical Stiffeners at Points of Bearing. Stiffeners shall be placed in pairs at end bearings of plate girders and at points of bearing of concentrated loads. They shall be connected to the web by enough rivets to transmit the load. Such stiffeners shall have a close bearing against the loaded flanges. Only that part of the stiffener cross section which lies outside the fillet of the flange angle shall be considered effective in bearing.

The moment of inertia of the stiffener shall not be less than that given by the formula:

$$I = I_s + \frac{P h^2}{74{,}000} \tag{5}$$

in which:

I_s is the moment of inertia, in inches to the fourth power, required to resist shear buckling (Fig. 9);

P is a local load concentration on the stiffener, in kips; and

h is the clear height of the web between flanges, in inches.

F–11. Horizontal Stiffeners. A horizontal stiffener of the type shown in Fig. 8 shall have a moment of inertia not less than that given by the following formula:

$$I_h = f\,t\,h^3\left[\left(16 + 90\,\frac{A_h}{h\,t}\right)\left(\frac{s}{h}\right)^2 + 6\right] \times 10^{-7} \tag{6}$$

in which:

I_h is the moment of inertia of the horizontal stiffener, in inches to the fourth power;

f is the compressive stress at the toe of the flange angles, in kips per square inch;

t is the thickness of the web, in inches;

h is the clear height of the web between flanges, in inches;

s is the distance between vertical stiffeners, in inches;

and

A_h is the gross area of cross section of the horizontal stiffener, in square inches.

For a stiffener composed of members of equal size on both sides of the web, the moment of inertia shall be taken about the center line of the web. In the case of a stiffener consisting of a member on one side only, the moment of inertia shall be taken about the face of the web in contact with the stiffener.

Eq. 6 must be solved by trial, since both the moment of inertia, I_h, and the area, A_h, of the stiffener are unknown. It is generally convenient to assume as a first approximation that the ratio A_h/ht has the value of 0.1.

Section G. Riveted and Bolted Connections

G–1. Allowable Loads. The allowable loads on rivets and bolts shall be calculated using the allowable shear and bearing stresses listed in Section A, with the following exceptions:

a. If a rivet or a bolt is used in relatively thin plates or shapes, the allowable shear stress shall be reduced in accordance with the information given in Table 4.

b. If the distance from the center of a rivet or bolt to the edge of a plate or shape toward which the pressure of the rivet or bolt is directed is less than twice the diameter of the rivet or bolt, the allowable bearing stress shall be reduced in accordance with the following:

Ratio of edge distance to rivet or bolt diameter	*Allowable bearing stress, in kips per square inch*
2 or more	36
1¾	33
1½	30

The allowable loads calculated for cold-driven 2117-T3 rivets are given in Table 5 and those for 6061-T43 hot-driven rivets are given in Table 6.

G–2. Effective Diameter. The effective diameter of rivets shall be taken as the hole diameter but shall not exceed the values of hole diameter given in Table 5 for cold-driven rivets and in Table 6 for hot-driven rivets. The effective diameter of pins and bolts shall be the nominal diameter of the pin or bolt.

G–3. Bearing Area. The effective bearing area of pins, bolts, and rivets shall be the effective diameter multiplied by the length in bearing; except that for countersunk rivets, half of the depth of the countersink shall be deducted from the length.

TABLE 4. PERCENTAGE REDUCTION IN SHEAR STRENGTH OF ALUMINUM ALLOY RIVETS RESULTING FROM THEIR USE IN THIN PLATES AND SHAPES

Ratio,[a] $\frac{D}{t}$ (1)	Loss in double shear[b] (3)	Ratio,[a] $\frac{D}{t}$ (1)	Loss in double shear[b] (3)	Ratio,[a] $\frac{D}{t}$ (1)	Loss in: Single shear (2)	Loss in: Double shear (3)	Ratio,[a] $\frac{D}{t}$ (1)	Loss in: Single shear (2)	Loss in: Double shear (3)
1.5	0	2.2	9.1	2.9	0	18.2	3.5	2.0	26.0
1.6	1.3	2.3	10.4	3.0	0	19.5	3.6	2.4	27.3
1.7	2.6	2.4	11.7	3.1	0.4	20.8	3.7	2.8	28.6
1.8	3.9	2.5	13.0	3.2	0.8	22.1	3.8	3.2	29.9
1.9	5.2	2.6	14.3	3.3	1.2	23.4	3.9	3.6	31.2
2.0	6.5	2.7	15.6	3.4	1.6	24.7	4.0	4.0	32.5
2.1	7.8	2.8	16.9	....	....	....	....	....	....

[a]Ratio of the rivet diameter, D, to the plate thickness, t. The thickness used is that of the thinnest plate in a single shear joint or of the middle plate in a double shear joint. [b]The percentage loss of strength in single shear is zero for D/t less than 3.0.

TABLE 5. ALLOWABLE DESIGN LOAD, IN KIPS PER RIVET, (SHEAR, 10 KIPS PER SQ IN. AND

Dimensions, in Inches: Rivet diameter / Hole diameter / Drill size	3/8 / 0.386 / W		7/16 / 0.453 / 29/64		1/2 / 0.516 / 33/64		9/16 / 0.578 / 37/64	
Thickness of plate, or shape, in inches:	RIVET IN SINGLE SHEAR (ss)							
	ss	ds	ss	ds	ss	ds	ss	ds
⅛	1.17	1.74[b]	1.58[c]	2.04[b]	2.01[c]	2.32[b]	2.52[c]	2.60[b]
3/16	1.17	2.19[c]	1.61	2.88[c]	2.09	3.48[b]	2.62	3.90[b]
¼	1.17	2.34	1.61	3.12[c]	2.09	3.91[c]	2.62	4.74[c]
5/16	1.17	2.34	1.61	3.22	2.09	4.13[c]	2.62	5.04[c]
⅜	1.17	2.34	1.61	3.22	2.09	4.18	2.62	5.25
7/16	....	....	1.61	3.22	2.09	4.18	2.62	5.25
½	....	....	....	....	2.09	4.18	2.62	5.25
9/16	....	....	....	....	....	....	2.62	5.25
⅝	....	....	....	....	....	....	....	....
¾	....	....	....	....	....	....	....	....
⅞	....	....	....	....	....	....	....	....
1	....	....	....	....	....	....	....	....

[a]Assuming distance from center of rivet to edge of member toward which the pressure of the ception b). [b]These values are governed by bearing. [c]These values are governed by reduced stress.

G–4. Arrangement and Strength of Connections. Connections shall be arranged to minimize the eccentricity of loading on the member. Members and connections shall be proportioned to take into account any eccentricity of loading introduced by the connections.

G–5. Net Section. The net section of a riveted tension member is the sum of the net sections of its component parts. The net section of a part is the product of the thickness of the part multiplied by its least net width. The net width for a chain of holes extending across the part in any straight or broken line shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain and adding $s^2/4g$ for each gage in the chain. In the correction quantity $s^2/4g$:

s is the spacing parallel to direction of load (pitch) of any two successive holes in the chain, in inches; and

g is the spacing perpendicular to direction of load (gage) of the same holes in inches.

The net section of the part is obtained from that chain which gives the least net width. The hole diameter to be deducted shall be the actual hole diameter for drilled or reamed holes.

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 the back of the angle, less the thickness.

For splice members, the thickness shall be only that part of the thickness of the member that has been developed by rivets beyond the section considered.

G–6. Effective Sections of Angles. If an angle in tension is connected on one side

FOR COLD-DRIVEN 2117-T3 RIVETS IN 2014-T6 STRUCTURES
BEARING, 36 KIPS PER SQ IN.[a])

5/8 0.641 41/64		3/4 0.766 49/64		7/8 0.891 57/64		1 1.016 1 1/64		Dimensions, in Inches: Rivet diameter Hole diameter Drill size
OR IN DOUBLE SHEAR (ds)								Thickness of plate, or shape, in inches:
ss	ds	ss	ds	ss	ds	ss	ds	
2.88[b]	2.88[b]	....	....	....	....	....	....	1/8
3.18[c]	4.33[b]	4.42[c]	5.17[b]	5.82[c]	6.02[b]	....	....	3/16
3.23	5.62[c]	4.61	6.89[b]	6.11[c]	8.02[b]	7.78[c]	9.14[b]	1/4
3.23	6.03[c]	4.61	8.14[c]	6.24	10.02[b]	8.04[c]	11.43[b]	5/16
3.23	6.31[c]	4.61	8.62[c]	6.24	11.12[c]	8.11	13.72[b]	3/8
3.23	6.45	4.61	8.97[c]	6.24	11.66[c]	8.11	14.58[c]	7/16
3.23	6.45	4.61	9.22	6.24	12.07[c]	8.11	15.16[c]	1/2
3.23	6.45	4.61	9.22	6.24	12.39[c]	8.11	15.64[c]	9/16
3.23	6.45	4.61	9.22	6.24	12.47	8.11	16.00[c]	5/8
....	....	4.61	9.22	6.24	12.47	8.11	16.22	3/4
....	....	....	....	6.24	12.47	8.11	16.22	7/8
....	....	....	....	....	....	8.11	16.22	1

**rivet is directed is not less than twice the nominal rivet diameter (see Specification G-1, ex-
shear strengths as indicated in Table 4. All other values are governed by basic allowable shear**

TABLE 6. ALLOWABLE DESIGN LOADS, IN KIPS PER RIVET,
(RIVETS DRIVEN AT 990° F TO 1,050° F; SHEAR, 8 KIPS

Dimensions, in Inches: Rivet diameter....... Hole diameter........ Drill size.............	3/8 0.397 X		7/16 0.469 15/32		1/2 0.531 17/32		9/16 0.594 19/32	
Thickness of plate, or shape, in inches:	RIVET IN SINGLE SHEAR (SS)							
	ss	ds	ss	ds	ss	ds	ss	ds
1/8	0.99	1.60[b]	1.35[b]	2.05[b]	1.70[b]	2.39[c]	2.08[b]	2.67[c]
3/16	0.99	1.85[b]	1.38	2.47[b]	1.77	3.01[b]	2.22	3.57[b]
1/4	0.99	1.98	1.38	2.68[b]	1.77	3.31[b]	2.22	4.00[b]
5/16	0.99	1.98	1.38	2.76	1.77	3.50[b]	2.22	4.26[b]
3/8	0.99	1.98	1.38	2.76	1.77	3.54	2.22	4.43
7/16	....	....	1.38	2.76	1.77	3.54	2.22	4.43
1/2	....	....	....	....	1.77	3.54	2.22	4.43
9/16	....	....	....	....	....	....	2.22	4.43
5/8	....	....	....	....	....	....	....	....
3/4	....	....	....	....	....	....	....	....
7/8	....	....	....	....	....	....	....	....
1	....	....	....	....	....	....	....	....

[a]Assuming distance from center of rivet to edge of member toward which the pressure of the
ception *b*). [b]These values are governed by reduced shear strengths as indicated in Table 4.
stress.

of a gusset plate, the effective section shall be the net section of the connected leg plus one half of the section of the outstanding leg, unless the outstanding leg is connected by a lug angle. In the latter case the effective section shall be the entire net section of the angle, and there shall be at least two extra rivets in the lug angle beyond the gusset plate.

G–7. Grip of Rivets. If the grip of rivets carrying calculated stress exceeds four and one-half times the diameter the allowable load per rivet shall be reduced. The reduced allowable load shall be the normal allowable load divided by $\left(\frac{1}{2} + \frac{G}{9D}\right)$, in which G is the grip and D the nominal diameter of the rivet. If the grip exceeds six times the diameter, special care shall be taken in driving the rivets to insure that the holes will be filled completely.

G–8. Pitch of Rivets in Built-Up Compression Members. The pitch in the direction of stress shall be such that the allowable stress on the individual outside plates and shapes, treated as columns having a length equal to the rivet pitch in accordance with Fig. 2, exceeds the calculated stress. In no case, however, shall the pitch in the direction of stress exceed six times the diameter of the rivets; and for a distance of one and one-half times the width of the member at each end, the pitch in the direction of stress shall not exceed three and one-half times the diameter of the rivets.

G–9. Stitch Rivets. Where two or more web plates are in contact, there shall be stitch rivets to make them act in unison. In compression members, the pitch of such rivets in the direction of stress shall be determined as outlined in Specification

FOR HOT-DRIVEN 6061-T43 RIVETS IN 2014-T6 STRUCTURES
PER SQ IN.; AND BEARING, 36 KIPS PER SQ IN.[a])

5/8 0.656 21/32		3/4 0.781 25/32		7/8 0.922 59/64		1 1.063 1 1/16		Dimensions, in Inches: Rivet diameter Hole diameter Drill size
OR IN DOUBLE SHEAR (ds)								Thickness of plate, or shape, in inches:
ss	ds	ss	ds	ss	ds	ss	ds	
2.49[b]	2.95[c]	....	....	....	....	....	....	1/8
2.66[b]	4.12[b]	3.68[b]	5.17[b]	4.98[b]	6.22[c]	....	....	3/16
2.70	4.71[b]	3.83	6.17[b]	5.23[b]	7.91[b]	6.82[b]	9.59[b]	1/4
2.70	5.06[b]	3.83	6.77[b]	5.34	8.88[b]	7.04[b]	10.88[b]	5/16
2.70	5.29[b]	3.83	7.17[b]	5.34	9.53[b]	7.10	12.04[b]	3/8
2.70	5.41	3.83	7.46[b]	5.34	9.99[b]	7.10	12.77[b]	7/16
2.70	5.41	3.83	7.67	5.34	10.34[b]	7.10	13.28[b]	1/2
2.70	5.41	3.83	7.67	5.34	10.61[b]	7.10	13.69[b]	9/16
2.70	5.41	3.83	7.67	5.34	10.68	7.10	14.02[b]	5/8
....	....	3.83	7.67	5.34	10.68	7.10	14.20	3/4
....	....	....	....	5.34	10.68	7.10	14.20	7/8
....	....	....	....	....	....	7.10	14.20	1

rivet is directed is not less than twice the nominal rivet diameter (see Specification G-1, ex-
[c]These values are governed by bearing. All other values are governed by basic allowable shear

G-8. The gage at right angles to the direction of stress shall not exceed twenty times the thickness of the outside plates. In tension members the maximum pitch or gage of such rivets shall be twenty times the thickness of the outside plates; and in tension members composed of two angles in contact, the pitch of the stitch rivets shall not exceed 10 in.

G-10. Minimum Spacing of Rivets. The distance between centers of rivets shall not be less than three times the diameter of the rivets.

G-11. Edge Distance of Rivets. The distance from the center of a rivet to a sheared, sawed, rolled, or planed edge shall be not less than one and one-half times the diameter, except in flanges of beams and channels, where the minimum distance may be one and one-fourth times the diameter. For rivets under computed stress, the distance from the center of the rivet to the edge of the plate or shape toward which the pressure of the rivet is directed should normally be at least twice the nominal diameter of the rivet. In cases where a shorter edge distance must be used, the allowable bearing stress shall be reduced in accordance with Specification *G-1*, exception *b*.

The distance from the edge of a plate to the nearest rivet line shall not exceed six times the thickness of the plate.

G-12. Sizes of Rivets in Angles. The diameter of the rivets in angles whose size is determined by calculated stress shall not exceed one fourth of the width of the leg in which they are driven. In angles whose size is not so determined, 1-in. rivets may be used in 3½-in. legs; ⅞-in. rivets, in 3-in. legs; and ¾-in. rivets, in 2½-in. legs.

G–13. Extra Rivets. If splice plates are not in direct contact with the parts which they connect, there shall be rivets on each side of the joint in excess of the number required in the case of direct contact, to the extent of two extra lines for each intervening plate.

If rivets carrying calculated stress pass through fillers, the fillers shall be extended beyond the connected member and the extension secured by enough additional rivets to distribute the total stress in the member uniformly over the combined section of the member and filler.

Section H. Miscellaneous Design Rules

H–1. Reversal of Load. Members subject to reversal of load under the passage of live load shall be proportioned as follows: Determine the tensile load and the compressive load and increase each by 50% of the smaller; then proportion the member and its connections so that the allowable stresses given in Sections A to G, inclusive, will not be exceeded by either increased load.

H–2. Slenderness Ratio of Tension Members. Long slender members have little resistance to lateral loads. Therefore, tension members with values of slenderness ratio L/r greater than 150 shall not be used unless special care is taken to insure that such members are designed to resist any lateral loads such as wind, dead load, or the weight of workmen and equipment. Stresses caused by the combined bending and tensile loadings shall not exceed allowable limits.

H–3. Stay Plates for Tension Members. Segments of tension members not directly connected to each other shall be stayed together. The length of the stay plate shall be not less than three fourths of the distance between rivet lines of the segments. Stay plates shall be connected to each segment of the tension member by at least three rivets. The distance between stay plates shall be such that the slenderness ratio of the individual segments of the member between stay plates does not exceed the slenderness ratio of the member as a whole.

H–4. Fatigue. Tests indicate that riveted members designed in accordance with the requirements of these specifications and constructed so as to be free from severe re-entrant corners and other unusual stress raisers will safely withstand at least 100,000 repetitions of maximum live load without fatigue failure regardless of the ratio of minimum to maximum load. Where a greater number of repetitions of some particular loading cycle is expected during the life of the structure, the calculated net section tensile stresses for the loading in question shall not exceed the values given by the curves in Fig. 10. When using the curves in Fig. 10 the reversal-of-load rule in Specification *H–1* should be ignored. The final member and connections selected, however, shall be strong enough to satisfy the requirements of Specification *H–1*.

In considering fatigue action on structures it is well to bear in mind the following points:

a. The most severe combination of loadings for which a structure is designed (dead load, maximum live load, maximum impact, maximum wind, etc.) rarely occurs in actual service and is of little or no interest from the standpoint of fatigue.

b. The loading of most interest from the fatigue standpoint is the steady dead load with a superimposed and repeatedly applied live load having an intensity consistent with day-to-day normal operating conditions.

c. The number of cycles of load encountered in structures is usually small compared with those encountered in fatigue problems involving machine parts. It takes many years of service to accumulate even 100,000 cycles of any significant stress application in most structures, as is indicated by the following examples: 100,000 cycles represent 10 cycles every day for 27 years; 10,000,000 cycles represent 20 cycles every hour for 57 years. Care must be taken not to overestimate grossly the number of cycles for any given load condition.

d. Careful attention to details in design and fabrication pays big dividends in fatigue life. When a fatigue failure occurs in a structure it is usually at a point of stress concentration where the state of stress could have been improved with little or no added expense.

Section I. Fabrication

I–1. Laying Out

a. Hole centers may be center-punched and cutoff lines may be punched or scribed. Center-punching and scribing shall not be used where such marks would remain on fabricated material.

b. A temperature correction shall be applied where necessary in the layout of critical dimensions. The coefficient of expansion shall be taken as 0.000012 per degree Fahrenheit.

I–2. Cutting

a. Material ½ in. thick or less may be sheared, sawed, or cut with a router. Material more than ½ in. thick shall be sawed or routed.

b. Cut edges shall be true and smooth, and free from excessive burrs or ragged breaks.

c. Edges of plates carrying calculated stresses shall be planed to a depth of ¼ in. except in the case of sawed or routed edges of a quality equivalent to a planed edge.

d. Re-entrant cuts shall be avoided wherever possible. If used they shall be filleted by drilling prior to cutting.

e. Flame cutting of aluminum alloys is not permitted.

I–3. Heating. Structural material shall not be heated, with the following exceptions:

a. Material may be heated to a temperature not exceeding 400° F for a period not exceeding 15 min. to facilitate bending. Such heating shall be done only when proper temperature controls and supervision are provided to insure that the limitations on temperature and time are carefully observed.

b. Hot-driven rivets shall be heated as specified in Section *I–5*.

I–4. Punching, Drilling, and Reaming. Rules for punching, drilling, and reaming are as follows:

a. Rivet or bolt holes in main members shall be subpunched or subdrilled and reamed to finished size after the parts are firmly bolted together. The amount by *which the diameter of a subpunched hole is smaller than that of the finished hole shall be at least one-quarter the thickness of the piece and in no case less than 1⁄32 in. If the metal thickness is greater than the diameter of the hole, punching shall not be used.*

b. Rivet or bolt holes in secondary material not carrying calculated stress may be punched or drilled to finished size before assembly.

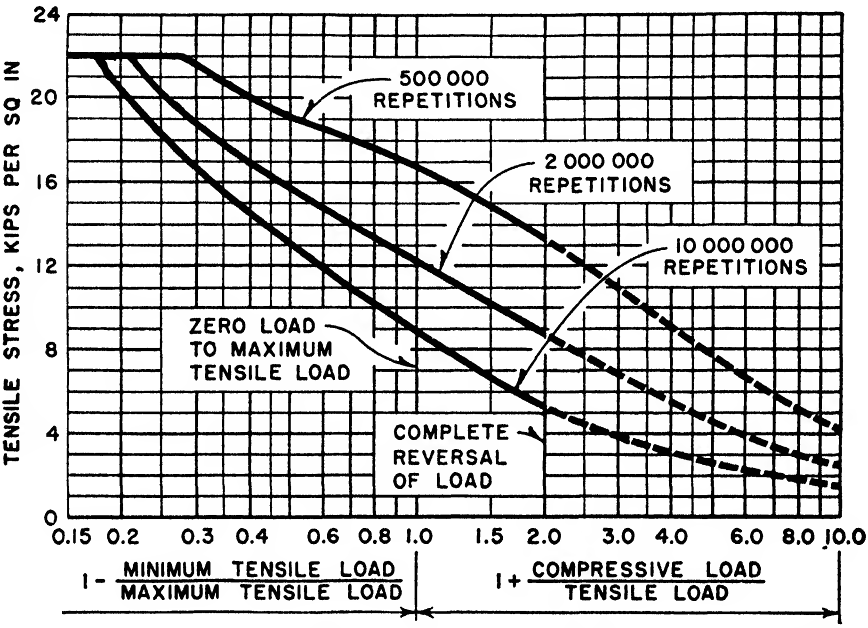


FIG. 10. Allowable tensile stresses on net section for various numbers of repetitions of load application.

c. The finished diameter of holes for cold-driven rivets shall be not more than 4% greater than the nominal diameter of the rivet.

d. The finished diameter of holes for hot-driven rivets shall be not more than 7% greater than the nominal diameter of the rivet.

e. The finished diameter of holes for unfinished bolts shall be not more than 1⁄16 in. larger than the nominal bolt diameter.

f. Holes for turned bolts shall be drilled or reamed to give a driving fit.

g. All holes shall be cylindrical and perpendicular to the principal surface. Holes shall not be drifted in such a manner as to distort the metal. All chips lodged between contacting surfaces shall be removed before assembly.

I-5. Riveting

a. The driven head of aluminum alloy rivets preferably shall be of the flat or the cone-point type, with dimensions as follows:

1) Flat heads shall have a diameter not less than 1.4 times the nominal rivet diameter and a height not less than 0.4 times the nominal rivet diameter.

2) Cone-point heads shall have a diameter not less than 1.4 times the nominal

rivet diameter and a height, to the apex of the cone, not less than 0.65 times the nominal rivet diameter. The included angle at the apex of the cone shall be approximately 127°.

b. Rivets shall be driven hot or cold as called for on the plans, provision for heating as follows:

1) Hot-driven rivets shall be treated in a hot air type furnace providing uniform temperatures throughout the rivet chamber and equipped with automatic temperature controls.

2) Hot-driven rivets shall be held at from 990° F to 1,050° F for not less than 15 min. and for not more than 1 hour before driving.

3) Hot rivets shall be transferred from the furnace to the work and driven with a minimum loss of time.

c. Rivets shall fill the holes completely. Rivet heads shall be concentric with the rivet holes and shall be in proper contact with the surface of the metal.

d. Defective rivets shall be removed by drilling.

I–6. Welding. Welding is not permitted.

I–7. Cleaning and Treatment of Metal Surfaces

a. Surfaces of metal shall be cleaned immediately before painting by a method which will remove all dirt, oil, grease, chips, and other foreign substances.

b. Either of the two following methods of cleaning may be used on exposed metal surfaces:

1) Chemical Cleaning. Parts may be immersed in, or swabbed with, a solution of phosphoric acid and organic solvents diluted with water in the ratio of 1:3. The solution temperature shall be between 50° F and 90° F. The solution shall remain in contact with the metal not less than 5 min. Residual solution shall be removed with clear water.

2) Sandblasting. Standard mild sandblasting methods may be used on sections more than ⅛ in. thick.

c. For contacting surfaces only, the metal may be cleaned in accordance with Specification *I–7b*, or with a solvent such as mineral spirits or benzine.

d. Flame cleaning is not permitted.

I–8. Painting. Specifications to control painting operations are as follows: Metal parts shall be painted as described in Specifications *I–8a* and *I–8b* (or in accordance with United States Military Specification MIL-T-704) except where the plans specifically permit a deviation.

a. Where the aluminum alloy parts are in contact with, or are fastened to, steel members or other dissimilar materials, the aluminum shall be kept from direct contact with the steel or other dissimilar material by painting as described below:

1) Aluminum surfaces to be placed in contact with steel shall be given one coat of zinc chromate primer in accordance with United States Joint Army-Navy Specification JAN-P-735 or the equivalent. Zinc chromate paint shall be allowed to dry before assembly of the parts. Additional protection may be provided by the application of one coat of a suitable nonhardening joint compound, in addition to the zinc chromate primer. The steel surfaces to be placed in contact with aluminum shall be painted with good quality priming paint, such as red lead conforming to Federal Specification TT-P-86a, followed by one coat of paint consisting of 2 lb of aluminum

paste pigment (ASTM Specification D962-49, Type II, Class B) per gallon of varnish meeting Federal Specification TT-V-81b, Type II, or the equivalent.

2) Aluminum surfaces to be placed in contact with wood, concrete, or masonry construction, except where the aluminum is to be imbedded in concrete, shall be given a heavy coat of an alkali-resistant bituminous paint before installation. The bituminous paint used shall meet the requirements of United States Military Specification MIL-P-6883. The paint shall be applied as it is received from the manufacturer without the addition of any thinner.

3) Aluminum surfaces to be imbedded in concrete shall, before installation, be given one coat of zinc chromate primer in accordance with United States Joint Army-Navy Specification JAN-P-735 or the equivalent. The paint shall be allowed to dry before the aluminum is placed in the concrete.

b. All contacting metal surfaces other than those covered in Specification *I-8a* shall be painted before assembly with one coat of zinc chromate primer in accordance with United States Joint Army-Navy Specification JAN-P-735 or the equivalent. The primer shall be allowed to dry before assembly of the parts. All other surfaces shall be given one shop coat of zinc chromate primer made in accordance with United States Joint Army-Navy Specification JAN-P-735 or the equivalent and then shall be given a second shop coat of paint consisting of 2 lb of aluminum paste pigment (ASTM Specification D962-49, Type II, Class B) per gallon of varnish meeting Federal Specification TT-V-81b, Type II, or the equivalent. Sufficient Prussian blue shall be added to permit detection of an incomplete application of the subsequent paint coat. After erection, bare spots shall be touched up with zinc chromate primer followed by a touch-up coat of aluminum paint, as specified above. The completed structure shall be finished according to one of the following methods:

1) One field coat of aluminum paint as specified above, except that Prussian blue shall be omitted from the field coat.

2) One or more field coats of alkyd base enamel pigmented to meet a desired color scheme.

PART III. EXPLANATION OF SPECIFICATIONS

Section A. Summary of Allowable Stresses

A-1. Basic Tensile Design Stress. The basic tensile design stress of 22 kips per sq in. represents a factor of safety of 2.41 based on the specified tensile yield strength. This is a larger factor of safety with respect to yield strength than is ordinarily encountered in specifications for structural steel. In selecting this rather large factor of safety on yield strength, the committee was influenced to a considerable extent by the fact that there is a smaller spread between yield strength and tensile strength in this aluminum alloy than is commonly encountered in structural steels.

A-8, A-9, and A-13. Allowable Stresses on Rivets. The allowable shearing and bearing stresses on rivets were selected on the basis of the results of numerous shearing and bearing tests.[2] The factors of safety used are greater than those used for most of the other allowable stresses.

A-6, A-11, and A-12. Allowable Stresses on Pins. The allowable bending, shearing, and bearing stresses on pins were selected to have about the same relation to the corresponding properties of the material as is the case in standard steel specifications. It is not anticipated that any wide use of pins will be made in aluminum alloy structures, but it is assumed that where they are used, they will be of the same material as the structural members themselves, and that they would probably be obtained in the form of rolled rod, ASTM Specification B211-54T(CS41A).

Section B. Column Design

B-1. Curves for Allowable Compressive Stresses in Axially Loaded Columns. The curves in Fig. 2 are the tangent-modulus column curves with a factor of safety of 2.5 and with a cutoff at the basic allowable design stress of 22 kips per sq in. The formulas for all three curves can be written[4, 5]

$$fc = \frac{\pi^2 E_t}{2.5\left(\frac{k\,L}{r}\right)^2} \tag{7}$$

in which:

fc is the allowable compressive stress on the gross cross-sectional area, in kips per square inch;

E_t is the tangent modulus taken from Fig. 1 at stress corresponding to 2.5 (fc), in kips per square inch;

L is the length of the column, in inches;

r is the least radius of gyration of the column, in inches; and

k is a factor describing the end conditions as defined in Fig. 2.

For values of slenderness ratio, L/r, greater than 72, the formula for the partial restraint curve in Fig. 2 reduces to

$$fc = \frac{74{,}000}{\left(\frac{L}{r}\right)^2} \tag{8}$$

B-7. Formula for Combined Compression and Bending. Eq. 1 is based on an interaction formula that gives good agreement with the results of tests on aluminum alloy members subjected to combined compressive end load and bending.[6, 7] The

[4] "Column Strength of Various Aluminum Alloys," by R. L. Templin, R. G. Sturm, E. C. Hartmann, and M. Holt, *Aluminum Research Laboratories Technical Paper No. 1*, Aluminum Co. of America, Pittsburgh, Pa., 1938.

[5] "Inelastic Column Theory," by F. R. Shanley, *Journal of the Aeronautical Sciences*, Vol. 14, 1947, pp. 261–268.

[6] "Plastic Buckling of Eccentrically Loaded Aluminum Alloy Columns," by J. W. Clark, *Proceedings*, ASCE, Separate No. 299, Vol. 79, October, 1953.

[7] "Designing Aluminum Alloy Members for Combined End Load and Bending," by H. N. Hill, E. C. Hartmann, and J. W. Clark, *Proceedings*, ASCE, Separate No. 300, Vol. 79, October, 1953.

formula is applicable to members in which the bending is applied about either the strong or the weak axis.

B–8. Formula for Transverse Shear on Columns. Eq. 3a is based on the transverse component of the column load at the point of maximum slope of the column in its deflected position. A derivation by Mr. Hartmann has been published elsewhere.[8]

Section C. Curve for Allowable Compressive Stress in Beam and Girder Flanges

C–1. The curve in Fig. 3 is based on the theoretical solution for the critical bending moment in I-beams as given by S. Timoshenko.[9] It represents a factor of safety of 2.5 applied to the buckling strength of beams subjected to a uniform bending moment. It is assumed that at the ends of the laterally unsupported length there is partial restraint against rotation about a vertical axis and complete restraint against lateral displacement and against rotation about a horizontal axis parallel to the web. The effect of partial restraint at the ends of the span has been taken into account by substituting $0.75L$ for the length in the formula for buckling of the beam under uniform bending moment.

The part of the curve for values of $\frac{L}{\sqrt{B/S_c}}$ greater than 35 is based on elastic action, whereas the remainder is simply an extension of the same formula using tangent modulus rather than initial modulus. The curve has a cutoff at the basic allowable compressive design stress of 14 kips per sq in. It is important to note that the term L is defined as "laterally unsupported length," which is not necessarily the same as the span of the beam or girder.

The case of uniform bending moment on a simple beam has been used in setting up Fig. 3, because it is a good approximation of conditions frequently encountered in actual design, and because it is somewhat more conservative than many of the other cases that might have been selected. The provision that four-thirds times the length shall be used for L in the case of a cantilever beam has the effect of canceling the length reduction coefficient of 0.75 that was introduced to take account of the effect of lateral restraint in simple beams. Thus, the buckling strength of a cantilever beam is assumed to be equal to the strength of a simply supported beam of the same length subjected to uniform bending moment. This gives a conservative approximation to the strength of a cantilever beam.[9]

For values of $\frac{L}{\sqrt{B/S_c}}$ greater than 27.5, the curve in Fig. 3 may be represented by the formula:

$$f_B = \frac{10{,}900}{\left(\frac{L}{\sqrt{B/S_c}}\right)^2} \tag{9}$$

[8] Discussion by E. C. Hartmann of "Rational Design of Steel Columns," by D. H. Young, *Transactions*, ASCE, Vol. 101, 1936, pp. 475–481.

[9] *Theory of Elastic Stability*, by S. Timoshenko, McGraw-Hill Book Co., Inc., New York, N. Y., 1936.

The curve in Fig. 3 is based on a theoretical solution applicable only to I-beams having cross sections symmetrical about both axes. The modified interpretation of the term I_y, indicated in specification *C–1* [Part III], however, permits the curve to be used without serious error for beams and girders having one flange differing in lateral stiffness from the other. It should be used with caution in cases of beams and girders which are unsymmetrical by a considerable margin. (In connection with this subject several supplementary references[7, 10, 11, 12, 13] will be of interest.)

Section D. Curves for Design of Flat Plates, Legs, and Webs

The curves of Figs. 4, 5, and 6 are based on values of critical stress compiled by Mr. Hill in 1940.[14] Partial restraint along the supported edges and loaded edges was assumed in all cases except for the supported edge in Fig. 4, which was considered simply supported. Parts of the curves that represent critical buckling stresses above the elastic range are computed by using the tangent modulus instead of the modulus of elasticity, a procedure which is known to be conservative when applied to problems of plate buckling.[15] A factor of safety of 2.5 against critical buckling has been used in all three charts, and in all cases the curves have a cutoff at the basic allowable design stress of 22 kips per sq in.

When a flat plate, leg, or web is built in along one or both edges to other parts of a compression member which offer partial edge restraint, the local buckling of the plate, leg, or web does not precipitate collapse of the member as a whole as it probably would in the case of a single-angle strut. For this reason it is proper to permit a decreased factor of safety against local buckling in such cases if suitable precautions are taken to avoid collapse. Step *c* of Specification *D–2* provides a simple method for accomplishing this result by introducing the well-known "effective width" concept. After a plate, leg, or web buckles, a part of its area is considered to be ineffective in supporting load, whereas a strip along each supported edge is considered still fully effective in working with the supporting material to which it is attached. The formula (Eq. 4) for effective width used in step *c* of Specification *D–2* is generally more conservative than other accepted methods of calculating effective width.[16, 17, 18, 19]

[10] "The Lateral Instability of Unsymmetrical I-Beams," by H. N. Hill, *Journal of the Aeronautical Sciences*, Vol. 9, 1942, pp. 175–180.

[11] "The Lateral Stability of Equal-Flanged Aluminum-Alloy I-Beams Subjected to Pure Bending," by C. Dumont and H. N. Hill, *Technical Note No. 770*, National Advisory Committee for Aeronautics, Washington, D. C., 1940.

[12] "Lateral Stability of Unsymmetrical I-Beams and Trusses in Bending," by George Winter, *Transactions*, ASCE, Vol. 108, 1943, pp. 247–268.

[13] "Strength of Beams as Determined by Lateral Buckling," by Karl de Vries, *ibid.*, Vol. 112, 1947, pp. 1245–1320.

[14] "Chart for Critical Compressive Stress of Flat Rectangular Plates," by H. N. Hill, *Technical Note No. 773*, National Advisory Committee for Aeronautics, Washington, D. C., 1940.

[15] "Buckling Stresses for Flat Plates and Sections," by Elbridge Z. Stowell, George J. Heimerl, Charles Libove, and Eugene E. Lundquist, *Transactions*, ASCE, Vol. 117, 1952, pp. 545–575.

[16] "The Strength of Thin Plates in Compression," by Theodor von Kármán, Ernest E. Sechler, and L. H. Donnell, *Transactions*, A.S.M.E., Vol. 54, 1932, pp. 53–57.

The limitation placed on the value of f_c in step *f* of Specification *D–2* is intended to provide a factor of safety of at least 1.67 against local buckling at the design load.

Section E. Curves for Allowable Shear Stress in Webs

E–1. The values of allowable stress in Fig. 7 are obtained by applying a factor of safety of 2 to the critical shear buckling stresses for flat plates with the edges about halfway between the fixed and hinged conditions.[9, 20, 21]

Those parts of the curves of Fig. 7 which represent critical buckling stresses above the elastic stress range are computed from formulas for elastic buckling, with the tangent modulus substituted for the modulus of elasticity. For a given value of critical shear stress, the tangent modulus is that corresponding to an axial stress equal to $\sqrt{3}$ times the shear stress.[22] As in the case of compressive buckling of flat plates, the tangent modulus is conservative.

For values of allowable stress below 10.4 kips per sq in., the curves of Fig. 7 may be represented by the formula:

$$f_v = \frac{35{,}000}{(b/t)^2}\left[1 + 0.75\left(\frac{b}{a}\right)^2\right] \tag{10}$$

Section F. Plate Girder Design

F–6. Curves for Allowable Longitudinal Compressive Stress in Webs of Girders. The curve in Fig. 8 for girders with no horizontal stiffeners is based on the critical buckling stress for rectangular flat plates under pure bending in the plane of the plate. Partial restraint is assumed at the toes of the flanges (about halfway between the solution given by Mr. Timoshenko for the case of a plate simply supported on all four edges[9] and the solution of K. Nolke for a plate with the loaded edges simply supported and the other two edges fixed).[23]

The curve in Fig. 8 for girders with a single horizontal stiffener is based on the critical buckling stress given by Mr. Timoshenko for plates simply supported on all four edges under combined bending and axial stress in the plane of the plate.[9] The simple support condition is used for this case because the horizontal stiffener would

[17] "The Apparent Width of the Plate in Compression," by Karl Marguerre, *Technical Memorandum No. 833*, National Advisory Committee for Aeronautics, Washington, D. C., 1937.

[18] "Strength of Thin Steel Compression Flanges," by George Winter, *Transactions*, ASCE, Vol. 112, 1947, pp. 527–576.

[19] "Performance of Thin Steel Compression Flanges," by George Winter, preliminary publication, 3d Cong. of the International Assn. for Bridge and Structural Engrs., Liege, Belgium, 1948.

[20] *Formulas for Stress and Strain*, by Raymond J. Roark, McGraw-Hill Book Co., Inc., New York, N. Y., 1938.

[21] "Observations on the Behavior of Aluminum Alloy Test Girders," by R. L. Moore, *Transactions*, ASCE, Vol. 112, 1947, pp. 901–920.

[22] "Critical Shear Stress of an Infinitely Long Plate in the Plastic Region," by Elbridge Z. Stowell, *Technical Note No. 1681*, National Advisory Committee for Aeronautics, Washington, D. C., 1948.

[23] "Buckling of Webs in Deep Steel I-Girders," by Georg Wastlund and Sten G. A. Bergman, rept. of investigation made at the Royal Inst. of Technology, Stockholm, Sweden, 1947.

provide comparatively little restraint against rotation. The location of the horizontal stiffener shown in the sketch in Fig. 8 is chosen so that the parts of the plate above and below the stiffener will buckle at approximately the same load.

A factor of safety against buckling of 1.5 was used for the curves of Fig. 8. Although this factor of safety is not as large as some used elsewhere in these specifications, it is considered adequate in this instance since tests have shown that the critical bending stress for girder webs may be considerably exceeded without affecting the load-carrying capacity of the girder.[21, 23] Use of Fig. 8, however, will prevent buckling from occurring at design stresses.

The curves of Fig. 8 may be represented by the following formulas: No horizontal stiffener—

$$f_a = \frac{200{,}000}{\left(\frac{h}{t}\right)^2} \tag{11}$$

and single horizontal stiffener—

$$f_a = \frac{800{,}000}{\left(\frac{h}{t}\right)^2} \tag{12}$$

The curves are cut off at the basic allowable design stress of 22 kips per sq in.

F–8 and F–9. Curves for Spacing and Moment of Inertia of Vertical Stiffeners. The curves for determining stiffener spacing, in Fig. 9, are merely replots of the data of Fig. 7. The curves of I_s/t^4 in Fig. 9 represent the following formula:

$$I_s = 8 \times 10^{-5} \frac{f_s\, h^3\, t \left(\frac{s}{h}\right)}{1 + 5\left(\frac{s}{h}\right)^3} \tag{13}$$

in which f_s is the average shear stress on the web in kips per square inch.

Eq. 13 is designed to fit the theoretical solution of M. Stein and R. W. Fralich[24] for values of s/h between 0.2 and 1.0. This solution does not cover values of s/h greater than 1.0. In this range, however, Eq. 13 is conservative in comparison with the recommendations of L. S. Moisseiff.[1]

F–10. Formula for Moment of Inertia of Stiffeners at Points of Bearing. Eq. 5 simply states that the moment of inertia of a stiffener at a point of bearing should be equal to the sum of the moment of inertia required to resist the tendency of the web to buckle and the moment of inertia required for the stiffener to carry the bearing load as a column with length equal to the height of the web.

F–11. Formula for Radius of Gyration of Horizontal Stiffeners. Eq. 6, for the moment of inertia of horizontal stiffeners, is based on the theoretical work of C. Dubas, reported by F. Bleich.[25]

[24] "Critical Shear Stress of Infinitely Long, Simply Supported Plate with Transverse Stiffeners," by Manuel Stein and Robert W. Fralich, *Technical Note No. 1851*, National Advisory Committee for Aeronautics, Washington, D. C., 1949.

[25] *The Buckling Strength of Metal Structures*, by Friedrich Bleich, McGraw-Hill Book Co., Inc., New York, N. Y., 1952, p. 422.

Section H. Miscellaneous Design Rules

H-4. Curves of Allowable Tensile Stress on Net Section for Various Numbers of Repetitions of Load Application. The curves in Fig. 10 are plotted from the results of fatigue tests conducted at the Aluminum Research Laboratories of the Aluminum Company of America at New Kensignton, Pa., on 2014-T6 butt joints with double straps joined with eight cold-driven ⅝-in. 2117-T6 rivets. The type of testing equipment and specimen (Type M1) used are illustrated in a paper by R. L. Templin,[26] M. ASCE, in 1939, and a paper by Mr. Hartmann, J. O. Lyst, and H. J. Andrews,[27] Jun. ASCE, in 1944.

A factor of safety of 1.2 has been applied to the test data and all curves are cut off at the basic allowable design stress of 22 kips per sq in. The right-hand part of the diagram is largely based on extrapolation of the data, but this is not considered to be a serious matter since the design of most members in this range will be governed primarily by Specification *H*-1 rather than by fatigue considerations.

Respectfully submitted,

F. Baron	E. H. Gaylord	W. A. Nash
P. E. Brandt	E. C. Hartmann	F. L. Plummer
R. Ebenbach	S. A. Kilpatric	W. K. Rey
C. N. Gaylord	R. B. B. Moorman	J. B. Scalzi
		F. J. Tamanini

J. W. Clark, Chairman

Committee of the Structural Division on Design in Lightweight Structural Alloys

[26] "Fatigue Machines for Testing Structural Units," by R. L. Templin, *Proceedings*, ASTM, Vol. 39, 1939, pp. 711–722.

[27] "Fatigue Tests of Riveted Joints," by E. C. Hartmann, J. O. Lyst, and H. J. Andrews, *Wartime Report W55*, National Advisory Committee for Aeronautics, Washington, D. C., 1944.

THE ALUMINUM ASSOCIATION ALLOY DESIGNATION SYSTEM FOR WROUGHT ALUMINUM

EFFECTIVE OCTOBER 1, 1954

Wrought aluminum and wrought aluminum alloys are designated by a four-digit index system. The first digit of the designation serves to indicate alloy groups. The last two digits identify the aluminum alloy or indicate the aluminum purity. The second digit indicates modifications of the original alloy or impurity limits.

Aluminum and Aluminum Alloy Groups

In the four-digit index system the first digit indicates the alloy group as shown in Table I. Thus 1xxx indicates aluminum of 99.00% minimum and greater, 2xxx indicates an aluminum alloy in which copper is the major alloying element, and 3xxx an aluminum alloy with manganese as the major alloying element, etc. Although most aluminum alloys contain several alloying elements, only one group—6xxx for alloys with magnesium and silicon as major alloying elements—designates more than one alloying element.

Aluminum

In the 1xxx group for aluminum of 99.00% minimum and greater, the last two of the four digits in the designation indicate the minimum aluminum percentage. These digits are the same as the two digits to the right of the decimal point in the minimum aluminum percentage when it is expressed to the nearest 0.01%.

The second digit in the designation indicates modifications in impurity limits. If the second digit in the designation is zero, it indicates that there is no special control on individual impurities; while integers 1 through 9, which are assigned consecutively as needed, indicate special control of one or more individual impurities. Thus 1030 indicates 99.30% minimum aluminum without special control on individual impurities and 1130, 1230, 1330, etc., indicate the same purity with special control on one or more impurities. Likewise, 1075, 1175, 1275, etc., indicate 99.75% minimum aluminum; and 1097, 1197, 1297, etc., indicate 99.97%.

Aluminum Alloys

In the 2xxx through 8xxx alloy groups the last two of the four digits in the designation have no special significance but serve only to identify the different alloys in the group. Generally these digits are the same as those formerly used to designate the same alloy. Thus 2014 was formerly 14S, 3003 was 3S, and 7075 was 75S. For new alloys these last two digits are assigned consecutively beginning with xx01.

The second digit in the alloy designation indicates alloy modifications. If the second digit in the designation is zero, it indicates the original alloy; while integers 1 through 9, which are assigned consecutively, indicate alloy modifications. In the former system, letters were used to designate alloy modifications. These were assigned consecutively beginning with A. Thus 17S is now 2017 and A17S is 2117, 18S is 2018 and B18S is 2218.

TABLE I. Designations for Alloy Groups

		AA Number
Aluminum—99.00% minimum and greater		1xxx
	Major Alloying Element	
Aluminum Alloys grouped by major Alloying Elements	Copper	2xxx
	Manganese	3xxx
	Silicon	4xxx
	Magnesium	5xxx
	Magnesium and silicon	6xxx
	Zinc	7xxx
	Other element	8xxx
Unused Series		9xxx

TABLE II. Aluminum Alloy Designation Conversions

OLD Commercial Designation	NEW AA Number	OLD Commercial Designation	NEW AA Number	OLD Commercial Designation	NEW AA Number
99.3[1]	1230	17S	2017	56S	5056
99.6, CD1S	1160	A17S	2117	XC56S	X5356
99.75[2]	1175	18S	2018	C57S, K157	5357
99.87, EB1S	1187	B18S	2218	61S	6061
EC[3]	EC	F18S	2618	62S	6062
AA1S	1095	24S	2024	63S	6063
BA1S	1099	25S	2025	66S	6066
CA1S	1197	B25S	2225	70S	7070
AB1S	1085	32S	4032	72S	7072
EB1S, 99.87	1187	43S, K145	4043	75S	7075
FB1S	1090	C43S, 44S, K143	4343	B77S	7277
AC1S	1070	XE43S	X4543	XA78S	X7178
BC1S	1080	44S, C43S, K143	4343	XB80S	X8280
CC1S, R998	1180	45S	4045	K112	8112
JC1S	1075	50S	5050	K143, C43S, 44S	4343
AD1S	1050	A50S, K155, R305	5005	K145, 43S	4043
BD1S	1060	XD50S	X5405	K155, A50S, R305	5005
CD1S, 99.6	1160	A51S	6151	K157, C57S	5357
ED1S	1150	XB51S	X6251	K160, J51S	6951
AE1S	1030	J51S, K160	6951	K162, R306[4]	6003
BE1S	1145	52S	5052	LK183	5083
2S	1100	F52S	5652	K186	5086
3S	3003	53S	6053	R301 Core, 14S	2014
4S	3004	B53S	6253	R305, K155, A50S	5005
XA5S	X3005	XD53S	X6453	R306, K162[4]	6003
11S	2011	E53S	6553	R308[5]	1130
14S, R301 Core	2014	A54S	5154	R399	8099
XB14S	X2214	B54S	5254	R995	1235
XC16S	X2316	X55S	X5055	R998, CC1S	1180

[1] *Cladding on Alclad 24S (Alclad 2024).* [2] *Cladding on No. 2 Reflector Sheet.*
[3] *EC—The designation for electrical conductor metal is not being changed since it is so firmly established in the electrical industry.*
[4] *Cladding on R301 and Alclad 14S (Alclad 2014).* [5] *No. 1 Reflector Sheet.*

Experimental Alloys

Experimental alloys are also designated in accordance with this system, but they are indicated by the prefix X. The prefix is dropped when the alloy becomes standard. During development, and before they are designated as experimental, new alloys are identified by serial numbers assigned by their originators. Use of the serial number is discontinued when the X number is assigned.

Temper Designations

The temper designation system in effect since December 31, 1947 is being continued without change. The temper designation follows the alloy designation and is separated from it by a dash. Thus 3S-0 is now 3003-0, Alclad 24S-T81 is Alclad 2024-T81, and 75S-T6 is 7075-T6.

TABLE III. ALUMINUM ALLOY DESIGNATION CONVERSIONS

NEW	OLD	NEW	OLD	NEW	OLD
AA Number	Commercial Designation	AA Number	Commercial Designation	AA Number	Commercial Designation
EC[1]	EC	2018	18S	5254	B54S
1030	AE1S	2024	24S	X5356	XC56S
1050	AD1S	2025	25S	5357	C57S, K157
1060	BD1S	2117	A17S	X5405	XD50S
1070	AC1S	X2214	XB14S	5652	F52S
1075	JC1S	2218	B18S	6003[5]	R306, K162
1080	BC1S	2225	B25S	6053	53S
1085	AB1S	X2316	XC16S	6061	61S
1090	FB1S	2618	F18S	6062	62S
1095	AA1S	3003	3S	6063	63S
1099	BA1S	3004	4S	6066	66S
1100	2S	X3005	XA5S	6151	A51S
1130[2]	R308	4032	32S	X6251	XB51S
1145	BE1S	4043	43S, K145	6253	B53S
1150	ED1S	4045	45S	X6453	XD53S
1160	CD1S, 99.6	4343	C43S, 44S, K143	6553	E53S
1175[3]	99.75	X4543	XE43S	6951	J51S, K160
1180	CC1S, R998	5005	A50S, R305, K155	7070	70S
1187	EB1S, 99.87	5050	50S	7072	72S
1197	CA1S	5052	52S	7075	75S
1230[4]	99.3	X5055	X55S	X7178	XA78S
1235	R995	5056	56S	7277	B77S
2011	11S	5083	LK183	8099	R399
2014	14S, R301 Core	5086	K186	8112	K112
2017	17S	5154	A54S	X8280	XB80S

[1] *EC—The designation for electrical conductor metal is not being changed since it is so firmly established in the electrical industry.*
[2] *No. 1 Reflector Sheet.* [3] *Cladding on No. 2 Reflector Sheet.*
[4] *Cladding on Alclad 2024 (Alclad 24S).* [5] *Cladding on Alclad 2014 (R301 and Alclad 14S).*

APPENDIX F

AMERICAN IRON AND STEEL INSTITUTE

SPECIFICATION FOR THE DESIGN OF LIGHT GAGE COLD-FORMED STEEL STRUCTURAL MEMBERS

1956 Edition

SECTION 1. GENERAL

1.1. Scope

This Specification shall apply to the design of structural members cold-formed to shape from sheet or strip steel less than $\frac{3}{16}$ inch thick and used for load-carrying purposes in buildings.

Nothing herein is intended to conflict with provisions of the Specifications issued by the American Institute of Steel Construction for the Design, Fabrication and Erection of Structural Steel for Buildings nor with the Standard Specifications for Open-Web Steel Joist Construction as issued by the Steel Joist Institute.

1.2. Material

Except as otherwise provided herein, steel shall conform to the following Standard Specifications of the American Society for Testing Materials, as amended to date:

Heavy Gage Structural Quality Flat Hot-Rolled Carbon Steel Sheets, ASTM designation: A245

Light Gage Structural Quality Flat Rolled Carbon Steel Sheets, ASTM designation: A246

Hot-Rolled Carbon Steel Strip of Structural Quality, ASTM designation: A303

The terms C, B, and A when used herein to designate grades of steel refer to grades provided by the above-listed ASTM Specifications.

Steel of higher strength than is covered by the above-mentioned ASTM specifications may be used at the unit stresses hereinafter specified for "other" grades of steel, provided the design is based upon the published minimum properties of such higher strength steel as certified by the manufacturer's test reports. It is the intent of this Specification to permit the use of special high-strength steels of suitable properties for purposes coming within the scope of this Specification, but not to permit the use of carbon steels at unit stresses higher than those specified in Section 3 for Grade C material unless the steel is produced to, and identified with, a published specification which specifically establishes its qualifications and properties.

SECTION 2. DESIGN PROCEDURE

2.1. Procedure

All computations for safe load, stress, deflection, and the like shall be in accordance with conventional methods of structural design except as otherwise specified herein.

2.2. Definitions

Where the following terms appear in this Specification they shall have the meaning herein indicated:

(a) *Stiffened Compression Elements.* The term *stiffened compression elements* shall mean flat compression elements (i.e., plane compression flanges of flexural members and plane webs and flanges of compression members) of which *both* edges parallel to the direction of stress are stiffened by connection to a stiffening means (i.e., web, flange, stiffening lip, intermediate stiffener, or the like) conforming to the requirements of Sec. 2.3.2.*

(b) *Unstiffened Compression Elements.* Any flat element which is stiffened at only *one* edge parallel to the direction of stress shall be considered an "unstiffened" element.

(c) *Multiple-Stiffened Elements.* A multiple-stiffened element is an element that is stiffened between webs, or between a web and an edge, by means of intermediate stiffeners which are parallel to the direction of stress and which conform to the requirements of Sec. 2.3.2.2. A *sub-element* is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.*

(d) *Flat-Width Ratio.* The flat-width ratio, w/t, is the ratio of the flat width, w, exclusive of edge fillets, of a single flat element to the thickness, t, of such element. In the case of sections such as **I**-, **T**-, channel- and **Z**-shaped sections, the width, w, is the width of the flat projection of flange from web, exclusive of fillets and of any stiffening lip that may be at the outer edge of the flange. In the case of *multiple-web* sections such as hat-, **U**- or box-shaped sections, the width, w, is the flat width of flange between adjacent webs, exclusive of fillets.

(e) *Effective Design Width.* Where the flat width, w, of an element is reduced for design purposes, the reduced design width, b, is termed the "effective width," or the "effective design width." "This effective design width" is determined in accordance with Sec. 2.3.1 and 2.3.5.*

2.3. Properties of Sections

Properties of sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design and shall be based on the full cross section of the members (or net section where the use of a net section is customary), except where the use of a reduced cross section, or "effective design width," is required by the provisions of Sec. 2.3.1 and 2.3.5 of this Specification.

2.3.1. Properties of Stiffened Compression Elements

Effective Design Width. In computing properties of sections of flexural members and in computing values of Q (Sec. 3.6.1) for compression members, the flat width, w, of any stiffened compression element having a flat-width ratio larger than $(w/t)_{\text{lim}}$ as hereinafter defined shall be considered as being reduced for design purposes to an *effective design width, b* or *b′*, determined in accordance with the provisions of Sec. 2.3.1.1 or 2.3.1.2, whichever is applicable, and subject to the limitations of Sec. 2.3.5 where applicable. That portion of the total width which is considered removed to arrive at the effective design width shall be located symmetrically about the center line of the element.†

* See sketches of "Effective Cross Sections," in Chart Nos. 1 and 2.

† See Charts Nos. 1 and 2 for stiffened elements and their effective cross sections. Charts 3A to 3D show effective design widths for stiffened elements.

TABLE 2.3.1.1.A

MAXIMUM WIDTH/THICKNESS RATIOS, $(w/t)_{lim}$, AT WHICH COMPRESSION ELEMENTS ARE FULLY EFFECTIVE $(b/t = w/t)$

Stress, f, psi	For Load Determination	For Deflection Determination
6,000	48.9	66.6
8,000	42.4	57.7
10,000	37.9	51.6
12,000	34.6	47.1
13,500	32.6	44.4
15,000	30.9	42.1
16,500	29.5	40.2
18,000	28.2	38.5
20,000	26.8	36.5
22,000	25.6	34.8
24,000	24.5	33.3
27,000	23.1	31.4
30,000	21.9	29.8
36,000	20.0	27.2
40,000	19.0	25.8
Other	$3790/\sqrt{f}$	$5160/\sqrt{f}$

TABLE 2.3.1.1.B

RATIO b/t OF EFFECTIVE DESIGN WIDTH TO THICKNESS OF STIFFENED COMPRESSION ELEMENTS, FOR $f = 18{,}000$ LB PER SQ IN.

w/t	b/t for Load Determination	b/t for Deflection Determination	w/t	b/t for Load Determination	b/t for Deflection Determination
28.2*	28.2	28.2	120	49.9	64.6
30	29.9	30.0	140	50.8	66.4
35	33.7	35.0	160	51.6	67.7
38.5**	35.8	38.5	180	52.1	68.7
40	36.5	39.9	200	52.6	69.5
45	38.8	44.0	225	53.0	70.3
50	40.5	47.3	250	53.4	71.0
55	42.0	50.0	275	53.7	71.5
60	43.2	52.3	300	53.9	72.0
70	45.1	55.8	350	54.3	72.7
80	46.6	58.4	400	54.6	73.2
90	47.7	60.5	450	54.8	73.6
100	48.6	62.1	500	55.0	74.0

* $(w/t)_{lim}$ for load determination; see Sec. 2.3.1.
** $(w/t)_{lim}$ for deflection determination; see Sec. 2.3.1.

2.3.1.1. Elements without Intermediate Stiffeners

The effective design widths of compression elements which are not subject to the provisions of Sec. 2.3.1.2 governing multiple-stiffened elements shall be determined from the following formulas:*

For load determination:

Flanges are fully effective ($b = w$) up to $\left(\frac{w}{t}\right)_{\text{lim}} = \frac{3790}{\sqrt{f}}$

(Table 2.3.1 A lists values of $\left(\frac{w}{t}\right)_{\text{lim}}$.)

For flanges with w/t larger than $(w/t)_{\text{lim}}$

$$\left(\frac{b}{t}\right) = \frac{7590}{\sqrt{f}}\left(1 - \frac{1900}{(w/t)\sqrt{f}}\right)$$

For deflection determination:

Flanges are fully effective up to $\left(\frac{w}{t}\right)_{\text{lim}} = \frac{5160}{\sqrt{f}}$

For flanges with w/t larger than $(w/t)_{\text{lim}}$

$$\left(\frac{b}{t}\right) = \frac{10320}{\sqrt{f}}\left(1 - \frac{2580}{(w/t)\sqrt{f}}\right)$$

In the above, w/t = flat-width ratio,
b = effective design width,
f = actual unit stress in the compression element computed on the basis of the effective design width.*

2.3.1.2. Multiple-Stiffened Elements

Where the flat-width ratio of a sub-element of a multiple-stiffened element does not exceed 60, the effective design width, b, of such sub-element shall be determined in accordance with the provisions of Sec. 2.3.1.1. Where such flat-width ratio exceeds 60, the effective design width, b', of the sub-element shall be determined from the following formula:

$$\frac{b'}{t} = \frac{b}{t} - 0.10\left(\frac{w}{t} - 60\right)$$

where

w/t = flat-width ratio of sub-element,
b = effective design width determined in accordance with the provisions of Sec. 2.3.1.1,
b' = effective design width of sub-element to be used in design computations.

For computing the effective structural properties of a member having multiple-stiffened elements, the area of any intermediate stiffener shall be considered reduced to an effective area as given by the expression which follows on page 624.

* The determination of effective design width may be facilitated by the use of Charts 3A to 3D, inclusive. Table 2.3.1.1.B shows effective design width for a unit stress of 18,000 psi. It is to be noted that where the flat-width ratio exceeds $(w/t)_{\text{lim}}$ the properties of the section must frequently be determined by successive approximations, since the unit stress and the effective design width are interdependent.

$$A_{\text{eff}} = A_{\text{full}}\,(b/w),$$

or

$$A_{\text{eff}} = A_{\text{full}}\,(b'/w),$$

whichever is applicable. In the above expressions, A_{eff} and A_{full} refer only to the area of the stiffener section, exclusive of any portion of adjacent elements.

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

2.3.2. Stiffeners for Compression Elements

2.3.2.1. Edge Stiffeners

In order that a flat compression element may be considered a "stiffened compression element," it shall be stiffened along each longitudinal edge parallel to the direction of stress by a web, lip, or other stiffening means, having the following minimum moment of inertia:

$$I_{\min} = (1.83t^4)\sqrt{(w/t)^2 - 144} \qquad \text{(See Table 2.3.2.1)}$$

where

w/t = flat-width ratio of stiffened element,
$I_{\min}$ = minimum allowable moment of inertia of stiffener (of any shape) about its own centroidal axis parallel to the stiffened element.

Where the stiffener consists of a simple lip bent at right angles to the stiffened element, the required over-all depth d of such lip may be determined with satisfactory accuracy by the following formula:

$$d = (2.8t)\sqrt[6]{(w/t)^2 - 144} \qquad \text{(See Table 2.3.2.1)}$$

A simple lip shall not be used as an edge stiffener for any element having a flat-width ratio greater than 60.

TABLE 2.3.2.1

MINIMUM PROPERTIES OF STIFFENERS FOR COMPRESSION ELEMENTS

w/t	I	d	w/t	I	d
12 or less	0	0	25	$40.2t^4$	$7.8t$
13	$9.2t^4$	$4.8t$	30	$50.4t^4$	$8.5t$
14	$13.2t^4$	$5.4t$	40	$69.9t^4$	$9.4t$
16	$19.4t^4$	$6.2t$	50	$89.0t^4$	$10.2t$
18	$24.6t^4$	$6.7t$	60	$107.6t^4$	$10.9t$
20	$29.3t^4$	$7.1t$	Over 60	$1.83t^3w$	

2.3.2.2. Intermediate Stiffeners

In order that a flat compression element may be considered a "multiple stiffened element," it shall be stiffened between webs, or between a web and an edge, by means of intermediate stiffeners parallel to the direction of stress, and the moment of inertia of each such stiffener shall be not less than twice the minimum allowable moment of inertia specified for edge stiffeners in Sec. 2.3.2.1. The following limitations also shall apply:

(a) If the spacing of stiffeners between two webs is such that the flat-width ratio of the sub-element between stiffeners is larger than w/t_{lim} (Sec. 2.3.1), only two intermediate stiffeners (those nearest each web) shall be considered effective.

(b) If the spacing of stiffeners between a web and an edge stiffener is such that the flat-width ratio of the sub-element between stiffeners is larger than w/t_{lim} (Sec. 2.3.1), only one intermediate stiffener shall be considered effective.

(c) If intermediate stiffeners are spaced so closely that the flat-width ratio between stiffeners does not exceed w/t_{lim} (Sec. 2.3.1), all the stiffeners may be considered effective. In computing the flat-width ratio of the entire multiple-stiffened element, such element shall be considered as replaced by an element without intermediate stiffeners whose width w_s is the *whole* width between webs or from web to edge stiffener, and whose equivalent thickness t_s is determined as follows:

$$t_s = \sqrt[3]{\frac{12\,I_s}{w_s}}$$

where I_s = moment of inertia of the full area of the multiple-stiffened element, including the intermediate stiffeners, about its own centroidal axis.

2.3.3. Maximum Allowable Flat-Width Ratios

Maximum allowable over-all flat-width ratios, w/t, disregarding intermediate stiffeners and taking as t the actual thickness of the element, shall be as follows:

(a) Stiffened compression element having *one* longitudinal edge connected to a web or flange element, the other stiffened by

A simple lip bent at right angle to the element. 60

Other stiffening means. 90

(b) Stiffened compression element with *both* longitudinal edges connected to a web or flange element (U-type or box-type sections). 500

(c) Unstiffened compression element. 60

Note: Unstiffened compression elements that have flat-width ratios exceeding approximately 30 and stiffened compression elements that have flat-width ratios exceeding approximately 250 are likely to develop noticeable deformation at the full allowable working stresses, without detriment to the ability of the member to carry design loads. Stiffened elements having flat-width ratios larger than 500 may be used with safety to support loads, but substantial deformation of such elements under load may occur and may render inapplicable the design formulas of this Specification.

(d) Unusually wide flanges: Where a flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

$$w_{\max} \sqrt{\frac{1{,}800{,}000th}{f_{\mathrm{av}}}} \times \sqrt[4]{\frac{100c}{h}}$$

where $w_{\max}$ = the width, in inches, of flange projecting beyond the web; or half of the distance between webs for box- or U-type beams,
t = thickness of flange in inches,
h = depth of beam in inches,
c = the amount of curling in inches.*
f_{av} = the *average* stress in the full, unreduced flange width. [Where members are designed by the *effective design width* procedure, the average stress = the maximum stress × (the ratio of the effective design width to the actual width).]

2.3.4. Maximum Allowable Web Depth

The ratio h/t of the webs of flexural members shall not exceed........... 150
where h = clear distance between flanges, in.,
t = thickness of web, in.,
where a web consists of two or more sheets, the h/t ratio of the individual sheets shall not exceed.. 150

2.3.5. Unusually Short Spans Supporting Concentrated Loads

Where the span of the beam is less than 30 w' (w' as defined below) and it carries one concentrated load, or several loads spaced farther apart than 2 w', the effective design width of any flange, whether in tension or compression, shall be limited to the following:

TABLE 2.3.5
SHORT, WIDE FLANGES
MAXIMUM ALLOWABLE RATIO OF EFFECTIVE DESIGN WIDTH TO ACTUAL WIDTH

L/w'	*Ratio*	*L/w'*	*Ratio*
30	1.00	14	0.82
25	0.96	12	0.78
20	0.91	10	0.73
18	0.89	8	0.67
16	0.86	6	0.55

In Table 2.3.5:
L = full span for simple spans; or the distance between inflection points for continuous beams; or twice the length of cantilever beams.
w' = width of flange projection beyond the web for I-beam and similar sections or half the distance between webs for box- or U-types sections.

For flanges of I-beams and similar sections stiffened by lips at the outer edges, w' shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.

* The amount of curling that can be tolerated will vary with different kinds of sections and must be established by the designer. Amount of curling in the order of 5 per cent of the depth of the section is usually not considered excessive.

SECTION 3. ALLOWABLE DESIGN STRESSES

The maximum allowable unit stresses to be used in design shall be as follows:

3.1. Basic Design Stress

Tension on the net section of tension members, and tension and compression, f_b, on the extreme fibers of flexural members shall not exceed the values specified below except as otherwise specifically provided herein.

Grade of Steel	Min. Yield Point, lb per sq in.	f_b lb per sq in.
C	33,000	18,000
B	30,000	16,500
A	25,000	13,500
Other	f_b = specified minimum yield point/1.85	

For special provisions for members resisting wind loads, see Sec. 3.8, Wind or Earthquake Stresses.

3.2. Compression on Unstiffened Elements

Compression, f_c, in pounds per square inch, on flat unstiffened elements:

(a) For w/t not greater than 12, $f_c = f_b$.
(b) For w/t greater than 12 but not over 30:

$$f_c = (1.67 f_b - 5430) - (1/18)(f_b - 8150)\, w/t.$$

[Values of f_c in accordance with formula are given in Table 3.2(b).]
(c) For w/t over 30 but not over 60*:

$$\text{Angle struts: } f_c = 7{,}330{,}000/(w/t)^2$$
$$\text{All other sections: } f_c = 12{,}600 - 148.5(w/t)$$

In the above, w/t = flat-width ratio as defined in Sec. 2.2.

TABLE 3.2 (b)

ALLOWABLE DESIGN STRESSES ON UNSTIFFENED ELEMENTS, SEC. 3.2(b); ASTM A245, A246, AND A303 GRADES OF STEEL.
FOR w/t RATIOS FROM 12 TO 30.

w/t	*Grade C*	*Grade B*	*Grade A*
12	18,000	16,500	13,500
14	16,910	15,580	12,910
16	15,810	14,650	12,310
18	14,720	13,720	11,720
20	13,630	12,790	11,130
22	12,530	11,860	10,530
24	11,440	10,940	9,940
26	10,340	10,010	9,340
28	9,250	9,080	8,750
30	8,150	8,150	8,150

* Unstiffened compression elements having ratios of w/t exceeding approximately 30 may show noticeable distortion of the free edges under allowable compressive stress without detriment to the ability of the member to support load.

For ratios of w/t exceeding approximately 60, distortion of the flanges is likely to be so pronounced as to render the section structurally undesirable unless load and stress are limited to such a degree as to render such use uneconomical.

TABLE 3.2(c)

ALLOWABLE DESIGN STRESSES ON UNSTIFFENED ELEMENTS, SEC. 3.2(c); FOR w/t RATIOS FROM 30 TO 60 (ALL GRADES OF STEEL).

Ratio w/t	Allowable f_c for		Ratio w/t	Allowable f_c for	
	Angle Struts	Other Sections		Angle Struts	Other Sections
30	8150	8150	46	3460	5770
32	7160	7850	48	3180	5470
34	6340	7550	50	2930	5180
36	5660	7250	52	2710	4880
38	5080	6960	54	2510	4580
40	4580	6660	56	2340	4280
42	4160	6360	58	2180	3990
44	3790	6070	60	2040	3690

3.3. Laterally Unbraced Single Web Beams

To prevent lateral buckling, the maximum compression f'_c, in pounds per square inch, on extreme fibers of compression flanges of laterally unsupported straight **I**-, **Z**-, or channel-shaped flexural members (not including multiple-web deck, **U**- and closed box-type members and curved or arch members) shall not exceed the allowable stress as specified in Sec. 3.1 or 3.2 nor maximum stresses as in Eqs. (a) and (b).

(a) For **I**- or channel-shaped sections

$$f'_c = \frac{250{,}000{,}000}{(L/r_y)^2}$$

(b) For **Z**-shaped sections

$$f'_c = \frac{125{,}000{,}000}{(L/r_y)^2}$$

where L is the unbraced length of the member, and r_y is the radius of gyration of the entire section of the member about its gravity axis parallel to the web.

3.4. Allowable Stresses in Webs of Beams

3.4.1. Shear Stresses in Webs

The maximum average shear stress, v, in pounds per square inch, on the gross area of a flat web shall not exceed value given in the following equation.

$$v = \frac{64{,}000{,}000}{(h/t)^2} \text{ with a maximum of } \frac{2}{3} f_b$$

where

t = web thickness, in.
h = clear distance between flanges, in.
f_b = basic working stress as specified in Sec. 3.1, psi.

Where the web consists of two or more sheets, each sheet shall be considered as a separate member carrying its share of the shear.

(Values in accordance with the above formula are given in Table 3.4.1.)

TABLE 3.4.1

MAXIMUM ALLOWABLE SHEAR IN FLAT WEBS

$$v = \frac{64{,}000{,}000}{(h/t)^2} \text{ WITH MAXIMUM } \tfrac{2}{3} f_b$$

h/t	v
60	17,780
70	13,060
73	12,000 (maximum for Grade C)
80	10,000
90	7,900
100	6,400
110	5,290
120	4,440
130	3,790
140	3,270
150	2,840

3.4.2. Bending Stress in Webs

The compressive stress f_w, in pounds per square inch, in the flat web of a beam due to bending in its plane, shall not exceed f_b, nor shall it exceed

$$f_w = \frac{520{,}000{,}000}{(h/t)^2}$$

where h = clear distance between flanges, in.

3.4.3. Combined Bending and Shear Stresses in Webs

For webs subject to both bending and shear stresses, the member shall be so proportioned that such stresses do not exceed the allowable values specified in Sec. 3.4.1 and 3.4.2 and so that the quantity

$(f'_b/f_w)^2 + (v'/v)^2$ does not exceed unity, where

$f_w = 520{,}000{,}000/(h/t)^2$

$v = 64{,}000{,}000/(h/t)^2$

f'_b = actual compressive stress at junction of flange and web

and v = actual average shear stress, i.e., shear force per web divided by web area, psi.

3.5. Web Crippling of Beams

To avoid crippling of flat webs of beams, concentrated loads and reactions shall not exceed the values P_{max} given below.

(a) Beams of Grade C steel having single unreinforced webs with inside corner radius equal to or less than the thickness of the sheet:

(1) For end reactions or for concentrated loads on the outer ends of cantilevers,
$P_{max} = 100\, t^2 \,[980 + 42(B/t) - 0.22\,(B/t)(h/t) - 0.11\,(h/t)]$
For other grades of steel and other corner radii, the value P_{max} given by the above formula is to be multiplied by

$$k\,(1.15 - 0.15n)\,(1.33 - 0.33k)$$

(2) For reactions of interior supports or for concentrated loads located anywhere on the span

$$P_{max} = 100\, t^2 \,[3050 + 23(B/t) - 0.09\,(B/t)(h/t) - 5(h/t)]$$

For other grades of steel and other corner radii, the value P_{max} given by the above formula is to be multiplied by

$$k\,(1.06 - 0.06n)\,(1.22 - 0.22k)$$

(b) For I-beams made of two channels connected back-to-back or for similar sections which provide a high degree of restraint against rotation of the web, such as I-sections made by welding two angles to a channel:

(1) For end reactions or for concentrated loads on the outer ends of cantilevers

$$P_{max} = t^2 f_b\,(7.4 + 0.93\,\sqrt{B/t})$$

(2) For reactions of interior supports or for concentrated loads located anywhere on the span

$$P_{max} = t^2 f_b\,(11.1 + 2.41\,\sqrt{B/t})$$

In all the above, P_{max} represents the load or reaction for one solid web sheet connecting top and bottom flanges. For webs consisting of two or more such sheets, P_{max} shall be computed for each individual sheet and the results added to obtain the allowable load or reaction for the composite web.

For loads located close to ends of beams, provisions (a–2) and (b–2) apply, provided that for cantilevers the distance from the free end to the nearest edge of bearing and for a load close to an end support, the distance from interior edge of end bearing to nearest edge of load bearing, is larger than $1.5h$. Otherwise provisions (a–1) and (b–1) apply.

In the above formulas,

P_{max} = allowable concentrated load or reactions, in pounds,
t = web thickness, in inches,
B = actual length of bearing, in inches, except that in the above formulas the value of B shall not be taken greater than h,
h = clear distance between flanges, in inches,
f_b = basic allowable design stress, in psi, (Sec. 3.1)
k = $f_b/18{,}000$,
n = ratio of inside bend radius divided by web thickness.

3.6. Axially Loaded Compression Members

3.6.1. Unit Stress

The average axial stress, P/A, in compression members, shall not exceed the values of F_a, as follows:*

For Grade C steel

L/r equal to or less than $132/\sqrt{Q}$: $\quad F_a = 15{,}300Q - 0.437Q^2(L/r)^2$

L/r equal to or greater than $132/\sqrt{Q}$: $\quad F_a = \dfrac{134{,}000{,}000}{(L/r)^2}$

For other Grades of steel

L/r equal to or less than $\dfrac{24{,}000}{\sqrt{f_y}\sqrt{Q}}$: $\quad F_a = 0.464Qf_y - \left(\dfrac{2Qf_yL/r}{100{,}000}\right)^2$

L/r equal to or greater than $\dfrac{24{,}000}{\sqrt{f_y}\sqrt{Q}}$: $\quad F_a = \dfrac{134{,}000{,}000}{(L/r)^2}$

In the above formulas,

P = total load, lb.
A = full, unreduced cross-sectional area of the member, in^2.
F_a = maximum allowable average axial stress in compression, psi.
L = unsupported length† of member, in.
r = radius of gyration of full, unreduced cross section, in.
f_y = yield point of steel, lb per sq in.
Q = a factor determined as follows:

(a) For members composed entirely of *stiffened* elements, Q is the ratio between the effective design area, as determined from the effective design widths of such elements, and the full or gross area of the cross section. The effective design area used in determining Q is to be based upon the basic design stress f_b, as defined in Sec. 3.1.

(b) For members composed entirely of *unstiffened* elements, Q is the ratio between the allowable compression stress f_c for the weakest element of the cross section (the element having the largest flat-width ratio) and the basic design stress f_b; where f_c is as defined in Sec. 3.2 and f_b is as defined in Sec. 3.1.

(c) For members composed of both *stiffened and unstiffened* elements, the factor Q is to be the product of a stress factor Q_s, computed as outlined in (b) above, and an area factor Q_a, computed as outlined in (a) above, except that the stress upon which Q_a is to be based shall be that value of the unit stress f_c which is used in computing Q_s; and the effective area to be used in computing Q_a shall include the full area of all unstiffened elements.

3.6.2. Maximum Slenderness Ratio

The maximum allowable ratio L/r of unsupported length, L, to radius of gyration, r, of compression members shall be as follows:

(a) Columns, and other primary compression members, except as provided otherwise in this Section 120

* Values of F_a for different Q and L/r values appear in Chart 4

† For continuous compression chords of trusses with rigid welded connections at panel points, the value of L to be used in computing L/r in the plane of the truss is ¾ the distance between panel points.

The slenderness ratio L/r of a main compression member may exceed 120, but not 200, provided its unit stress under full design load does not exceed the following fraction of that stipulated under Sec. 3.6.1.

$$1.6 - (L/200r)$$

(b) Load-bearing wall studs.. 160
The slenderness ratio L/r of a load-bearing wall stud may exceed 160, but not 200, provided its unit stress under full design load does not exceed the following fraction of that stipulated under Sec. 3.6.1.

$$2.6 - (L/100r)$$

(c) Secondary members.. 200
Exception: During construction only, L/r may exceed the foregoing limits but shall not exceed 300.

If members which are temporarily unbraced during construction are to act as permanent load-carrying members in the completed structure they must be so braced prior to completion of the structure as to reduce the L/r ratio to a value not exceeding that given in (a), (b), or (c) above, whichever may apply.

3.7. Combined Axial and Bending Stresses

Members subject to both axial compression and bending stresses shall be so proportioned that the quantity

$$(f_a/F_a) + (f'_b/F_b) \text{ shall not exceed unity,}$$

where

F_a = maximum axial unit stress in compression that is permitted by this Specification where axial stress only exists. (Sec. 3.6.1)

F_b = maximum bending unit stress in compression that is permitted by this Specification where bending stress only exists. (Sec. 3.1 and 3.2)

f_a = axial unit stress = axial load divided by full cross-sectional area of member, P/A.

f'_b = bending unit stress = bending moment divided by section modulus of member, M/S, noting that for members having stiffened compression elements the section modulus shall be based upon the effective design widths of such elements.

3.8. Wind or Earthquake Stresses

3.8.1. Wind or Earthquake Only

Members and assemblies subject only to stresses produced by wind or earthquake forces may be proportioned for unit stresses 33⅓ per cent greater than those specified for dead- and live-load stresses. A corresponding increase may be applied to the allowable unit stresses in connections and details.

3.8.2. Combined Forces

Members and assemblies subject to stresses produced by a combination of wind or earthquake and other loads may be proportioned for unit stresses 33⅓ per cent greater than those specified for dead- and live-load stresses, provided the section thus required is not less than that required for the combination of dead load and live load. A corresponding increase may be applied to the allowable unit stresses in connections and details.

3.9. Cylindrical Tubular Compression Members

The ratio, D/t, of mean diameter to wall thickness of a cylindrical tubular compression member shall not exceed $3{,}300{,}000/f_y$. For such members, the allowable unit stress P/A under axial load shall be as prescribed by Sec. 3.6.1, with $Q = 1$.

SECTION 4. CONNECTIONS

4.1. General

Connections shall be designed to transmit the maximum stress in the connected member with proper regard for eccentricity. In the case of members subject to reversal of stress, except if caused by wind or earthquake loads, the connection shall be proportioned for the sum of the stresses.

4.2. Welds

4.2.1. Fusion Welds

For all grades of steel, fusion welds shall be proportioned so that the unit stresses therein do not exceed 13,600 psi in shear on the throat of fillet or plug welds. The allowable unit stress in tension or compression on butt welds shall be the same as prescribed for the base metal being joined, provided the weld penetrates 100 per cent of the section. Stresses due to eccentricity of loading, if any, shall be combined with the primary stresses; and the combined unit stresses shall not exceed the values given above.

Stresses in a fillet weld shall be considered as shear on the throat for any direction of the applied stress. Neither plug nor slot welds shall be assigned any value in resistance to any stresses other than shear.

All fusion welding shall comply with the provisions of the *Standard Code for Arc and Gas Welding in Building Construction* of the American Welding Society, of latest edition, except as otherwise specified herein and excepting such provisions of that Code as are clearly not applicable to material of the thicknesses to which this Specification applies.

4.2.2. Resistance Welds

In sheets joined by spot welding, the design strength per spot shall be as given in the table on page 634.

Thickness of Thinnest Outside Sheet, Inches	Design Strength per Spot, Pounds
0.010	50
0.020	125
0.030	225
0.040	350
0.050	525
0.060	725
0.080	1075
0.094	1375
0.109	1650
0.125	2000
0.155	2700
0.185	3300

(The above values are based upon the American Welding Society's *Recommended Practice for Resistance Welding*. They are applicable for all structural grades of low-carbon steel, up to a yield point of 70,000 lb per sq in. and are based on a factor of safety of approximately two and one-half. The welding procedure shall conform to that set forth in the *Recommended Practice* published by the American Welding Society.)

4.3. Welds Connecting Two Channels to Form an I-Section for Use as a Beam

The required tension strength of welds connecting two channels to form an I-beam shall be determined from the following formula:

$$S_w = \frac{mqs}{2c}$$

where S_w = required strength of weld in tension, lb.
s = longitudinal spacing of welds, in.
c = vertical distance between the two rows of welds near or at top and bottom flanges, in.
q = intensity of load, pounds per linear inch of beam (for method of determination, see below).
m = the distance of shear center of channel from midplane of the web, in.
For simple channels without stiffening lips at the outer edges,

$$m = \frac{w^2}{2w + h/3}$$

For C-shaped channels with stiffening lips at the outer edges,

$$m = \frac{wht}{4I_x}[wh + 2d(h - d)]$$

where w = projection of flanges beyond web, in inches. (For channels with flanges of unequal width, w shall be taken as the width of the wider flange.)
h = depth of channel or beam, in.
d = depth of lip, in.
I_x = the moment of inertia of one channel about its centroidal axis normal to the web, in.^{4}

The intensity of load, q, is obtained by dividing the magnitude of concentrated loads or reactions by the length of bearing or by longitudinal spacing of welds, s, whichever is larger. For beams designed for "uniformly distributed load," the intensity q shall be taken equal to three times the intensity of the uniformly distributed design load.

The required strength of welds depends upon the intensity of the load directly at the weld. Therefore, if uniform diameter and spacing of welds are used over the whole length of the beam, the necessary strength of the welds shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods may be adopted: (a) the weld spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The required strength *in shear* of the welds connecting these plates to the flanges shall then be determined from the formula for S_w specified herein but c shall then represent the depth of the beam.

4.4. Spacing of Connections in Compression Elements

The spacing, in line of stress, of welds, rivets, or bolts connecting a compression cover plate or sheet to a non-integral stiffener or other element shall not exceed

(a) That which is required to transmit the shear between the connected parts on the basis of the design strength per connection specified elsewhere herein; nor

(b) $s = 6000t/\sqrt{f}$, where s is spacing in inches, t is the thickness of the cover plate or sheet in inches, and f is the design stress in the cover plate or sheet in pounds per square inch; nor

(c) Three times the total flat width, w, of the narrowest unstiffened compression element in that portion of the cover plate or sheet which is tributary to the welds but need not be less than 36 times the thickness of such element unless closer spacing is required by (a) or (b) of this Sec. 4.4.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds plus ½ in. In all other cases the spacing shall be taken as the center-to-center distance between connections.

Exception: The requirements of this Sec. 4.4 do not apply to cover sheets which act only as sheathing material and are not considered as load-carrying elements.

4.5. Bolted Connections

The following requirements govern bolted connections of light gage steel structural members.

4.5.1. Minimum Spacing and Edge Distance in Line of Stress

The clear distance between bolts which are arranged in rows parallel to the direction of force, also the distance from the center of any bolt to that end or other boundary of the connecting member toward which the pressure of the bolt is directed, shall not be less than $1\frac{1}{2}\,d$ nor less than

$$P/f_b t$$

where d = diameter of bolt, in.
P = force transmitted by bolt, lb.
t = thickness of thinnest connected sheet, in.
f_b = basic design stress, as defined elsewhere, psi.

4.5.2. Tension Stress on Net Section

The tension stress on the net section of a bolted connection shall not exceed f_b, nor shall it exceed

$$\left(0.1 + \frac{3d}{s}\right)f_b$$

where s = spacing of bolts perpendicular to line of stress, in. In the case of a single bolt, s = width of sheet.
d and f_b are as previously defined.

4.5.3. Bearing Stress in Bolted Connections

The bearing stress on the area ($d \times t$) shall not exceed $3.5f_b$.

4.5.4. Shearing Stress on Bolts

The allowable value for shear on unfinished bolts is 10,000 lb per sq in.

SECTION 5. BRACING REQUIREMENTS

Structural members and assemblies of light gage steel construction shall be adequately braced in accordance with good engineering practice. The following provisions cover certain special cases and conditions.

5.1. Wall Studs

The safe load-carrying capacity of a stud may be computed on the basis that wall material or sheathing (attached to the stud) furnishes adequate lateral support to the stud in the plane of the wall, provided the wall material and its attachments to the stud comply with the following requirements:

(a) Wall material or sheathing must be attached to both faces or flanges of the studs being braced.

(b) The spacing of attachments of wall material to the stud being braced shall not exceed a_{max} as determined from the formula:

$$a_{max} = \frac{8EI_2 k}{A^2 f_y^2}$$

where k is the test value as defined in (d), and the other terms are as defined in (c) and (d).

The slenderness ratio of the stud between attachments, a/r_2, shall not exceed $L/2r_1$. Therefore, the spacing of attachments shall not exceed that specified above nor shall it exceed

$$a_{max} = \frac{Lr_2}{2r_1}$$

where L = length of stud, in.

r_1 = radius of gyration of stud about its axis parallel to wall = $\sqrt{I_1/A}$, in.

r_2 = radius of gyration of stud about its axis perpendicular to wall = $\sqrt{I_2/A}$, in.

(c) The minimum modulus of elastic support, k, to be exerted laterally by the wall material and its attachments in order to brace the stud, shall be not less than,

For steel of Grade C, $k = \dfrac{4.5aA^2}{I_2}$

For Grade B, $k = \dfrac{3.7aA^2}{I_2}$

For Grade A, $k = \dfrac{2.6aA^2}{I_2}$

For steel of grade other than Grades A, B, and C:

$$k = \frac{f_y^2\, aA^2}{240{,}000{,}000\, I_2}$$

where f_y = yield point of the steel in the studs, psi.

a = spacing of attachments of wall material to stud measured along the length of stud, (a = 1 for continuous attachment), in.

A = area of cross section of stud, in.^{2}

I_2 = moment of inertia of cross section of stud about its axis perpendicular to wall, in.^{4}

k = spring constant or modulus of elastic support of wall material (on each [one] side of stud) plus attachment, i.e., $k = F/y$ where F is the force in pounds which produces an elongation of y inches of a strip of wall material of width a and of length equal to the distance between adjacent studs, lb per in.*

(d) The lateral force, F, which each single attachment of the wall material shall be capable of exerting on the stud in the plane of the wall (in order to prevent lateral buckling of the stud) shall not be less than:

$$F_{\min} = \frac{keP}{2\sqrt{(EI_2k/a)} - P}$$

where k = modulus of elastic support of the wall material as determined from tests, i.e., the value of k used in the formula to determine the maximum allowable spacing of attachments in (b), lb per in.

e = stud length in inches/240

P = design load on stud, lb.

I_2 = moment of inertia of stud about its axis perpendicular to the wall, in.^{4}

a = spacing of attachments measured along stud, in. (a = 1 in. for continuous attachment).

E = modulus of elasticity = 29,500,000 psi.

* Whether a given wall material or means of attachment satisfies the requirements of this Section may be established by the test procedure described in Part II of the AISI Manual. Also included in Part II are a number of k values, determined by the test procedure described, for several common types of wall sheathing.

5.2. Channel and Z-Sections Used as Beams

The following provisions for the bracing, against twist, of channel and Z-sections used as beams apply only when (a) neither flange is connected to deck or sheathing material in such a manner as to restrain effectively the lateral deflection of the connected flange, and (b) such members are loaded in the plane of the web.*

5.2.1. Spacing of Braces

Braces shall be attached both to the top and bottom flanges of the sections at the ends and at intervals not greater than one-quarter of the span length in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces. If one-third or more of the total load on the beams is concentrated over a length of one-twelfth or less of the span of the beam, an additional brace shall be placed at or near the center of this loaded length.

5.2.2. Design of Braces

Each intermediate brace, at top and bottom flange, shall be designed to resist a lateral force P_b determined as follows:

(a) For a uniformly loaded beam, $P_b = 1.5K$ times the load within a distance $0.5a$ each side of the brace.

(b) For concentrated loads $P_b = 1.0K$ times the concentrated load P within a distance $0.3a$ each side of the brace, plus a force F determined from the following formula, for each such concentrated load P located farther than $0.3a$, but not farther than a from the brace:

$$F = \frac{1.0}{0.7}\left(1 - \frac{x}{a}\right)PK$$

In the above formulas:

For channels: $K = m/h$

where m = distance from shear center to midplane of the web, as specified in Sec. 4.3, in.

h = depth of channel, in.

For Z-sections: $K = I_{xy}/I_x$

where I_{xy} = product of inertia of full section about centroidal axes parallel and perpendicular to web, in.^{4}

I_x = moment of inertia of full section about centroidal axis perpendicular to web, in.^{4}

For channels and Z-sections:

x = distance from concentrated load P to brace, in.

a = length of bracing interval, in.

* When only one flange is connected to a deck or sheathing material to effectively restrain the lateral deflection of the connected flange, bracing may or may not be needed to prevent twisting of the member, depending upon the dimensions of the member and span and upon whether the unconnected flange is in compression or tension.

End braces shall be designed for one-half of the above forces.

Braces shall be so designed as to avoid local crippling at the points of attachment to the member.

5.2.3. Allowable Stresses

For channel and **Z**-beams intermediately braced according to the requirements of Sec. 5.2.1 and 5.2.2, the maximum compression stress f'_c shall be that specified in Sec. 3.3, except that the length of the bracing interval, a, shall be used instead of the length L in the formulas of that Section.

5.3. Laterally Unbraced Box Beams

For closed box-type sections used as beams the ratio of the laterally unsupported length, L, to the distance between the webs of section shall not exceed 75.

SECTION 6. TESTS FOR SPECIAL CASES

6.1. General

Where elements, assemblies, or details of structural members formed from sheet or strip steel are such that calculation of their safe load-carrying capacity or deflection cannot be made in accordance with the provisions of Sec. 2 through 5 of this Specification, their structural performance shall be established from test procedure as specified in Sec. 6.2.

6.2. Test Procedure*

It is recommended that tests for the purposes defined in Sec. 6.1 be conducted in accordance with the following procedure:

(a) Where practicable, evaluation of test results shall be made on the basis of the mean values resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test resulting from the mean value obtained from all tests does not exceed ± 10 per cent. If such deviation from the mean exceeds 10 per cent, at least three more tests of the same kind shall be made. The average of the three lowest values of all tests made shall then be regarded as the result of the series of tests.

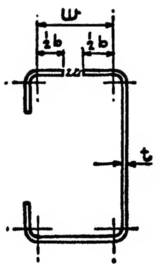
(b) Determinations of allowable load carrying capacity shall be made on the basis that the member, assembly, or connection shall be capable of sustaining during the test without failure a total load, including the weight of the test specimen, equal to twice the live load plus twice the dead load. Furthermore, harmful local distortions shall not develop during the test at a total load, including the weight of the test specimen, equal to the dead load plus one and one-half times the live load. For members and assemblies subjected to wind or earthquake loads, appropriate modification of the foregoing factors shall be made in accordance with Sec. 3.8.

* The test procedures and test factors specified in Sec. 6.2 are not applicable to confirmatory tests of members and assemblies whose properties have been calculated according to Sec. 2 through 5; for the latter, the Specification provides generally a safety factor of 1.85.

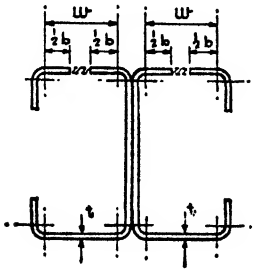
(c) In evaluating test results, due consideration must be given to any differences that may exist between the yield point of the material from which the tested sections are formed and the minimum yield point specified for the material which the manufacturer intends to use.

(d) Tests shall be made by an independent testing laboratory or by a manufacturer's testing laboratory.

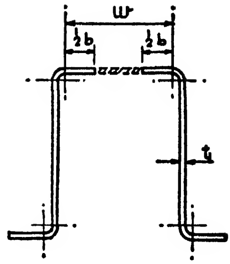
EFFECTIVE CROSS SECTIONS OF MEMBERS IN BENDING



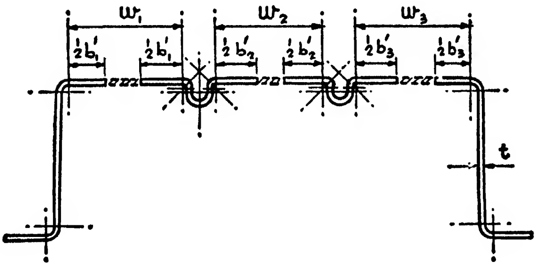
Lipped Channel
or
"C" Section



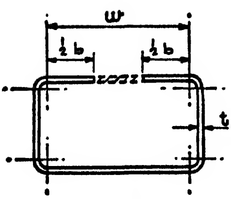
Double Channel I-Beam
with Stiffened Flanges



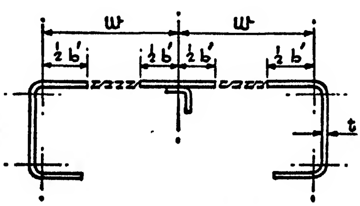
Hat Section



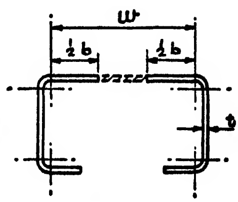
Multiple Stiffened Hat Section



Box-Type Section



Multiple Stiffened
Inverted "U" Section



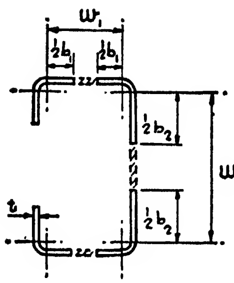
Inverted "U" Section

Chart 1.

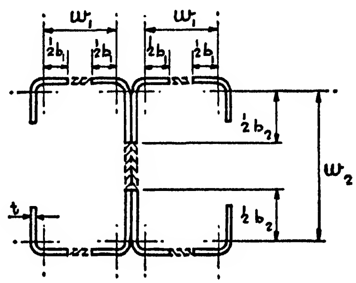
EFFECTIVE DESIGN AREA FOR DETERMINING "Q" FOR CROSS SECTIONS OF MEMBERS IN COMPRESSION

(See Section 3.6.1)

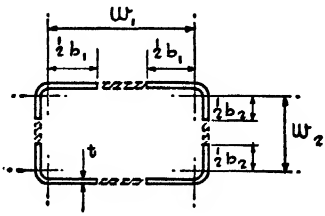
$$Q = \frac{\text{Total Area} - \text{Shaded Area}}{\text{Total Area}}$$ (WHERE ALL ELEMENTS ARE STIFFENED AT BOTH EDGES)



Lipped Channel
or
"C" Section

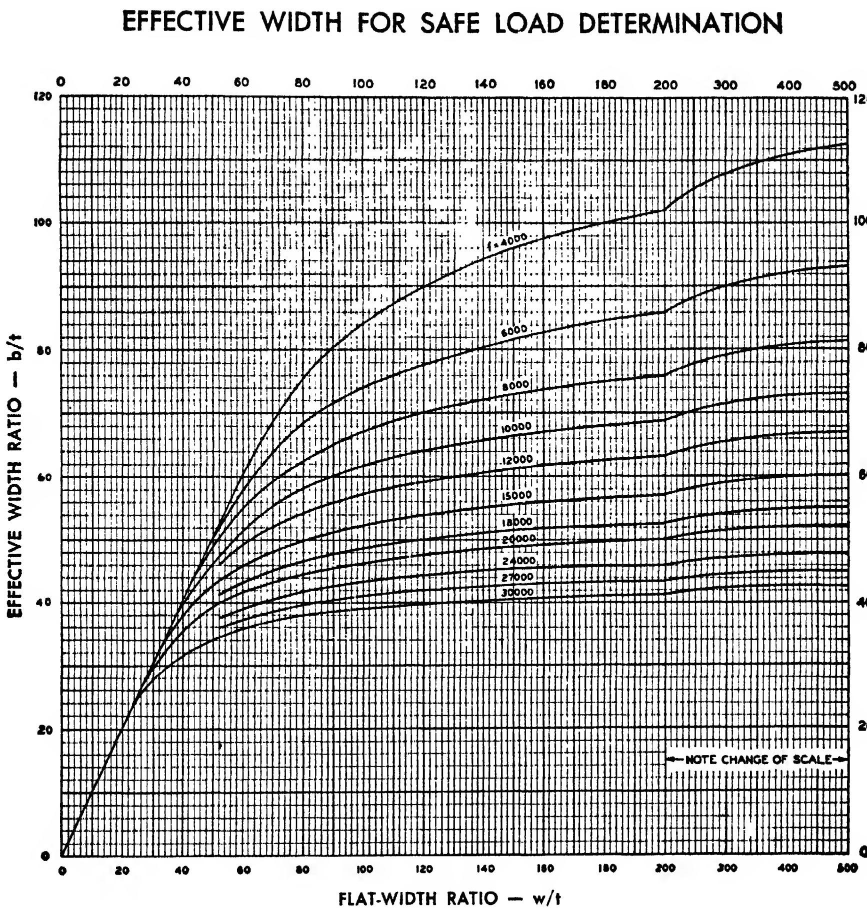


Double Channel I-Beam
with Stiffened Flanges



Box-Type Section

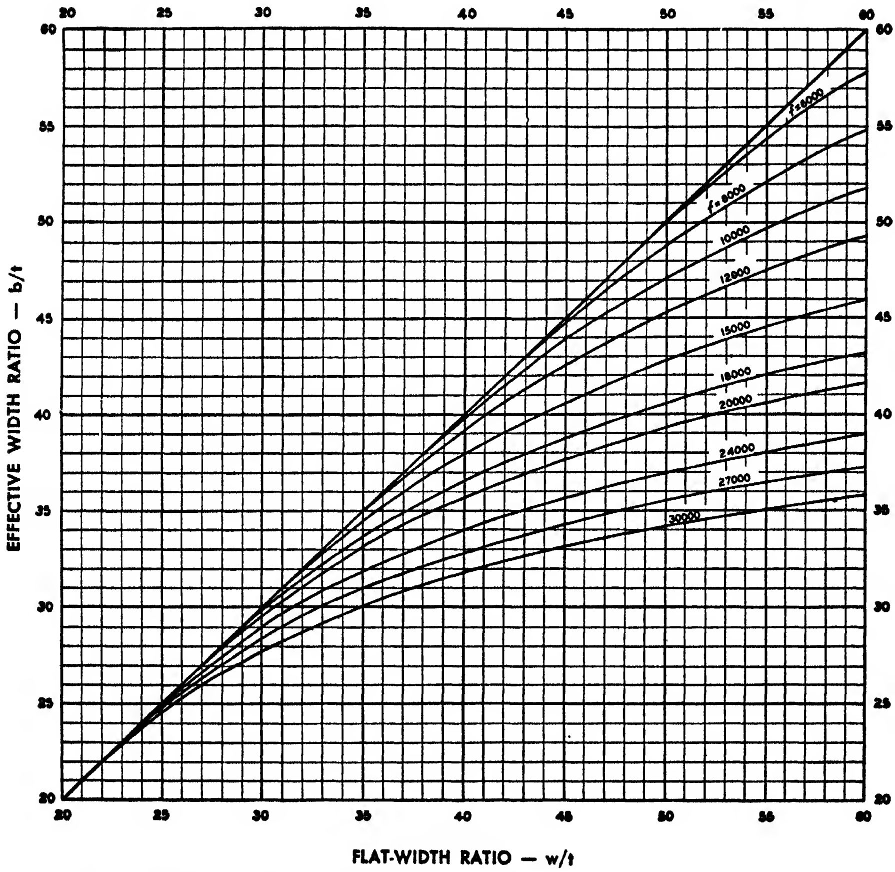
Chart 2.



COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES (Large w/t Ratios)

Chart 3a.

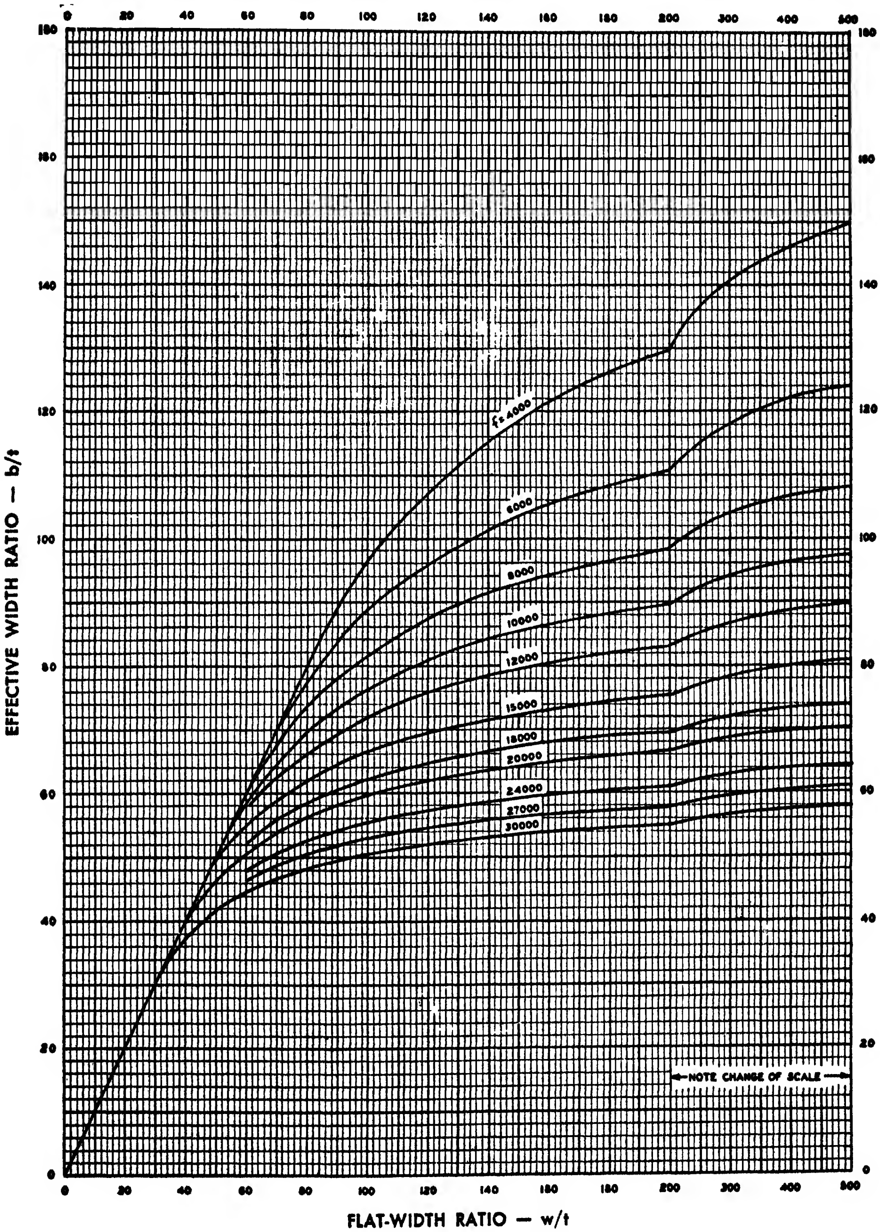
EFFECTIVE WIDTH FOR SAFE LOAD DETERMINATION



COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES (Small w/t Ratios)

Chart 3b.

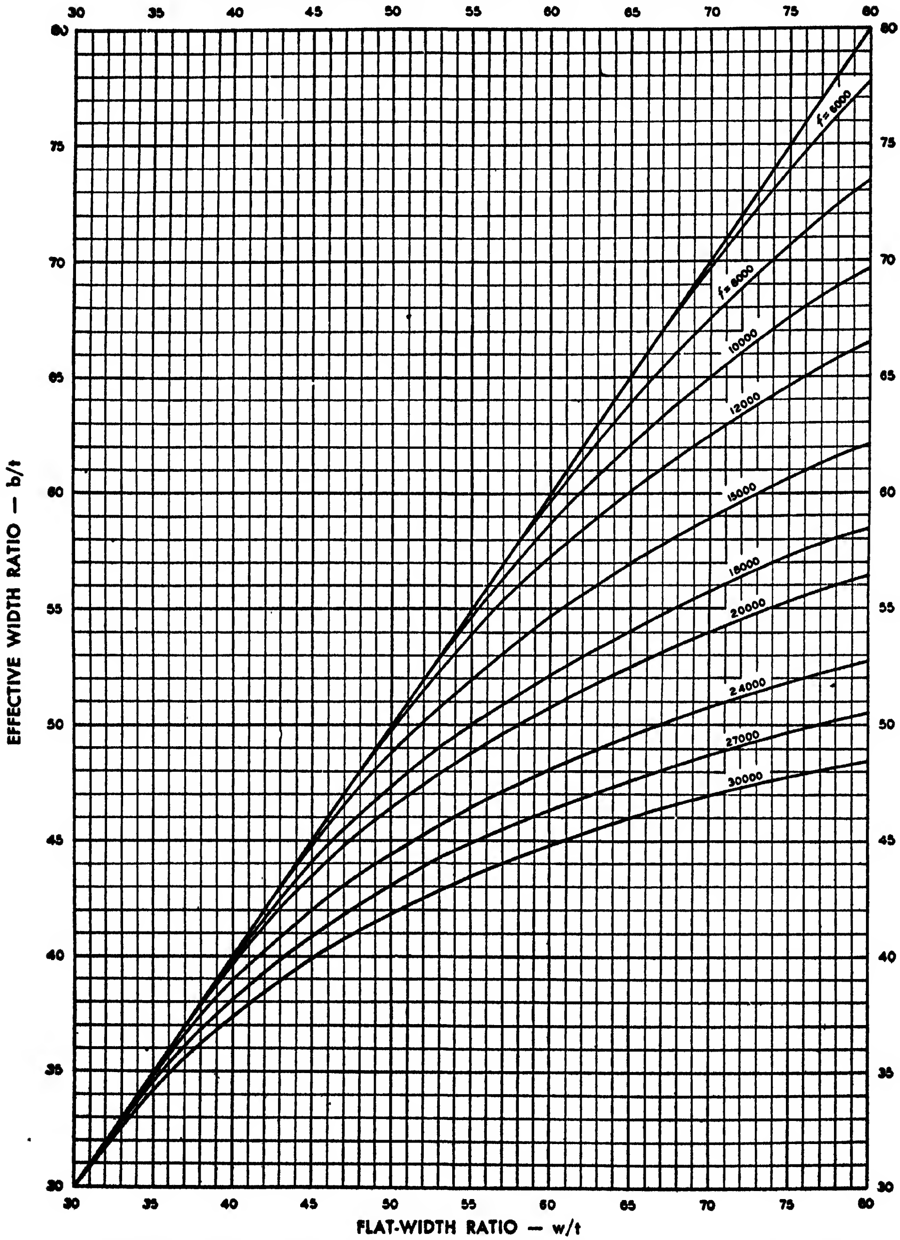
EFFECTIVE WIDTH FOR DEFLECTION DETERMINATION



COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES (Large w/t Ratios)

Chart 3c.

EFFECTIVE WIDTH FOR DEFLECTION DETERMINATION



COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES (Small w/t Ratios)

Chart 3d.

COLUMN DESIGN CURVES
FOR COLD FORMED SHAPES
BASED ON GRADE C STEEL

F_a for varying Q and $\frac{L}{r}$ values

For $\frac{L}{r} \leqslant \frac{132}{\sqrt{Q}}$: $F_a = 15{,}300\,Q - 0.437\,Q^2\left(\frac{L}{r}\right)^2$;

For $\frac{L}{r} \geqslant \frac{132}{\sqrt{Q}}$: $F_a = \dfrac{134{,}000{,}000}{\left(\frac{L}{r}\right)^2}$

This Chart may be used for other grades of steel than Grade C by using a Q value equal to the actual value of Q for the section (based upon the grade of steel actually being used) multiplied by the ratio $f_y/33{,}000$. The curves on this chart for Q values greater than 1.00 do not apply to Grade C Steel.

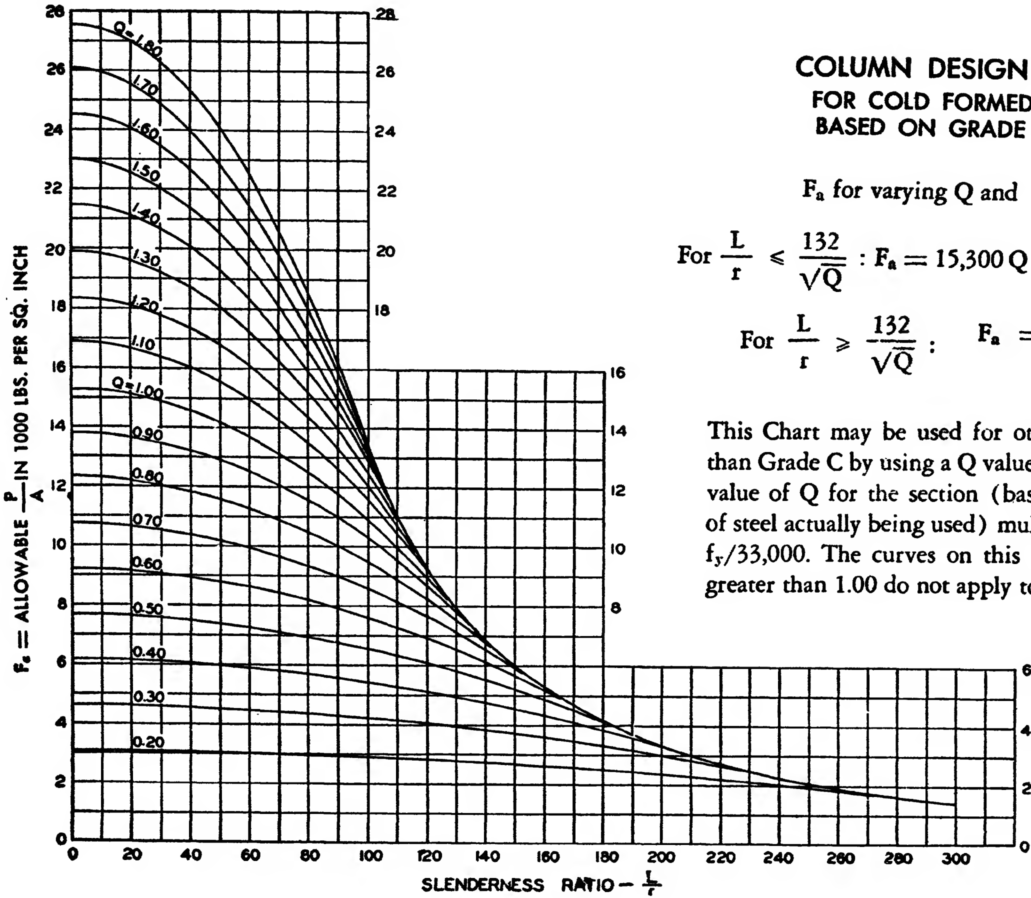


Chart 4.

SUBJECT INDEX

NAME INDEX