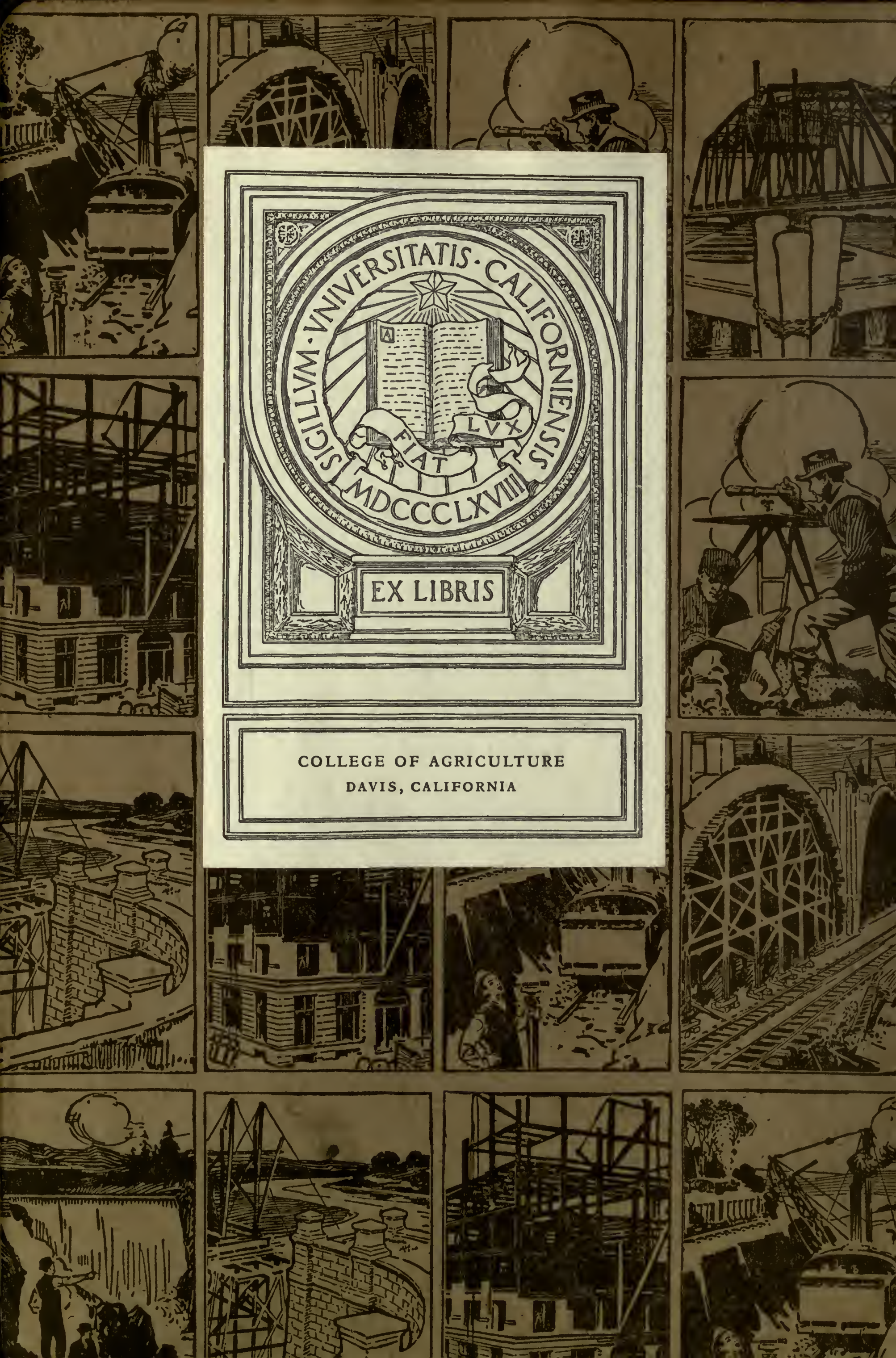

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Cyclopedia

of

Civil Engineering

A General Reference Work on

SURVEYING, HIGHWAY CONSTRUCTION, RAILROAD ENGINEERING, EARTHWORK, STEEL CONSTRUCTION, SPECIFICATIONS, CONTRACTS, BRIDGE ENGINEERING, MASONRY AND REINFORCED CONCRETE, MUNICIPAL ENGINEERING, HYDRAULIC ENGINEERING, RIVER AND HARBOR IMPROVEMENT, IRRIGATION ENGINEERING, COST ANALYSIS, ETC.

Prepared by a Corps of

CIVIL AND CONSULTING ENGINEERS AND TECHNICAL EXPERTS OF THE HIGHEST PROFESSIONAL STANDING

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Grateful acknowledgment is here made also for the invaluable co-operation of the foremost Civil, Structural, Railroad, Hydraulic, and Sanitary Engineers and Manufacturers in making these volumes thoroughly representative of the very best and latest practice in every branch of the broad field of Civil Engineering.

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EQUITABLE BUILDING, NEW YORK CITY

This building, on the site of the old Equitable Building, is a fine example of modern steel construction.

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Foreword

OF all the works of man in the various branches of engineering, none are so wonderful, so majestic, so awe-inspiring as the works of the Civil Engineer. It is the Civil Engineer who throws a great bridge across the yawning chasm which seemingly forms an impassable obstacle to further progress. He designs and builds the skeletons of steel to dizzy heights, for the architect to cover and adorn. He burrows through a great mountain and reaches the other side within a fraction of an inch of the spot located by the original survey. He scales mountain peaks, or traverses dry river beds, surveying and plotting hitherto unknown, or at least unsurveyed, regions. He builds our Panama Canals, our Arrow Rock and Roosevelt Dams, our water-works, filtration plants, and practically all of our great public works.

¶ The importance of all of these immense engineering projects and the need for a clear, non-technical presentation of the theoretical and practical developments of the broad field of Civil Engineering has led the publishers to compile this great reference work. It has been their aim to fulfill the demands of the trained engineer for authoritative material which will solve the problems in his own and allied lines in Civil Engineering, as well as to satisfy the desires of the self-taught practical man who attempts to keep up with modern engineering developments.

¶ Books on the several divisions of Civil Engineering are many and valuable, but their information is too voluminous to be of the greatest value for ready reference. The Cyclopedia of Civil Engineering offers more condensed and less technical treatments of these same subjects from which all unnecessary duplication has been eliminated; when compiled into nine handy volumes, with comprehensive indexes to facilitate the looking up of various topics, they represent a library admirably adapted to the requirements of either the technical or the practical reader.

¶ The Cyclopedia of Civil Engineering has for years occupied an enviable place in the field of technical literature as a standard reference work and the publishers have spared no expense to make this latest edition even more comprehensive and instructive.

¶ In conclusion, grateful acknowledgment is due to the staff of authors and collaborators—engineers of wide practical experience, and teachers of well recognized ability — without whose hearty co-operation this work would have been impossible.

Table of Contents

VOLUME III

STEEL CONSTRUCTION . . . *By Henry Jackson Burt*† Page *11

Structural Steel: Introduction—Methods of Manufacture: Iron Ore to Pig Iron, Pig Iron to Steel, Rolling Ingots—Steel Sections: Classification, I-Beams, Channels, Angles, Zees, Tees, Plates, H-Sections, Miscellaneous—Properties of Sections—Price Basis—Quality of Material—Standard Specifications—Chemical Composition of Steel—Physical Properties of Steel—Inspection and Tests—Unit Stresses—Rivets and Bolts: Spacing, Rivet Heads, Driving, Functions, Riveted Joists, Bolts—**Beams:** Theory of Beam Design: Maximum Bending Moment, Maximum Shear, Deflection, Flexure, Vertical Shear, Modulus of Elasticity—Calculation of Load Effects: Uniformly Distributed Loads, Concentrated Loads, Combined Loads, Typical Loadings—Calculation of Resistance: Resisting Moment, Shearing Resistance, Deflection, Lateral Supports—Practical Applications: Floor Framing Panel, Lintels, Cantilevers—Details of Construction: Beam and Column Connections, Separators, Tie Rods, Bearings, Anchors—**Riveted Girders:** Theory of Design: Determination of Resisting Moment, Calculation of Load Effects—Design of Plate Girder: Depth, Thickness of Web, Flange Sections, Length of Flange Plates, Web Stiffeners, Rivets and Rivet Spacing—Other Forms of Riveted Girders—Practical Applications—Construction Details: End Bearings, Column Connections, Splices, Lateral Supports—**Compression Members:** Steel Columns: Concentric Loads, Eccentric Loads, Typical Cases, Strength of Columns, Properties of Column Sections, Tables, Details of Construction, Bases—Cast-Iron Columns: Characteristics, Column Sections, Method of Design, Tables, Details of Construction—Tension Members: Axial Tensions—Tension Due to Eccentricity—Sections—Net Area—Details of Connections: Riveted Connections, Rods, I-Bars—**Wind Bracing:** Horizontal Pressure—Paths of Stress—Systems of Frame-Work: Triangular, Rectangular—Design of Wind Bracing Girders: End Connection for Riveted Girders, End Connections for I-Beam Girders—Combined Wind and Gravity Stresses in Girders—Effect of Wind Stresses on Columns—**Practical Design of a Sixteen-Story Fireproof Hotel:** Description of Building—Plates A to X—Fireproof Specifications—Loads—Type of Floor Construction—Framing Specifications: Arrangement of Girders and Joists, Beam Elevations, Arrangement of Columns, Design of Beams—Column Specifications—Column Pedestals—Wind Bracing—Miscellaneous Features—Dimensioning Drawings—**Protection of Steel from Rust:** Nature of Rust—Theory of Formation—Rate of Rusting—**Paint:** Purpose, Qualities, Composition, Method of Application, Cement as a Rust Preventive—**Protection of Steel from Fire:** Effects of Heat on Steel—Protective Methods—Fireproof Material: Cinder Concrete, Portland Cement Concrete, Hollow-Tile, Brick—Selection of Fireproofing—**Typical Specifications:** General Characteristics and Outline—Instructions to Bidders—General Conditions—Scope—Loads—Unit Stresses—Quality of Materials—Structural Steel for Buildings—Gray-Iron Castings—Paint—Inspection and Testing—Erection

REVIEW QUESTIONS . . . . Page 389

INDEX . . . . Page 397

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* For page numbers, see foot of pages.

† For professional standing of author, see list of Authors and Collaborators at front of volume.

CONTINENTAL AND COMMERCIAL BANK BUILDING, CHICAGO, IN PROCESS OF CONSTRUCTION

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STEEL CONSTRUCTION

PART I

INTRODUCTION

Scope of Work. The subject of steel construction as here used covers the use of structural steel for the supports for buildings, whether in the forms of isolated members or complete framework. It deals especially with architectural structures, such as business buildings, office buildings, warehouses, residences, etc. Mill buildings and roof trusses might properly be included under this subject, but as they are covered elsewhere in the course of study, they are not repeated here.

Consideration is given first to the structural steel sections, i. e., the shapes in which the material is available, such as plates, angles, I-beams, etc., studying their properties and uses. Certain definite sizes, shapes, and weights of sections can be purchased in the market. Acquaintance with these sections and some knowledge of the purposes for which the special shapes are adapted are essential preliminaries to the study of steel design.

The designer should know the quality of the material which he is using; therefore, a brief discussion of the chemical composition and physical properties of steel for structural purposes is given.

Experience and experiment have established the working loads, i. e., unit stresses, that can be applied safely to structural steel under various conditions. The values now used are so well established that they may be considered as standard. Consequently, the unit stresses are given with only such discussion as is necessary to explain their application.

After these preliminary considerations comes the study of design. As rivets and bolts are used in all forms of structural members, a section of the text is devoted to them before taking up beams, columns, and tension members. The study of these members gives a review of the theory involved, the formulas, the computation of loads, the application to assumed cases, and details of construction.

Having studied the elements of the structure as described above, complete structures are then investigated and designed. Examples of existing structures are taken for this purpose. And, finally, there is a discussion of painting, fireproofing, and specifications.

Structural steel is a perishable material if exposed to the elements and is so to a considerable extent when enclosed in a building but exposed freely to the air. It is a dangerous material when exposed to fire. A part of the designer's duty is to provide the necessary protection from corrosion and from fire; consequently, considerable attention is given to painting and fireproofing.

The specifications for structural steel are quite well standardized so far as usual provisions are concerned. Nevertheless, some modifications or additions are usually required for each job. The requirements are outlined briefly in the text.

Purpose. It is the purpose of this book to give a thorough presentation of the theory and practice of design. It is believed that careful study of the text and faithful work in solving the problems will furnish the proper equipment for designing any ordinary steel construction. The ability to deal with complicated problems will follow naturally after practice with the simpler ones.

In addition to its uses as a textbook, this work is suitable for a reference book for designers, being especially useful to those who have to design steel work only occasionally, and to beginners in practical work. It does not pretend to offer anything new, but aims to explain in a simple way the established theory and practice.

Preparation. *Fundamental Principles.* In order to take up the design of structural steel work, it is necessary that one have an understanding of the theory and the formulas used in the design of the steel members. It is assumed that the essential parts of the theory, as referred to in "Strength of Materials", "Structural Drafting", "Statics", and "Roof Trusses", have been mastered, and if this is not true, these subjects should be reviewed before proceeding with "Steel Construction".

It is of the greatest importance that the fundamental principles, that is, the theory underlying the operations in designing, be kept in mind. Only in this way can one be sure that no step in the work has been omitted. This understanding of the theory will, in a large measure, remove the necessity for formulas. It would be impossible to illustrate all the problems that come up in actual practice, so that the designer must understand the theory in order to design with reasonable assurance of correctness and to solve the innumerable problems that arise.

Simple Mathematical Requirements. The mathematics required in designing are little more than arithmetic. It is true that the formulas are expressed in algebraic terms, but as these formulas are in the form required for direct application to the problems, no algebraic transformations are necessary in ordinary cases. The work to be done simply consists in substituting numerical values for the letters and performing the additions, subtractions, multiplications, and divisions indicated by the symbols. The formulas will be stated in words as well as in letters so that the designer need not follow set examples.

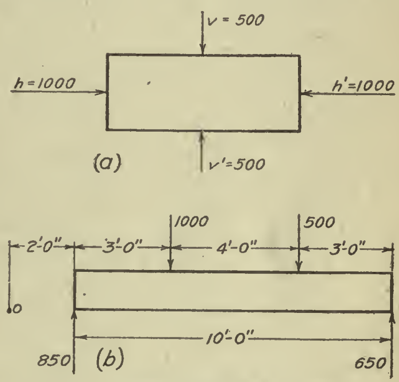

Fig. 1. Diagram Showing Forces in Equilibrium

Equilibrium Relations. The three fundamental relations of equilibrium, illustrated in Fig. 1, must always be kept in mind, viz:

(1) Summation of horizontal forces equals zero

(2) Summation of vertical forces equals zero

(3) Summation of moments equals zero

In the textbook on "Statics," equilibrium is defined as follows: *When a number of forces act upon a body which is at rest, each tends to move it; but the effects of all the forces acting upon that body may counteract or neutralize one another, and the forces are said to be balanced or in equilibrium.*

Fig. 1-a represents a body to which certain forces are applied. The horizontal forces h and h' are equal and opposite in direction,

thus satisfying the first relation. Likewise the vertical forces satisfy the second relation. The horizontal forces are in the same straight line and the vertical forces are in one straight line, hence there is no tendency to rotate and the third relation is satisfied. All of this is evident from the drawing.

Fig. 1-b represents a more complicated case. There are no horizontal forces. The vertical forces acting downward are $1000+500=1500$; acting upward are $850+650=1500$; hence the summation equals zero. Taking any point *o* as a center, the moments clockwise are

$$\begin{array}{r} 5\times 1000=5000 \\ 9\times\ \ 500=4500 \\ \hline 9500 \end{array}$$

The moments in the opposite direction are

$$\begin{array}{r} 2\times\ \ 850=1700 \\ 12\times\ \ 650=7800 \\ \hline 9500 \end{array}$$

Hence the summation of moments equals zero, and the forces acting on the body are in equilibrium.

It is because it is essential that these relations be mastered that they are stated here. They will be referred to frequently throughout the work on designing.

Method of Presentation. Throughout the discussion relating to the design of structural steel members, the order of presentation is

(a) Review of Theory
(b) Calculation of Loads
(c) Calculation of Resistance
(d) Practical Application
(e) Details of Construction

Review of Theory. Although it has been assumed that the student has had some training in the theory of design, this subject is briefly reviewed.

Calculation of Loads. The calculation of loads on steel members is usually the most laborious part of designing. This work has to be done in each individual case, as it is not possible to standardize the loads which are applied to structures. Accurate data as to the weights of the materials of construction which must be supported by the steel framework are not always available; in fact, the weights of certain materials, as furnished by different manufacturers, vary considerably. The live, or imposed, loads must generally be assumed or approximated from prospective conditions of use which may be more or less uncertain. Consequently, this branch of the study involves not only careful computation, but the exercise of judgment.

Calculation of Resistance. The calculation of resistance of steel members to the loads applied is also a laborious matter when a start must be made from the beginning, but the steel construction has been so standardized that the number of sizes of material used is relatively small. Tables are available, giving the properties and resistance factors of these sections, so that it is usually an easy matter to design the section required for a given situation after the loads have been computed. This statement does not apply very generally to built-up sections such as plate girders and columns, as these members have been standardized only to a limited extent. Consequently, it is necessary for the designer to be able to compute the resistance of the member, having given only its dimensions and the permissible unit loads. Even in the case of I-beams there are many cases where the work must go back to the fundamental relations; as, for example, in cases where holes are punched in the tension flange of a beam at the point of maximum bending moment, or where a portion of the flange is cut away.

Practical Application. Numerous examples are worked out to illustrate the principles and methods covered by the text, and similar problems are submitted for solution. The examples and problems are taken from actual construction work, as it is believed that they are more useful and interesting than abstract illustrations.

Details of Construction. This section of the work explains the usual methods used in detailing the connections of steel members to each other and is illustrated by numerous drawings.

Reference Books. Tables giving the properties of steel sections and data giving the strength of steel members are given in the handbooks published by the steel manufacturers. These books are so convenient for reference and so easily obtainable that no attempt is made to repeat in this text the tables and data given in them, the supposition being that the reader either has one or will provide himself with one of these handbooks. References are repeatedly made to the handbooks and, as far as practicable, are made in general terms, so that any one of the reference books may be used. This is an important point, as these reference books are being revised from time to time and the one in use at the present time might be supplanted in a year or two by one of another manufacturer which is more up-to-date. Handbooks are published by The Cambria Steel Company, Johnstown, Pa.; Carnegie Steel Company, Pittsburgh, Pa.; Jones and Laughlins, Pittsburgh, Pa.; and Bethlehem Steel Company, South Bethlehem, Pa.

In addition to the handbooks there are a number of other reference books available for special purposes that can be purchased through the book stores. They are not essential for this study, but are of considerable use to designers. They will be referred to in the text in connection with the special features to which they relate.

Tables. The tables given in reference books are generally reliable; nevertheless, errors do occur in them and it is prudent to check them with the formulas sufficiently to make sure that they are computed on a correct basis, or *that the user understands the basis on which they are computed.* As an illustration of the latter point, attention is called to the fact that some tables of strength are stated in *tons* and others in *thousands of pounds.* Of course the heading of the table should show this, but special care should be taken to make sure which is used. A designer may be using a table for beams given in tons and a table for columns given in thousands of pounds, in which case it would be very easy to get columns designed only half strong enough or beams with twice the necessary strength. Similarly, there is a chance for confusion between moments expressed in foot-pounds and moments expressed in inch-pounds. Also there is a chance for error in using the weight per lineal foot of a section when it is intended to use the cross-sectional area, or *vice versa.* This matter is given further consideration later.

PROBLEM

Refer to the handbook and make a list of all the tables therein in which the strength is given in tons, and another list in which the strength is given in pounds or thousands of pounds.

If the handbook has been well edited, all tables will have the same basis. Make a careful search of the book to ascertain definitely its make-up in this relation. When there is occasion to use a different handbook, investigate immediately in the same manner.

PROBLEM

Refer to the handbook for all references and tables relating to moments. Make a list of all cases where moments are expressed in foot-pounds and another list of cases where they are expressed in inch-pounds.

Note that moments of inertia are always expressed in inches, so that in all cases where the *moments of inertia* of sections are used in computations, the bending moment must be expressed in inch-pounds. On the other hand, the resisting moments of beams are usually given in foot-pounds, and the bending moments must be computed in the same units.

PROBLEM

Select at random from the handbook twenty or more different sizes of angles, I-beams, plates, etc., and set down in parallel columns the area in square inches and the weight per lineal foot of each item.

Note that in each case the weight is 3.4 times the area. That is, a piece of steel having a cross-sectional area of one square inch weighs 3.4 pounds per lineal foot.

PROBLEM

What is the weight of one cubic foot of steel? Of one cubic inch of steel?

Factor of Safety. Older works and specifications dealing with steel construction frequently use the expression "factor of safety." It is used to express the ratio of the ultimate strength of the material to the safe working strength. In steel construction, this ratio is commonly stated to be 4, being based on the ultimate strength of 64,000 pounds per square inch and a working strength of 16,000 pounds per square inch. This expression is a misnomer and its use is to be discouraged, because it gives a wrong understanding of the facts and leads to an unwarranted sense of security. Later on in this treatise it is shown that the actual strength of steel work under loads continuously applied is only about one-half of the ultimate strength of the material, so that the real factor of safety is 2 where the nominal factor of safety is 4.

Further this expression has been used unscrupulously in arguments with owners to persuade them to use lighter steel work than standard practice permits; and, on the other hand, it has been used by the owners themselves without realizing the true meaning of the expression, in an attempt to reduce cost.

This expression is quite certain to come up from time to time in discussions with laymen and in such cases the distinction between the actual and the nominal factors of safety must be made clear.

Procedure in Furnishing Structural Steel. There are three steps in furnishing structural steel: *first,* the rolling of the plain material; *second,* the fabrication of the plain material into the conditions required for use; and *third,* the erection of the material in the structure.

The work of the rolling mill consists in rolling the steel sections of the sizes and lengths as required by the order. The work of the fabricating shop is to do the punching, cutting, assembling, riveting, and painting of the material as required for use in the structure. The work of the erector is to place the pieces in position in the structure and bolt or rivet them together. Some concerns perform all three of these steps; many perform only the second and third; and in still other cases the second and third steps may be performed by separate organizations. The owner may deal with a general contractor who undertakes to secure the performance of all three steps; or he may deal separately with a fabricating company and with an erection company. The former undertakes to deliver the fabricated material ready for erection, purchasing the material from the rolling mills. It is only in very rare instances that separate contracts are made for furnishing the plain material and for fabricating.

The design of the structural steel work is usually made by an architect, or by an engineer co-operating with the architect. The design drawings should show all the necessary dimensions of the structure, sizes of members, loads on the individual members, and details of connections other than those considered as standard. These drawings show the members assembled in their proper relations to each other. They must also show any connections required for attaching or supporting other construction materials.

As a part of the work of fabricating, working drawings must be prepared by the engineering department of the fabricating company, or by other engineers employed by it. These working drawings, or shop details, divide the work into individual members, and a complete drawing is made of each member, showing all dimensions, the position of rivets, and the exact location of the open holes required for connections with other members of the structure.

STRUCTURAL STEEL

METHODS OF MANUFACTURE

The procedure in the manufacture of structural steel sections from iron ore consists of the following operations: (1) smelting the iron ore and producing pig iron; (2) converting the pig iron into steel ingots; and (3) rolling the ingots into steel sections.

Iron Ore to Pig Iron. Iron ore is a chemical combination of iron and oxygen. It exists in several forms. Pure ore has a maximum of about 70 per cent of iron. The ores as mined are mixed with various substances, chiefly water, silica, and limestone, with small quantities of phosphorus, sulphur, titanium, manganese, etc., so that commercial ore contains only 50 per cent of iron, or even less.

Process of Smelting. The purpose of smelting the ore is to break down the chemical combination of iron and oxygen, and to eliminate the greater part of the impurities from the resulting metallic iron. This is accomplished by melting the ore in a blast furnace. The heat for melting the ore is supplied by coke, and the melting point is brought to a lower temperature than otherwise would be required by mixing limestone with the ore. As the contents of the furnace melt, they drip down to the bottom where the molten iron separates from the molten slag by gravity, the iron, being heavier, settling to the bottom.

A section of a blast furnace and skip hoist is shown in Fig. 2. The skip or car at the bottom of the machine is loaded with ore, limestone, and coke from the bins; it is then hauled up the incline where the material is charged into the blast furnace. Fig. 3 shows a section through the bottom part of the furnace, which represents graphically the melting charge and the accumulation of iron and slag in separate layers at the bottom of the furnace. The blast of air required for burning the coke is admitted through the openings, called "tuyeres," near the bottom of the furnace.

The operation of the blast furnace is continuous from the time it is fired until it is shut down for repairs, or for other reasons. As the metal and slag accumulate at the bottom, they are drawn off, the metal into molds to form pigs, Fig. 4, and the slag to the dump. More material is added at the top of the furnace as the contents melt.

Pig Iron. The pig iron resulting from this operation contains 3 or 4 per cent of carbon; a small amount of sulphur which has been absorbed from the coke; about 4 per cent of silicon; and smaller quantities of manganese and phosphorus which remain from the ore.

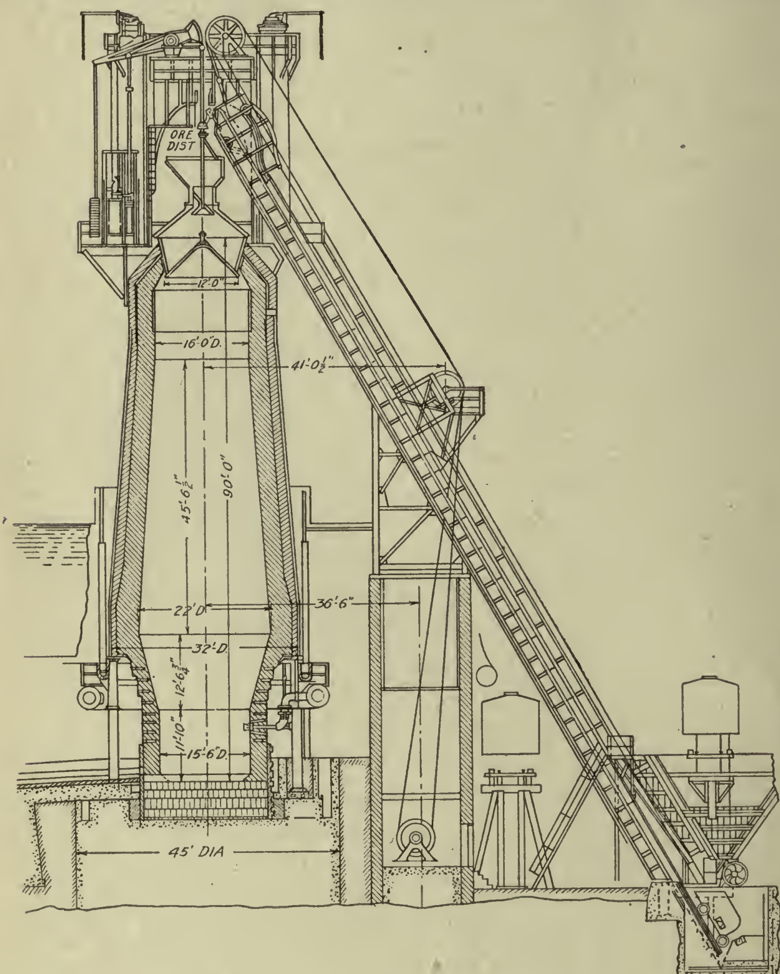

Fig. 2. Cross Section of Blast Furnace and Skip Hoist
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy McGraw-Hill Publishing Company

The iron may not be cast into pigs but may be maintained in a molten condition ready for the next operation, if the Bessemer process is used. In this case it is poured into a large vessel, called a "mixer," Fig. 5, which may hold as much as 500 tons. Heat can be applied to it if needed.

Pig Iron to Steel. The change from pig iron to steel consists of the reduction of the carbon to about 0.2 per cent and the elimina-

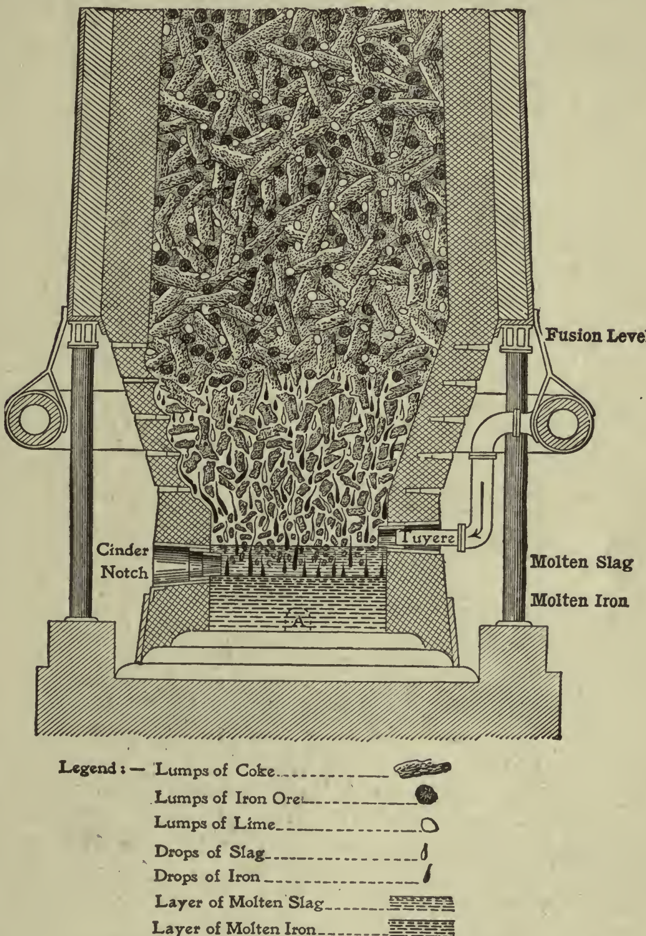

Fig. 3. Section Through Base of Furnace Showing Layers of Molten Iron and Slag with Unmelted Ingredients Above
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy McGraw-Hill Publishing Company

tion of impurities as fully as possible. There are two processes of doing this, the Bessemer and the Open Hearth. They are described in "Metallurgy of Iron and Steel"* as follows:

*By Bradley Stoughton, Copyright 1913, McGraw-Hill Publishing Company.

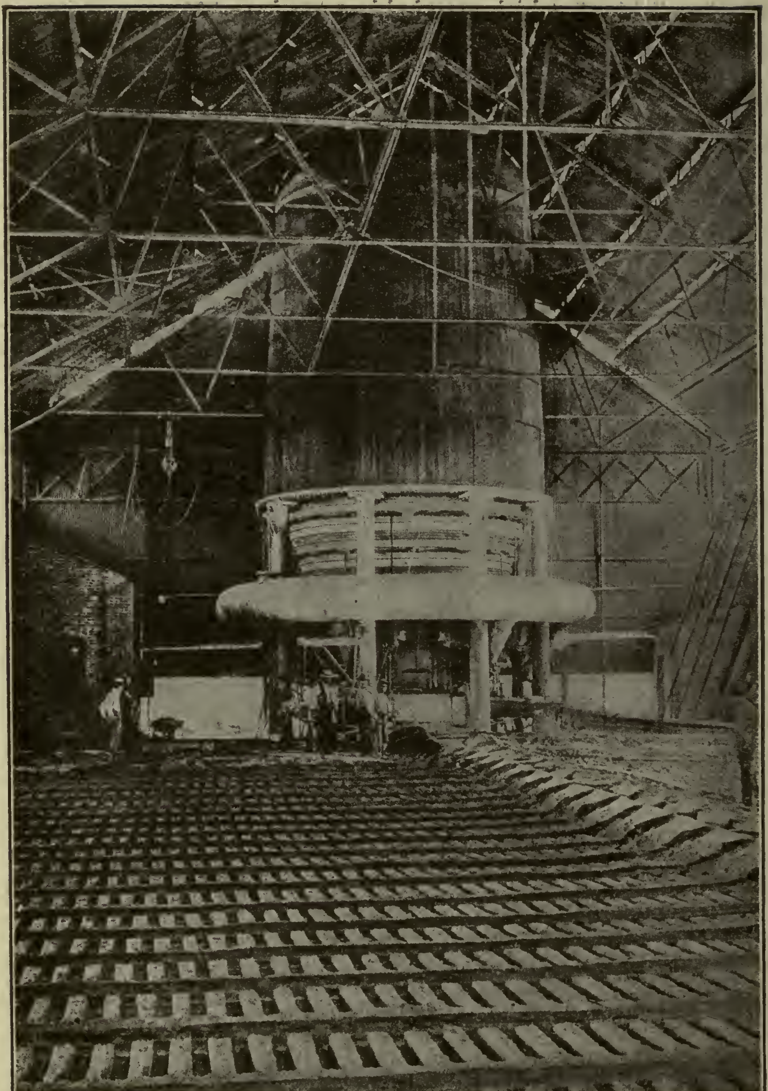

Fig. 4. Pig Beds
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy, McGraw-Hill Publishing Company

"*Bessemer Process.* In the Bessemer process, perhaps 10 tons of melted pig iron are poured into a hollow pear-shaped converter, Figs. 5 6 and 7, lined with silicious material. Through the molten

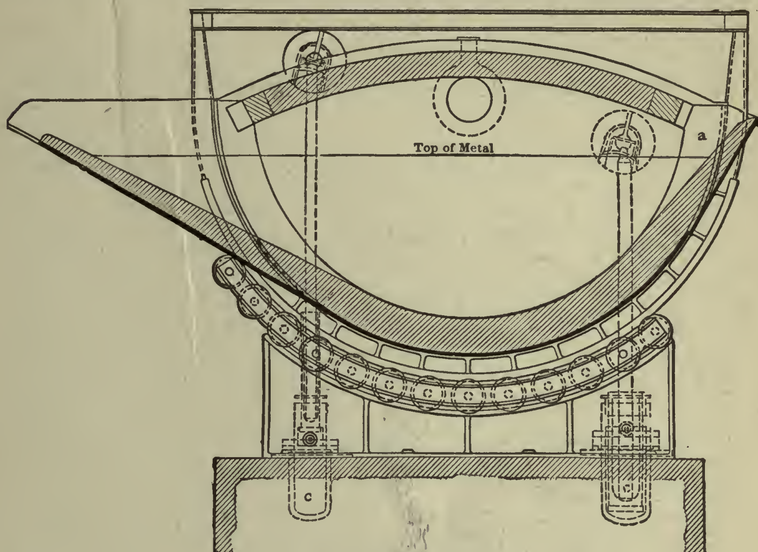

Fig. 5. Section Through a Mixer
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy, McGraw-Hill Publishing Company

material is then forced 25,000 cubic feet of cold air per minute. In about four minutes the silicon and manganese are all oxidized by the oxygen of the air and have formed a slag. The carbon then begins to oxidize to carbon monoxide, CO, and this boils up through the metal and pours out of the mouth of the vessel in a long brilliant flame, Fig. 8. After another six minutes, the flame shortens or 'drops'; the operator now knows that the carbon has been eliminated to the lowest practicable limit, say 0.04 per cent, and the operation is stopped. So great has been the heat evolved by the oxidation of the impurities that the temperature is now higher than it was at the start, and we have a white-hot liquid mass of relatively pure metal. To this is added a carefully calculated amount of carbon to produce the desired degree of strength or hardness, or both; also about 1.5 per cent of manganese and 0.2 per cent of silicon. The manganese is added to remove from the bath the oxygen with which it has become charged during the ope-

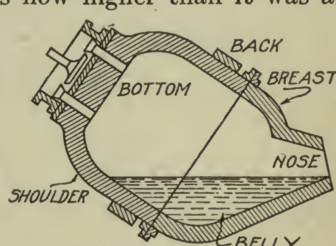

Fig. 6. Parts of Converter
Courtesy McGraw-Hill Publishing Company

ration and which would render the steel unfit for use. The silicon is added to get rid of the gases which are contained in the bath. After adding these materials, or "recarburizing" as it is called, the metal is poured into ingots which are allowed to solidify, and then rolled, while hot, into the desired sizes and forms. The characteristics of the Bessemer process are: (a) great rapidity of purification, say ten minutes per "heat"; (b) no extraneous fuel is used; and

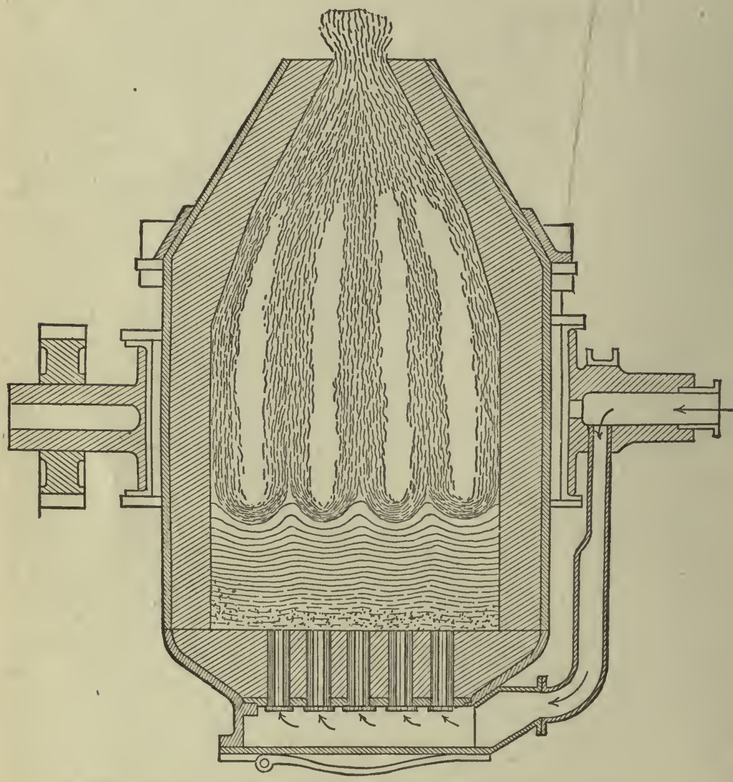

Fig. 7. Section Through Bessemer Converter While Blowing
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy McGraw-Hill Publishing Company

(c) the metal is not melted in the furnace where the purification takes place.

"*Acid Open-Hearth Process.* The acid open-hearth furnace is heated by burning within it gas and air, each of which has been highly preheated before it enters the combustion chamber. A section of the furnace is shown in Fig. 9. The metal lies in a shallow pool on the long hearth, composed of silicious material, and is heated by radiation from the intense flame produced as described. The impurities are oxidized by an excess of oxygen in the furnace gases over that necessary to burn the gas. This action is so slow, however, that the 3 to 4 per cent of carbon in the pig iron takes a

Fig. 8. A Bessemer Blow
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy McGraw-Hill Publishing Company

long time for combustion. The operation is therefore hastened in two ways: (a) iron ore is added to the bath, and (b) the carbon is diluted by adding varying amounts of cold steel scrap. The steel scrap is added to the furnace charge at the beginning of the process, and it takes from 6 to 10 hours to purify a charge, after which we recarburize and cast the metal into ingots. The characteristics of the open-hearth process are: (a) long time occupied in purification; (b) large charges treated in the furnace (modern practice is usually 30 to 70 tons to a furnace); (c) at least part of the charge melted in the purification furnace; and (d) furnace heated with preheated gas and air, Fig. 10.

"*Basic Open-Hearth Process.* The basic open-hearth operation is similar to the acid open-hearth process, with the difference that we

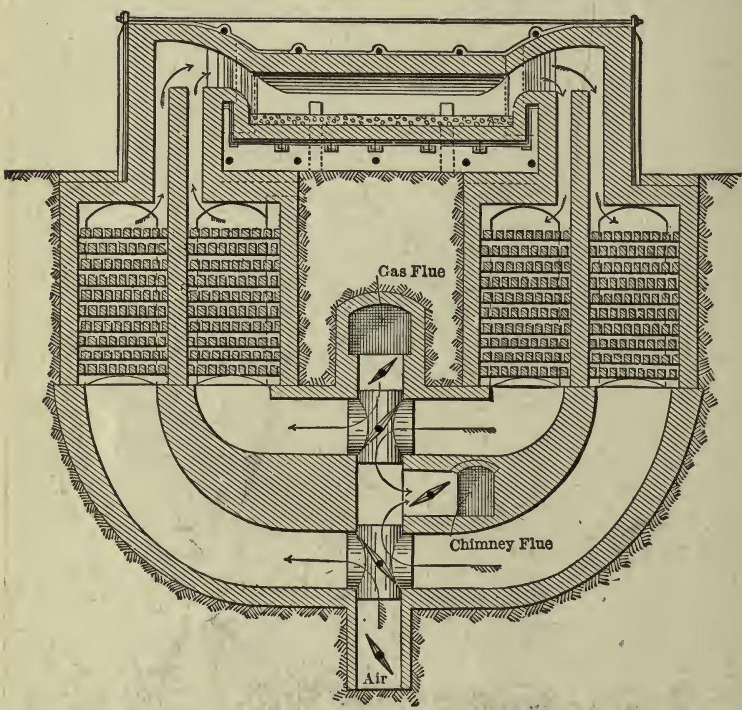

Fig. 9. Section of Regenerative Open-Hearth Furnace
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy McGraw-Hill Publishing Company

add to the bath a sufficient amount of lime to form a very basic slag. This slag will dissolve all the phosphorus that is oxidized, which an acid slag will not do. We can oxidize the phosphorus in any of these processes, but in the acid Bessemer and the acid open-hearth furnaces the highly silicious slag rejects the phosphorus, and it is immediately deoxidized again and returns to the iron. The characteristics of the basic open-hearth process are the same as those of the acid open-hearth with the addition of: (e) lime added to

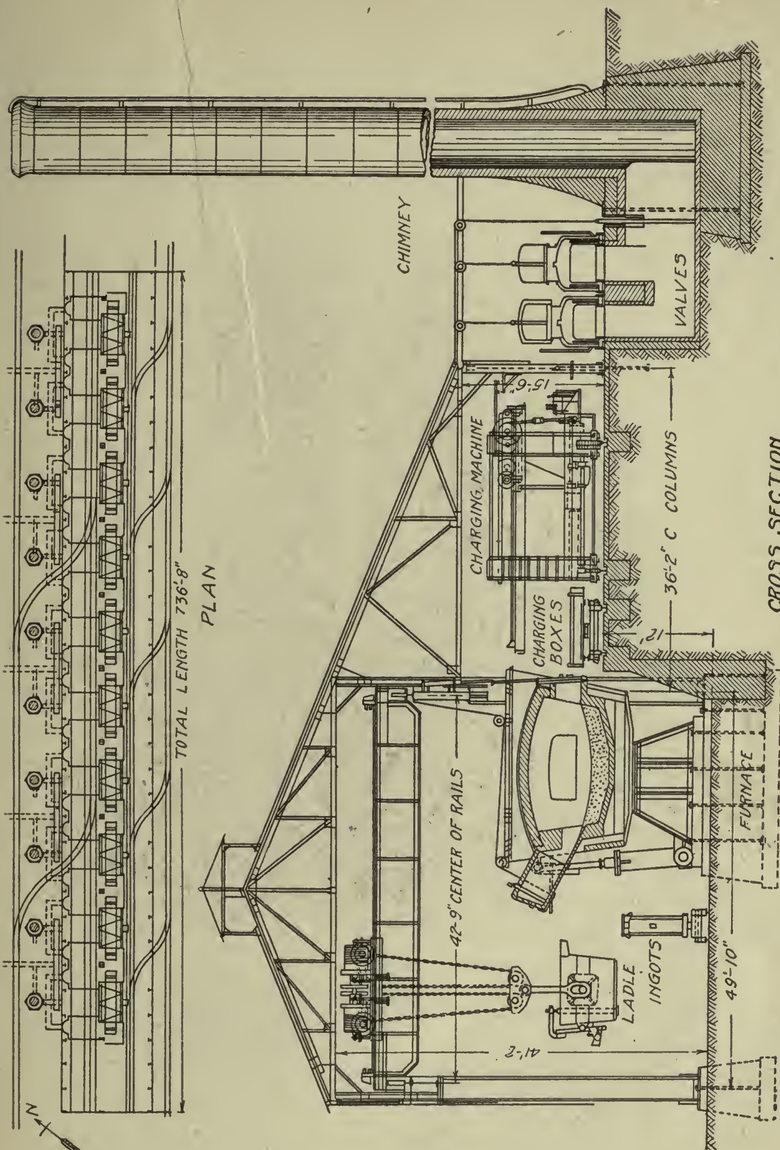

Fig. 10. Plan and Section of Open-Hearth Furnace Plant
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy McGraw-Hill Publishing Company

produce a basic slag; (f) hearth lined with basic, instead of silicious, material, in order that it may not be eaten away by this slag; and (g) impure iron and scrap may be used, because phosphorus, and, to a limited extent, sulphur can be removed in the operation."

Rolling the Ingots. The steel in the ingot is in its final condition as to chemical composition, Figs. 11 and 12, and must now be

Fig. 11. Steel Ingots Incased in the Molds and Resting on Car
From Stoughton's Metallurgy of Iron and Steel
Courtesy McGraw-Hill Publishing Company

worked into the shapes required for structural uses. This is done by passing the steel between rolls.

Rolls are used in pairs, called a "two-high mill", as shown in Fig. 13, or in sets of three, called a "three-high mill", as shown in Fig. 14. As the piece goes through the same mill several times, the two-high mill must be reversed for each pass or else the piece must be taken over or around the mill between the successive passes. These disadvantages are eliminated by the use of the three-high mill, in which the rolls rotate continuously and work is done on the piece as it passes back and forth.

Blooming. Before going to the rolls, the ingot is placed in a furnace, called the "soaking pit", in which it is heated to a high temperature. In passing between the rolls, Fig. 13, a heavy pressure is exerted on the metal, which reduces it in thickness, increases it in width to some extent, and extends it greatly in length. If the material is destined to be made into plates, it is rolled into a slab in the first set of rolls; if it is for structural shapes, the ingot will be turned alternately from side to edge in passing through the rolls so that it will be kept approximately square in section until it is reduced to the proper size for beginning to form the shape. At this stage it is called a "bloom" and the rolls are called "blooming rolls", Fig. 15.

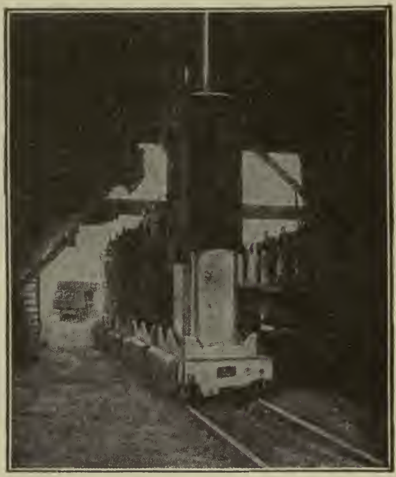

Fig. 12. Stripping the Ingots
Courtesy McGraw-Hill Publishing Company

Fig. 13. Action on Steel in "Two-High" Mill
Courtesy McGraw-Hill Publishing Company

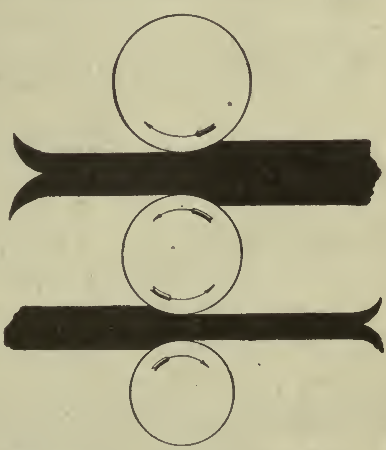

Fig. 14. Action on Steel in "Three-High" Mill
Courtesy McGraw-Hill Publishing Company

Roughing and Finishing Rolls. The next step is to pass the steel through the roughing rolls. These rolls are grooved in such

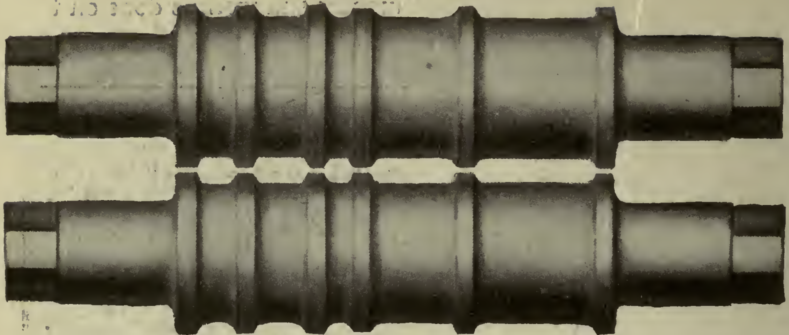

Fig. 15. "Two-High" Blooming Rolls
Courtesy Seaman, Sleeth Company

a way that the successive passes gradually develop the metal toward the required shape. Finally it goes through the finishing rolls which bring the section to the required shape and size. This process is clearly illustrated by Figs. *16, 17, 18, 19, and 20.

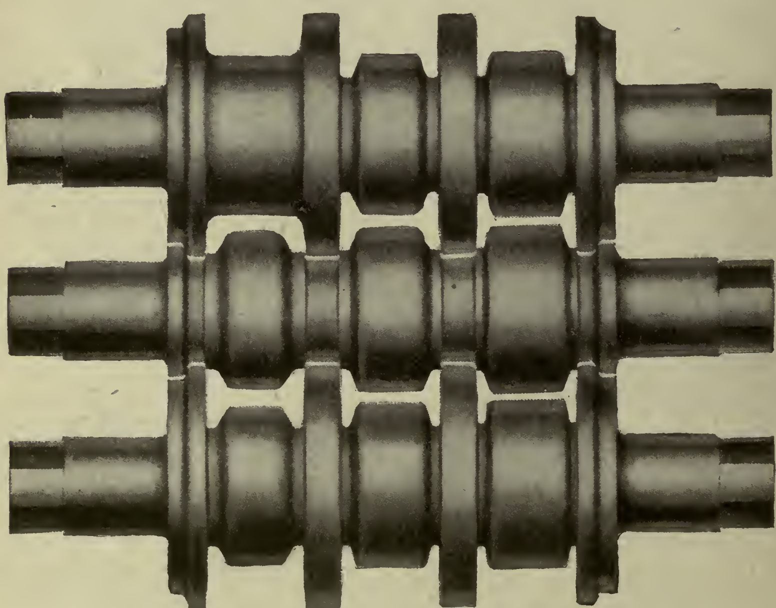

Fig. 16. "Three-High" I-Beam Roughing Rolls
Courtesy Seaman, Sleeth Company

*Catalogue of Phoenix Roll Works, by permission.

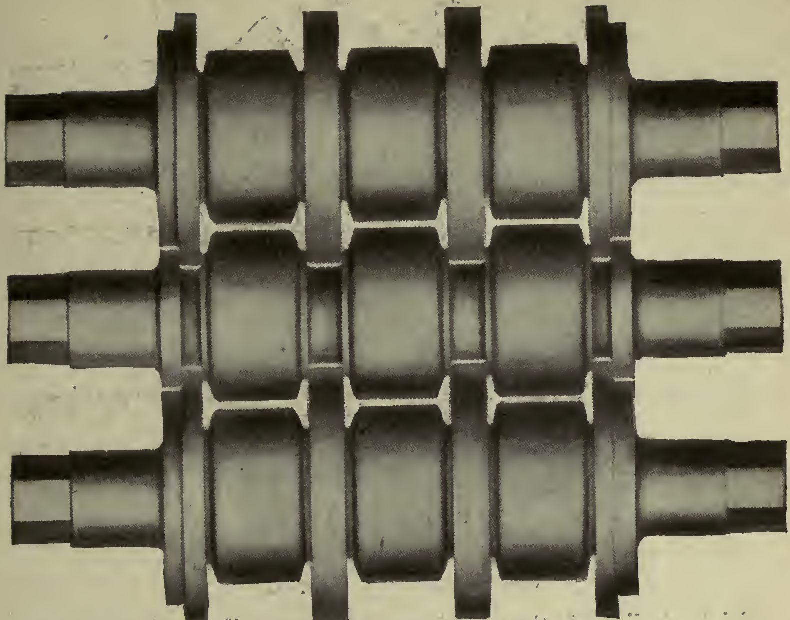

Fig. 17. "Three-High" I-Beam Finishing Rolls
Courtesy Seaman, Sleeth Company

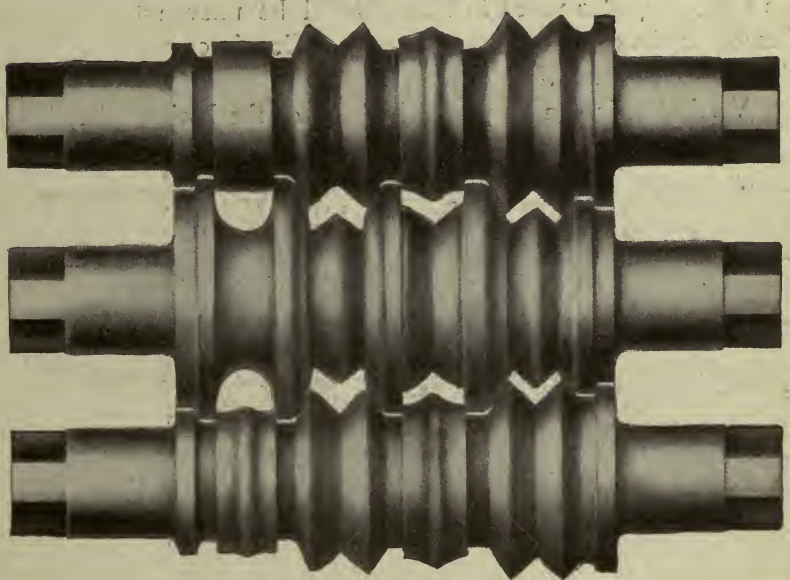

Fig. 18. "Three-High" Equal Angle Roughing Rolls
Courtesy Seaman, Sleeth Company

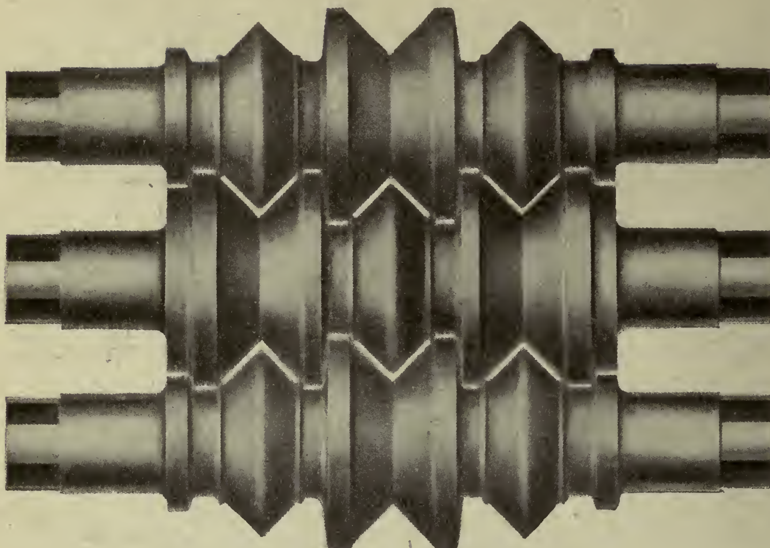

Fig. 19. "Three-High" Equal Angle Finishing Rolls
Courtesy Seaman, Sleeth Company

Plate Rolls. A three-high set of plate rolls is shown in Fig. 21. There is nothing to control the width of the plates, therefore the edges of plates rolled in this mill will be uneven and must be sheared to the correct width after the rolling is completed. Such plates are known as "sheared plates."

Vertical rolls can be placed in front of the horizontal rolls to

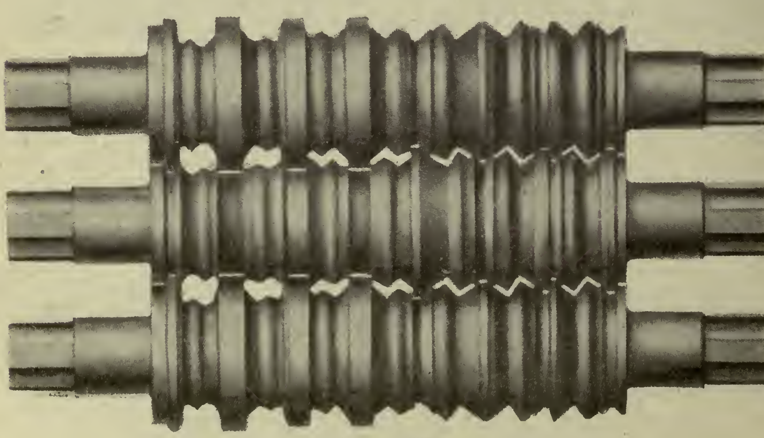

Fig. 20. "Three-High" Z-Bar Rolls
Courtesy Seaman, Sleeth Company

control the width, as shown in the left-hand view, Fig. 22. Such a mill is called a "Universal Mill" and the plates produced by it are

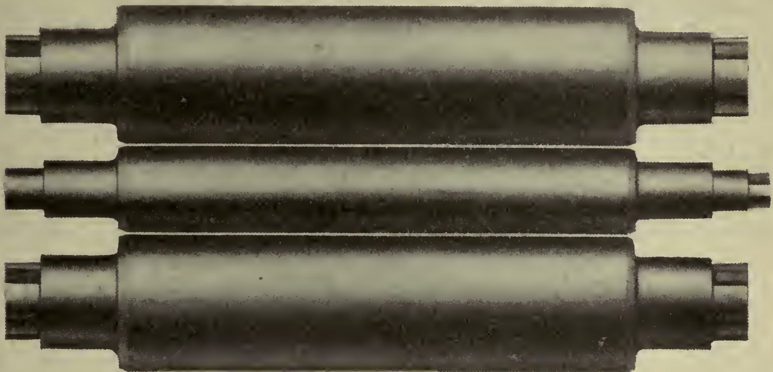

Fig. 21. "Three-High" Chill Plate Rolls
Courtesy Seaman, Sleeth Company

called "Universal Mill plates," or edged plates. Fig. 22 is a special form known as the Grey mill and is used by the Bethlehem Steel Company for making I-beams and column sections. Fig. 23 is a 3-high Universal Mill manufactured by the United Engineering and Foundry Company, Pittsburgh.

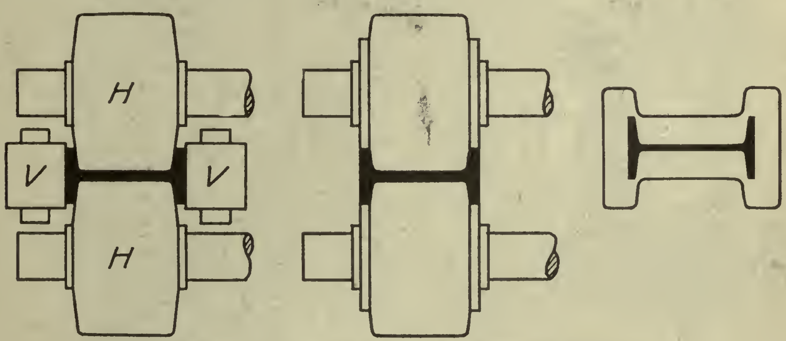

Fig. 22. Universal Mill for Rolling Bethlehem Beams

STEEL SECTIONS—ADAPTABILITY AND USE

Classification of Sections. Structural steel members are generally designated by the shapes of their cross sections. Thus a member whose cross section has the shape of a capital letter I is called an I-beam. The other important sections are channels, angles, zees,

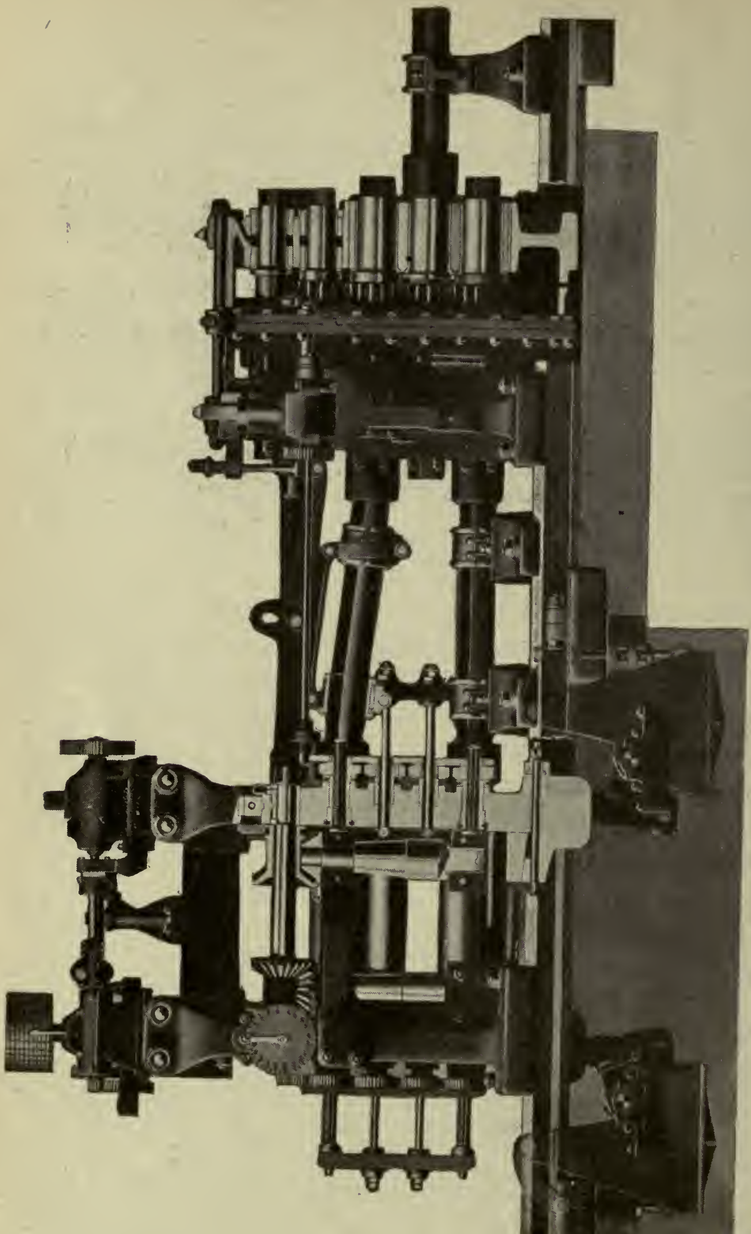

Fig. 23. 48-Inch "Three High" Universal Mill
Courtesy United Engineering & Foundry Company

tees, and H-sections, whose shapes are indicated by the names. Round and square members are called "rods" and "bars". Flat members six inches wide and less are usually designated as "bars" or "flats". Flat members wider than six inches are designated as "plates". Structural sections are frequently designated as "plates" and "shapes". In general, the structural shapes are standard.

Standard Sections. The shapes in common use conform to the standards of the Association of American Steel Manufacturers. These standard shapes as made by the various manufacturers are identical in dimensions and weights; therefore, in designing it is only necessary to specify the sections and not the name of the manufacturer.

Special Sections. In addition to the standard sections, most manufacturers make some special sections. Some of these are now so common that they are as available as standard sections, but generally it is advisable for the designer to give the name of the manufacturer in specifying them. The handbooks indicate which sections are standard and which are special.* The designer should generally use only standard sections. This matter is given full consideration elsewhere in this text. Use the handbook for constant reference in the following discussion of the sections.

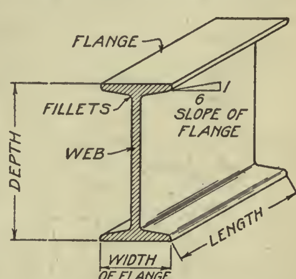

Fig. 24. Details of Standard I-Beam Section

I-Beams. *Standard Sections.* An I-beam, Fig. 24, is designated by the depth and the weight per lineal foot, thus:

12″ I $31\frac{1}{2}$#

The standard depths are 3, 4, 5, 6, 7, 8, 9, 10, 12, 15, 18, 20, and 24 inches, respectively. For each depth there are several standard weights. Most of the mills also make some special weights, viz:

12″ deep weighing 40 to 55#
15″ deep weighing 60 to 80#
15″ deep weighing 80 to 100#
20″ deep weighing 80 to 100#

*The 1903 edition of the "Carnegie Handbook" used the term *standard* in relation to beams and channels to apply to the minimum weight of each size. It is preferable to limit the use of the term to the sections adopted by the Association of American Steel Manufacturers.

Carnegie Sections. The Carnegie Steel Company rolls some additional sizes of special beams which are similar to the standard beams, as follows:

24″ deep weighing 105 to 115#
18″ deep weighing 75 to 100#

It also rolls special sizes of certain depths which are lighter than the minimum weight standard beams. They are as follows:

*10″ I 22#	18″ I 46#
12″ I 27½#	21″ I 57½#
15″ I 36#	24″ I 69½#
27″ I 83#	

A distinctive feature of these beams is that the fillets connecting flange to web form a compound curve instead of a simple curve as in the standard beams.

Bethlehem Sections. The Bethlehem Steel Company† makes a series of special I-beams ranging in depth from 8 to 30 inches. The minimum weights of these beams are about 10 per cent less than the minimum weights of the corresponding standard beams. The section is so designed that the theoretical strength of the minimum section is about the same as that of the standard section. This is accomplished by putting less metal in the web and more in the flanges. Fig. 25 gives the dimensions of the Bethlehem 15″ I 38#. Comparison with the corresponding standard beam shows:

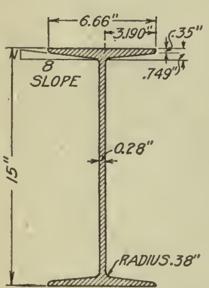

Fig. 25. Bethlehem Special Section 15″ I 38#

	15″ I 38#	15″ I 42#
Web thickness	.29″	.41″
Flange width	6.66″	5.50″
Moment of inertia	442.60	441.80

The Bethlehem Company also makes a series of girder beams ranging in depth from 8 to 30 inches. These beams are much

*Apply to the nearest office of the Carnegie Steel Company or the Illinois Steel Company, for a circular giving the properties of these beams, or see "Pocket Companion," Carnegie Steel Company, 1913.

†Complete data are given in the Company's handbook.

heavier than either the standard beams or the Bethlehem special beams and the flanges are also much wider. Fig. 26 gives the dimensions of the Bethlehem girder 15″×73#.

Efficiency of Minimum Sections. Note in the handbook that the weights of beams of a given depth are grouped. The beams in a group are rolled with the same rolls, the minimum section being produced when the rolls are set close together, and the heavier sections being made by spreading the rolls. In this change the depth remains constant, while the web is thickened and the flanges widened. In Fig. 27, the shaded portion represents the minimum section, and the unshaded portion represents the metal added to produce the heavier section. From this it is clear that most of the added metal is in the web, and is not placed to such good advantage as the metal in the minimum section. The increased strength is not nearly so great as the increased weight. For example, compare 15″ I 42# with 15″ I 60# of the same group. The increase in weight is 18 pounds, or $\frac{18}{42}=43\%$. The increase in strength as indicated by the change in the moment of inertia from 441.8 to 538.6 is 96.8, or $\frac{96.8}{441.8}=22\%$. Thus it appears that the minimum weight of each group is the most efficient. As a consequence the range in weight from a given set of rolls is limited to about 20 pounds. When a greater range is required for a given depth of beam, more than one set of rolls is used. Now compare the standard 15″ I 60# and the special 15″ I 60#. Their respective moments of inertia are 538.6 and 609.0. The difference is 70.4, or $\frac{70.4}{538.6}=13\%$. This illustrates the advantage of having the additional set of rolls. More than one set of rolls is provided for 12-inch, 15-inch, 18-inch, 20-inch and 24-inch beams.

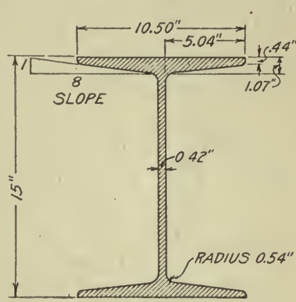

Fig. 26. Bethlehem Girder 15″×73#

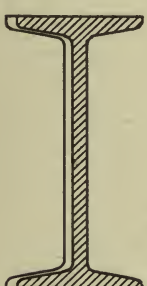
Fig. 27. Showing Method of Increasing Section of I-Beams

Problem

Make full-size drawings on tracing paper of the following sections:

Standard	15″	I	42#
Standard	15″	I	55#
Special	15″	I	60#
Special	15″	I	80#
Special	15″	I	100#
Carnegie	15″	I	36#
Bethlehem	15″	I	38#
Bethlehem	15″	GB	73#

Superimpose these tracings and note the difference in thickness of web, width of flange, and shape of fillets.

Characteristics and Uses. An inspection of an I-beam section shows it is much stiffer in one direction than in the other. The section is designed to resist bending in one direction only, i. e., in the plane of the web of the beam. The I-beam is used almost exclusively for this purpose, though to a limited extent it is used in built-up columns. When used in a column, it is economical only when combined with other sections to give stiffness in both directions. It is sometimes used alone as a column when the limitations of space offset the lack of economy in weight.

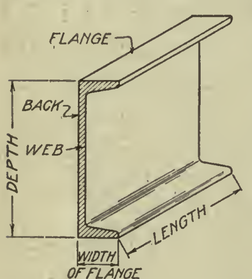

Fig. 28. Details of Channel Section

Beams less than 6 inches deep are not often used in the framework for buildings. On many jobs the minimum is 8 inches.

Channels. *Standard and Special Sections.* A channel, Fig. 28, is designated by the depth and the weight per lineal foot, thus:

15″ ⊏ 33#

The standard depths are 3, 4, 5, 6, 7, 8, 9, 10, 12, and 15 inches, respectively. For each depth there are several weights. A number of special sizes and weights are made but they are not much used for structural purposes. The Cambria Steel Company makes a group of channels 18 inches deep, weighing from 45 to 60 pounds.

The weights of channels are increased in the same manner as the weights of beams, Fig. 29, and the comments regarding beams in this respect apply to them.

Characteristics and Uses. Channels, like beams, are much stronger in one direction than in the other. This makes them suitable for use as beams when the loads are applied in the plane of the web. However, they are not so economical as I-beams and require more lateral support to keep them from buckling. Hence, they are not used for this purpose except when there is some condition which makes them specially suitable. This occurs around wellholes in floors, against walls, where nailing strips are to be bolted on, in wall spandrels or lintels, etc.

Fig. 29. Showing Method of Increasing Section of Channels

The most important use of channels is in the construction of columns and truss members. For this purpose they are used in pairs connected together with lacing, tie plates, or cover plates. They are also used to some extent for girder flanges and for many miscellaneous purposes.

Angles. *Standard and Special Sections.* There are two styles of angles: angles with equal legs and angles with unequal legs; Fig. 30. An angle is designated by the lengths of the legs and the thickness or the weight per lineal foot, thus:

$$\text{L}\ 4'' \times 4'' \times \tfrac{5}{8}''$$
$$\text{or}\ \ \text{L}\ 4'' \times 4'' \times 15.7\#$$

$$\text{L}\ 6'' \times 3\tfrac{1}{2}'' \times \tfrac{3}{8}''$$
$$\text{or}\ \ \text{L}\ 6'' \times 3\tfrac{1}{2}'' \times 11.7\#$$

The standard sizes of angles with equal legs are 1½, 2, 2½, 3, 3½, 4, 6, and 8 inches, respectively. There are a number of special sizes, the most important of which is 5 inches. The 1½-inch angle is seldom used in structural work.

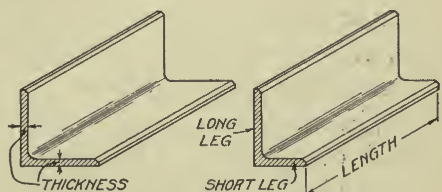

Fig. 30. Details of Angle Sections

The standard sizes of angles with unequal legs are 2½″ × 2″, 3″ × 2½″, 3½″ × 2½″, 3½″ × 3″, 4″ × 3″, 5″ × 3″, 5″ × 3½″, 6″ × 3½″, 6″ × 4″. The important special sizes usually obtainable are 3″ × 2″, 7″ × 3½″, 8″ × 6″.

Each size of angle is furnished in several thicknesses varying by $\frac{1}{16}$ inch. Although some of the smaller sizes of angles are made in less thickness than $\frac{1}{4}$ inch, this is the minimum that should be used for structural purposes. On important work the minimum should be $\frac{3}{8}$ inch. The minimum and maximum thickness for the several sizes are given in the handbook and need not be repeated here.

Angles are increased from the minimum thickness by spreading the rolls. In Fig. 31 the minimum thickness is shaded and the added metal unshaded. As the thickness is increased, a corresponding amount is added to the length of each leg. In the case of larger sizes, some mills use two sets of rolls, as has been described for I-beams. This additional length of the legs of angles must be taken into account in allowing for clearance. The actual length of legs for any angle is easily computed, thus: L $3'' \times 3'' \times \frac{5}{8}''$; minimum thickness for this size $\frac{1}{4}''$, increase over minimum $\frac{3}{8}''$, length of leg $3'' + \frac{3}{8}'' = 3\frac{3}{8}''$.

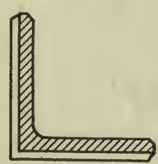

Fig. 31. Showing Method of Increasing Section of Angles

PROBLEM

Compute the actual lengths of legs for the maximum thickness of all the standard and special angles listed in the handbook. Assume a second set of rolls is used on the following sizes: $5''\times4''\times\frac{9}{16}''$; $4''\times4''\times\frac{1}{2}''$; $3\frac{1}{2}''\times3\frac{1}{2}''\times\frac{1}{2}''$; $5''\times3\frac{1}{2}''\times\frac{1}{2}''$; $5''\times3''\times\frac{1}{2}''$; $4''\times3\frac{1}{2}''\times\frac{1}{2}''$; $4''\times3''\times\frac{1}{2}''$; $6''\times3\frac{1}{2}''\times\frac{9}{16}''$; $6''\times4''\times\frac{9}{16}''$; $5''\times5''\times\frac{9}{16}''$; $6''\times6''\times\frac{11}{16}''$; $7''\times3\frac{1}{2}''\times\frac{3}{4}''$; $8''\times6''\times\frac{3}{4}''$; $8''\times8''\times\frac{3}{4}''$.

Record the results in the handbook in the tables of "Properties."

The results in the above problem may not agree with the sizes of angles furnished by the various mills but will be sufficiently exact for the uses of the designer.

Characteristics and Uses. Angles are the most adaptable of the structural sections. They are used with plates or other shapes in built-up members, such as columns, plate girders, etc.; for connecting members together, as beams and girders to columns; as beams for special conditions of loading, as lintels; singly or in pairs as struts; singly or in pairs as tension members.

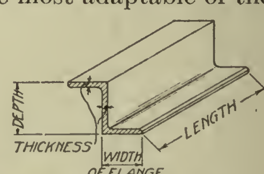

Fig. 32. Details of Zee Bar

Zees. *Standard Sections.* A Zee, Fig. 32, is designated by its nominal depth and thickness, thus:

$$Z\ 3''\times\tfrac{1}{4}''$$

The sizes listed by the Carnegie Steel Company are 3, 4, 5, and 6 inches, respectively. The thicknesses vary by $\frac{1}{16}$ inch. The minimum and maximum thicknesses are:

for 3″ Z, $\frac{1}{4}$″ and $\frac{9}{16}$″
for 4″ Z, $\frac{1}{4}$″ and $\frac{3}{4}$″
for 5″ Z, $\frac{5}{16}$″ and $\frac{13}{16}$″
for 6″ Z, $\frac{3}{8}$″ and $\frac{7}{8}$″

Zees are increased in thickness by spreading the rolls. In Fig. 33 the shaded portion indicates the minimum section, and the unshaded part the additional section. The thickness of its web and flanges are increased equally, and thereby the depth of web and width of flange are increased by the same amount. Three sets of rolls are used for each depth, so that the overrun is $\frac{1}{16}$ inch for 3-inch zees and $\frac{1}{8}$ inch for larger sizes.

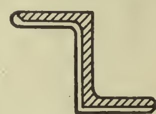

Fig. 33. Showing Method of Increasing Section of Zees

Uses. Zee bars have been used extensively for columns, but they are rapidly becoming obsolete and should not be used unless there is some special reason for so doing.

Tees. *Standard Sections.* A Tee, Fig. 34, is designated by the width of flange, length of stem, and weight per lineal foot, thus:

T 4″ × 3″×9.3# T 3″ × 4″×9.3#

always giving the width of flange first.

Some recent handbooks do not list tees. The sizes that have been available range from 1″ × 1″ × 1.0# to 5″ × 3″ × 13.6# with more than 50 intermediates. These are listed and their properties given in the Carnegie Steel Company's "Pocket Companion", 1913 edition.

Characteristics and Uses. As indicated above tees are going out of use, and as the demand decreases they will become more

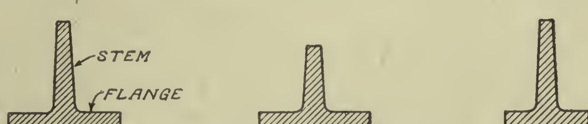

Fig. 34. Typical Tee Sections

difficult to obtain. The section is not an economical one for the common uses of structural steel. It is not efficient as a beam or as a strut, and is not suited for use in built-up sections.

It is well adapted for supporting book tile in ceiling and roof construction, Fig. 35. In cases where the T-section is needed to

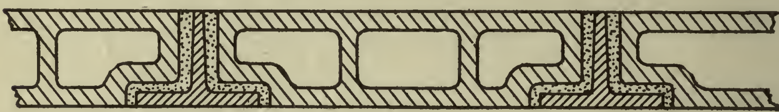

Fig. 35. Section Showing Tees Supporting Book Tile

meet any special condition it can be made up of two angles placed back to back. In this manner a large variety of tees can be made.

Plates. *Standard Sizes.* A Plate, Fig. 36, is designated by its width and thickness, thus:

Pl. 48″ × $\frac{7}{16}$″

or by its width and *weight per square foot*, thus:

Pl. 36″ × 10.2#

The former method is used on design drawings for structural steel work, and the latter on mill orders and shop details, also on design drawings for tank work.

Plates are made in thicknesses varying by $\frac{1}{16}$ inch from $\frac{3}{16}$ inch up to 2 inches. Steel plates thinner than $\frac{3}{16}$ inch are called "sheets" and are not used for structural work. The minimum thickness commonly used is $\frac{1}{4}$ inch, and on many jobs nothing less than $\frac{3}{8}$ inch is permitted. Plates thicker than 1 inch are seldom used on account of

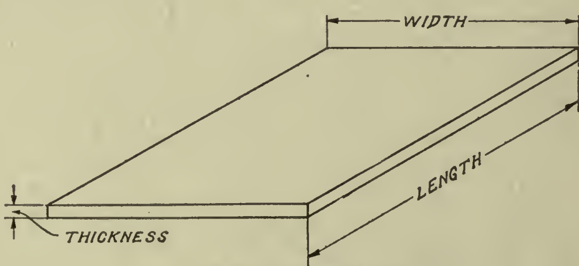

Fig. 36. Rolled Steel Plate

difficulty in punching. When a greater thickness is needed, it is made up of two or more plates.

Styles. There are two styles of plates: the Universal Mill Plate, or Edged Plate, and the Sheared Plate.

The Universal Mill Plate is rolled to exact width, the width being controlled by a pair of vertical rolls as previously described and illustrated, Fig. 22. They vary in width by intervals of 1 inch from 6 inches to 48 inches.

Sheared plates, as the name indicates, are sheared to required width after rolling. The stock sizes range in width from 24 inches to 132 inches in intervals of 6 inches, but they can be furnished in any intermediate width, even in fractions of an inch.

The extreme lengths of plates that can be furnished are given in the handbooks. This data should be consulted to determine

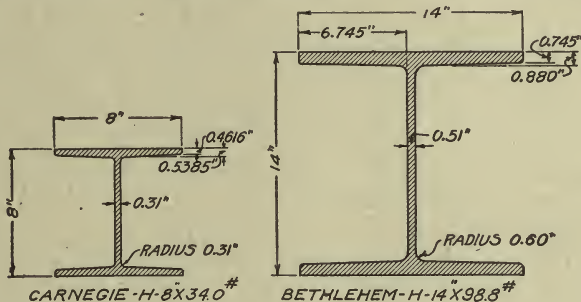

Fig. 37. Typical H-Sections

whether the required lengths can be obtained. In many cases the web plates of girders must be spliced on this account.

Plates alone are not used for structural members. They are used in built-up members, such as columns and girders; for web and cover plates; and to connect members together.

H-Sections. The H-section, Fig. 37, is designated by the name of the maker, the depth, and the weight per lineal foot, thus:

Carnegie 8″ H 34.0#
Bethlehem 14″ H 98.8#

The H-section is not standard. At this time it is made only by the Carnegie Steel Company and the Bethlehem Steel Company. The Carnegie H's* are

8″ H 34.0#	5″ H 18.7#
6″ H 23.8#	4″ H 13.6#

There is but one weight for each size.

*Apply to the nearest office of the Carnegie Steel Company, or the Illinois Steel Company, for circular giving properties, or see Carnegie Steel Company's Pocket Companion, 1913 edition.

The nominal sizes of the Bethlehem H-sections are 8, 9, 10, 11, 12, 13, and 14 inches, respectively. The actual sizes range from $7\frac{7}{8}$ inches to $16\frac{7}{8}$ inches in intervals of $\frac{1}{8}$ inch. The extreme weights are 34.6 pounds and 291.2 pounds per lineal foot.

The H-sections are designed for use as columns and struts. They are not intended to be used in built-up members, except a special section which is designed to be increased by adding flange plates.

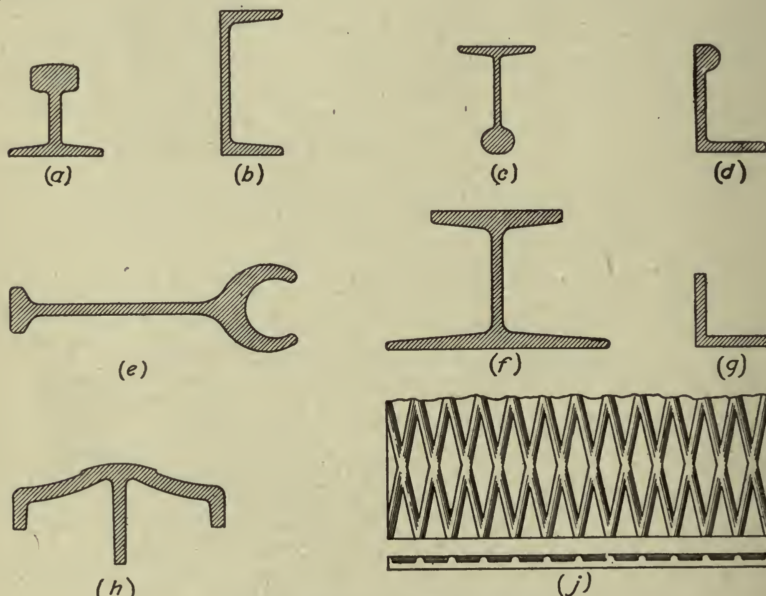

Fig. 38. Miscellaneous Special Sections

Miscellaneous Sections. In addition to the regular structural sections just described there are a number of special sections, Fig. 38, with which the designer should be familiar, viz:

(a) Railroad Rails
(b) Wide-Flanged Channels
(c) Bulb Beams
(d) Bulb Angles
(e) Steel Sheet Piling
(f) Steel Railroad Ties
(g) Square Root Angles
(h) Hand Rail Tees
(i) Checkered Floor Plates

These sections are not often used in steel construction for buildings, but occasionally conditions have to be met to which some of them are specially suited.

PROPERTIES OF SECTIONS

Under the heading "Properties of Sections" the handbooks give tables of the numerical values of the various functions of the sections. Referring to these tables, certain items need no explanation, viz: dimensions; thickness of metal; area; weight per lineal foot. Other items are not self-evident and will be explained in detail.

Center of Gravity (*C.G.*). See "Strength of Materials" for definition. The I-beam, H-section, and Z, Fig. 39, being symmetrical

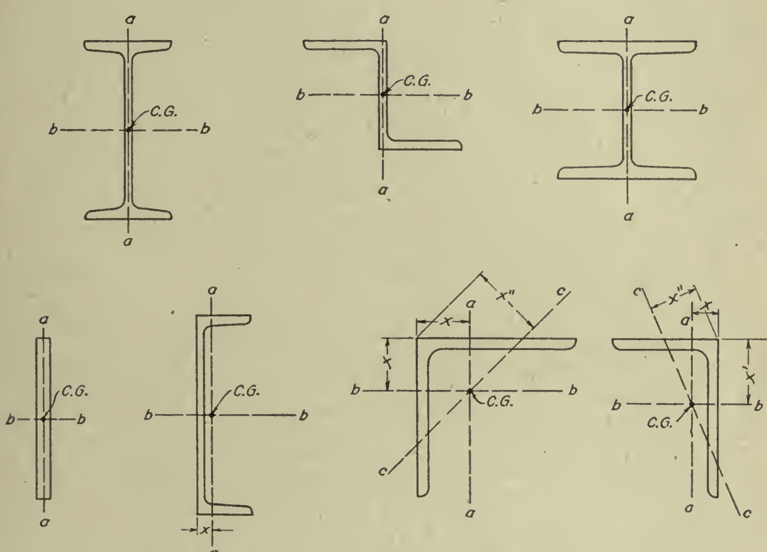

Fig. 39. Location of Center of Gravity of Sections. Values of x, x', and x'' to be taken from Tables in Handbook

about both axes, the center of gravity is in the center of the web and no values are given in the handbook tables. The ⊏-section, Fig. 39, is symmetrical only about the axis which is perpendicular to the web; the center of gravity must, therefore, lie on this axis. The table gives the distance of the center of gravity from the back of the channel.

Angles not being symmetrical about either axis, the center of gravity must be located by dimensions from the backs of both legs. If the legs are equal, both dimensions are the same; if the legs are unequal, the dimensions are unequal, the distance from the short leg x' being greater than that from the long leg **x**, Fig. 39.

The position of the center of gravity must be known in order to compute the moment of inertia of the section and the moments of inertia of built-up members. The former values are given in the tables; the latter must usually be computed by the designer.

Illustrative Example. Compute the position of the center of gravity of L4″ × 4″ × $\frac{1}{2}$″, disregarding fillets and rounded corners, Fig. 40. Divide the angle into two rectangles (1) and (2) as shown. Their centers of gravity are at c_1 and c_2.

Area of (1)	4 ″ × $\frac{1}{2}$″ =	2.00 sq. in.
Area of (2)	$3\frac{1}{2}$″ × $\frac{1}{2}$″ =	1.75 sq. in.
Total area		3.75 sq. in.

$$\text{Moments about } a'\,a' \text{ for (1)} = 2.00 \times \tfrac{1}{4} = 0.50$$
$$\text{Moments about } a'\,a' \text{ for (2)} = 1.75 \times 2\tfrac{1}{4} = 3.94$$
$$\text{Total moment} \quad 4.44$$

$$\text{Distance } x = \frac{4.44}{3.75} = 1.18''$$

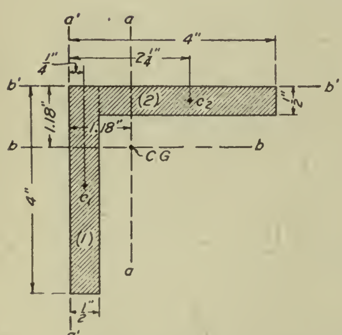

Fig. 40. Diagram Showing Computation of Position of Center of Gravity of Angle

Similar computations apply about the axis $b'b'$ and give the same result.

PROBLEM

Compute the position of the center of gravity of the following:

L 5″×3″×$\frac{3}{8}$″
15″ [33#

Moment of Inertia (I). Refer to "Strength of Materials" for definition and method of computing moment of inertia. Moment of inertia is designated by the letter I. When a subscript is added it indicates which axis is used. Thus I_a means the moment of inertia about the axis a. Note that this symbol is the same as is used for the beam. Care must be taken to avoid confusion. The meaning can be determined in each case by the context. The tables in the handbook give the value of I about both of the rectangular axes of the section and, in the case of angles, about a diagonal axis also. The position of this diagonal axis is so chosen as to give the minimum value of I. For I-beams and channels the minimum value is about the axis parallel to the web.

The moment of inertia enters into the formulas for bending and for deflection. It is also used in computing the radius of gyration of columns. Its values are given in the handbooks for the structural shapes and for plates, but it must be computed for most built-up sections, especially for plate girders. *The factors entering into the computation of the moments of inertia are always in inches.*

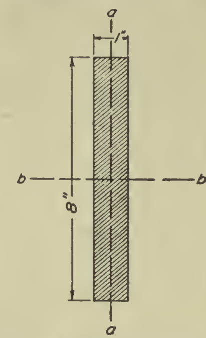

Fig. 41. Diagram for Moment of Inertia of Rectangular Plate

Illustrative Examples. 1. Compute I_a and I_b for the plate shown in Fig. 41.

$$I_a = \frac{1}{12} \times 8 \times 1 \times 1 \times 1 = \tfrac{2}{3}$$

$$I_b = \frac{1}{12} \times 1 \times 8 \times 8 \times 8 = 42\tfrac{2}{3}$$

2. Compute I_a for the plate girder section in Fig. 42 made of 1 Pl. 42″ × ½″ and 4Ls 6″ × 6″ × ½″.

for 1 Pl. 42″ × ½″ $\quad I_a$ (from tables) $= 3087$

for 4 Ls 6″ × 6″ × ½″ $\quad I_c$ (from tables) $4 \times 19.91 = 80$

for 4 Ls 6″ × 6″ × ½″ $\quad I_a \; 4 \times 5.75 \times 19.57 \times 19.57 = 8809$

$\qquad\qquad 11{,}976$

Deductions for rivet holes at m

Area of 2 holes $= 2 \times 1\tfrac{1}{2}'' \times \tfrac{7}{8}'' =$ 2.625 sq. in.

For 1 hole $I_d = \tfrac{1}{12} \times 1\tfrac{1}{2}'' \times \tfrac{7}{8}'' \times \tfrac{7}{8}'' \times \tfrac{7}{8}'' = .08$

(a value so small that it is neglected)

$I_a = 2.625 \times 18.75 \times 18.75 = \qquad 923$

Total net value $I_a \qquad = \qquad 11{,}053$

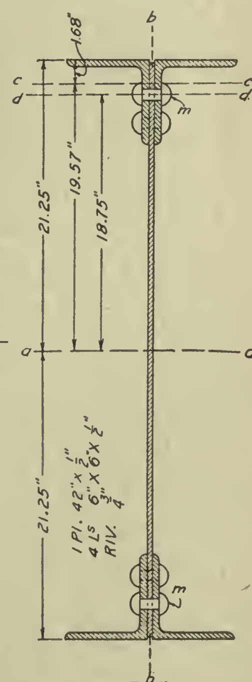

Fig. 42. Diagram for Moment of Inertia of Plate Girder

PROBLEMS

1. Compute the values of I for the section in Fig. 43. Deduct rivet holes. The section is made up of 4 Ls $6'' \times 4'' \times \frac{9}{16}''$ connected with lacing bars (lacing not figured).

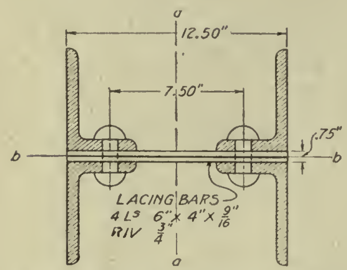

Fig. 43. Diagram for Moment of Inertia of Four Angles

2. Compute the values of I for the section shown in Fig. 44.

1 [$12'' \times 20\frac{1}{2}$#
1 L $4'' \times 3'' \times \frac{3}{8}''$

The axes $a\,a$ and $b\,b$ are through the center of gravity. The section not being symmetrical, the position of the center of gravity must be computed.

Radius of Gyration (r). The radius of gyration is a value derived from the moment of inertia, but as its definition involves higher mathematical relations it need not be given here. It is represented by r, and is expressed in inches.

The radius of gyration is derived from the moment of inertia by dividing by the area A in square inches and taking the square root of the result. This is expressed by the formulas

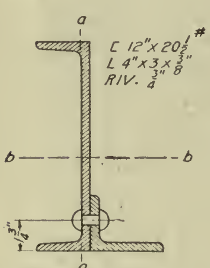

Fig. 44. Diagram for Moment of Inertia of Channel and Angle

$$I = Ar^2 \quad r^2 = \frac{I}{A} \quad r = \sqrt{\frac{I}{A}}$$

Illustrative Examples. 1. Referring to Fig. 41, the value of $I_b = 42\frac{2}{3}$; and $A = 8 \times 1 = 8$ sq. in. Therefore $r^2 = 42\frac{2}{3} \div 8 = 5\frac{1}{3}$, or $r = \sqrt{5\frac{1}{3}} = 2.31''$.

2. Referring to Fig. 42, the value of $I_a = 11{,}976$ (disregarding rivet holes). To find the radius of gyration

$$A = \begin{cases} 1 \text{ Pl. } 42'' \times \frac{1}{2}'' & = 21 \text{ sq. in.} \\ 4 \text{ Ls } 6'' \times 6'' \times \frac{1}{2}'' & = 23 \text{ sq. in.} \end{cases} = 44 \text{ sq. in.}$$

$$\therefore \quad r^2 = \frac{11{,}976}{44} = 272.2$$

$$r = \sqrt{272.2} = {}^*16.5''$$

*Refer to the textbook on Arithmetic for method of extracting the square root. Tables are given in the handbooks from which the values can be taken.

Problems

1. Compute the values of r for the sections given in Figs. 43 and 44.
2. Check the values given in the handbook for r for a 12″ I $31\frac{1}{2}$#.

The radius of gyration is used in the column formula as explained later in the text.

Section Modulus $\left(\frac{I}{c}\right)$. In the formula for the resisting moment of sections subjected to bending occurs the expression $\frac{I}{c}$, in which I is the moment of inertia and c is the distance from the neutral axis to the extreme fiber of the section. $\frac{I}{c}$ has a definite value for each section, and is called the section modulus. It saves one operation in arithmetic to have these values given for the various sections and they are given in the handbooks. As indicated by the fraction $\frac{I}{c}$, the value of the section modulus is determined by dividing the moment of inertia by the value of c.

Illustrative Examples. 1. Compute $\frac{I}{c}$ for an 8″ I 18# about the axis perpendicular to the web.

From the table, $I = 56.9$. The distance c is half the depth $= 4''$

$$\therefore \qquad \frac{I}{c} = \frac{56.9}{4} = 14.2$$

2. Compute $\frac{I}{c}$ for a channel 12″×20.5# about the axis parallel to the web. Not being a symmetrical section it has two values. From handbook, $I = 3.91$; $c = (2.94 - .70) = 2.24$, and $c = 0.70$.

$$\therefore \quad \frac{I}{c} = \frac{3.91}{2.24} = 1.75 \text{ and } \frac{I}{c} = \frac{3.91}{0.70} = 5.59$$

Problem

Compute the values of $\frac{I}{c}$ for

15″ I 42#	about axis parallel to web
9″ I 21#	about axis perpendicular to web
15″ I 33#	about axis perpendicular to web
L 3″×3″×$\frac{3}{8}$″	about axis at 45° to legs
L 6″×4″×$\frac{1}{2}$″	about axis parallel to short leg

Miscellaneous Properties. The handbooks include in the tables values of other properties of sections such as Coefficient of Strength, Coefficient of Deflection, and Resisting Moment.

Strictly speaking, these are not properties of the sections, as they depend upon the value of the unit stress. They will be discussed in the text relating to beams.

GENERAL INFORMATION

Price Basis. The designer needs to be posted on the basis of prices for structural steel. For a number of years Pittsburgh, which has been the recognized center of steel production, has been the basing point for steel prices. Given a certain price for steel at Pittsburgh, the price at any other point is determined by adding to the base price the freight from Pittsburgh. Thus, if the price of steel at Pittsburgh is \$1.50 per hundred pounds, the price in Chicago is \$1.68 per hundred pounds, the freight rate being (at the time of writing) 18 cents per hundred pounds.

Certain sizes of material are called "base" sizes. They are usually sold at a uniform price. The base sizes are: I-beams, 3 inches to 15 inches inclusive; angles, 3 inches to 6 inches inclusive; channels, 3 inches to 15 inches inclusive; tees, 3 inches and over; zees, all sizes. I-beams over 15 inches, angles over 6 inches, and angles and tees under 3 inches are charged for at a higher rate, usually 10 cents per hundred pounds, above base price. Special sections and sections rolled exclusively by one manufacturer are sold at prices varying from the base price according to market conditions. The base price itself varies from time to time, usually from \$1.25 per hundred pounds to \$1.50 per hundred pounds; occasionally it goes beyond these limits.

Mill and Stock Orders. Structural steel orders are handled on two bases: (a) based on securing the plain material for the job from the rolling mills; (b) based on securing it from stock. Of course there may be a combination of the two.

The mill basis is cheaper, as it eliminates waste, saves expense of handling, saves interest cost on the value of material, and may save a profit or premium demanded by the dealer for quick service. Consequently all work is carried out on the mill basis, if the time allowed for completion permits it to be done.

When the material is to be furnished on the mill basis, the engineer who makes the detail drawings or the engineering department of the fabricating company makes a list of the individual pieces required. These pieces are then ordered from the rolling mills, cut to the lengths required (a small variation in length is usually allowed; short pieces are usually ordered in multiple lengths). Thus there is practically no waste of material.

Material carried in stock is ordered from the rolling mills in lengths as long as can be handled conveniently. The lighter sections are ordered in lengths of 30 feet and 36 feet, and the heavier sections in lengths of 60 feet. In cutting this stock material there is necessarily considerable waste. This stock material is not usually available direct from the rolling mills. The dealers in stock are usually fabricating companies, jobbers, or brokers. They charge an advance in price over the mill price to cover waste, handling, cutting, and other expenses incidental to the business, and to cover such profit as the market condition may permit. This advance in price varies from 10 cents to 50 cents per hundred pounds.

Stocks of plain material are carried in all the larger cities. Printed lists of the material on hand are issued at frequent intervals. These lists should be consulted and used as a guide in selecting the sections that are to be used in all cases where stock is required.

Whether mill or stock material will be used depends upon the size of the job and the time service required. Small jobs, say less than 100 tons, will usually be taken from stock unless only one or two sections are required. If delivery of fabricated material is required within 60 days, it will usually have to be taken from stock. Even for much more extended deliveries, all or part of the material must be taken from stock,* if there is a great demand.

Variation in Weight. Attention is called to the provision in the specifications, p. 360, which permits a slight variation in the weight of the finished steel as compared with its theoretical weight. This variation in the case of sections other than plates is 2.5 per cent above or below the theoretical weight. This represents the practicable limits in adjusting the rolls of the mill. The variation applies to individual pieces and not to a bill of steel as a whole; some pieces will be overweight and some underweight, so that the average on a bill of considerable size should agree very closely with the theoretical weight. In the case of plates,

*Apply to the nearest dealer for a copy of his stock list. Use it in solving the problems in this book.

a much larger variation is allowed, amounting in some cases to as much as 18 per cent. It will be noticed that this variation is greater when plates are ordered to be of a certain gage or thickness than it is when they are ordered to be of a certain weight. The reason for this is that plates are slightly thicker in the middle than they are along the edges and, therefore, as the thickness must necessarily be measured near the edge, there is an excess of metal near the middle of the plate which is not counted. This excess is due to the springing of the rolls. Plates can be ordered by weight, that is, to have a certain weight per square foot of surface, and when so ordered the allowable variation is less because the rolls can be adjusted to give the average weight. The result is that the fabricating shop usually orders large plates by weight per square foot. In a job involving a large amount of plate work, as for chimneys, tanks, etc., this may become a matter of importance, but for building work a relatively small number of plates are required and it is not customary to specify them by weight, but by thickness.

QUALITY OF MATERIAL

Reliability of Structural Steel. Structural steel is the most reliable material used in building construction. Its manufacture has been a continuous development to the extent that the quality of material produced is under almost absolute control. The ingredients are tested and measured before being put into the furnace, and the product is analyzed and tested physically to make sure that it fulfills the required standards; so that, with a reasonable amount of inspection and test, the purchaser can have definite assurance that he is securing the quality of material which he needs.

The manufacturers and users of structural steel have co-operated in developing the material in order to attain the most practicable results. On the one hand, the manufacturers have insisted on keeping the quality such as to make its manufacture commercially satisfactory. On the other hand, the users of steel have demanded the best material that it is possible to make and still keep within reasonable limitation of cost of manufacture.

STANDARD SPECIFICATIONS

As a result of the efforts of the manufacturers and users, Standard Specifications have been formulated covering the quality of structural steel. There are three sets of specifications that may safely be used, viz:*

(a) Manufacturers' Standard Specifications for Structural Steel—Class B †

(b) Standard Specifications for Structural Steel for Buildings, adopted by the American Society for Testing Materials (Given in full p. 359)

(c) Specifications for Structural Steel, adopted by the American Railway Engineering Association

Comparison of Specifications. A brief comparison of the provisions of these three sets of specifications is of interest.

Range of Application. The specifications (a) and (b) are intended primarily to apply to steel for building work, whereas (c) is for railway bridges. In buildings, the greater part of the load to be supported is permanent or dead load. The variable or live load usually is applied gradually, without shock or vibration. In railway bridges the conditions are quite different. The permanent load for a short span is the smaller part of its capacity. The live load, being much larger than the dead load and being applied quickly, produces great shocks and vibration. Because of these conditions, specification (c) is more rigorous in its requirements than are (a) and (b).

Process of Manufacture. Specification (c) requires the open-hearth process of manufacture; (a) and (b) permit either open-hearth or Bessemer.

Chemical Analysis. Specification (c) requires the chemical analysis to report the percentages of sulphur, phosphorus, carbon, and manganese, and limits the amount of sulphur; (a) and (b) limit the phosphorus.

Tensile Strength. Specification (c) places the desired ultimate tensile strength of steel sections at 60,000 pounds per square inch, allowing a variation of 4000 pounds, thus making the range of strength 56,000 to 64,000 pounds; (b) allows a range from 55,000 to 65,000 pounds; (a) allows the same range as (b) and in addition

* (a) Published in the handbooks issued by the Steel Manufacturers; (b) Published by American Society for Testing Materials, Edgar Marburg, Secretary, University of Pennsylvania, Philadelphia, Pa.; published in full in Carnegie Steel Company's Pocket Companion, 1913 edition; (c) Published by American Railway Engineering Association, 910 South Michigan Boulevard, Chicago, Ill.

† Class A is for railroad bridges.

permits a maximum of 70,000 pounds if the percentage of elongation is the same as for steel having a tensile strength of 65,000 pounds.

Rivet Steel Strength. Specification (c) specifies the desired strength of rivet steel at 50,000 pounds, allowing 4000 pounds variation, thus making the range of strength from 46,000 to 54,000 pounds; (a) allows a range from 46,000 to 56,000 pounds; and (b) allows a range from 48,000 to 58,000 pounds.

Elongation and Fracture. The three specifications are in close agreement as to their requirements for elongation of the test specimen and the character of fracture.

Bending Requirements. Specification (c) is somewhat more rigorous than the others in the bending requirements.

Either of these specifications will give satisfactory results, but specification (b) of the American Society for Testing Materials is recommended as being most suitable for building work. It is given in full on p. 359.

DISCUSSION OF IMPORTANT FEATURES

Method of Manufacture. A brief description has been given of the two methods of manufacture of steel. The Bessemer process is more rapid and, as a result, is less subject to accurate control than the open hearth. In the Bessemer process the operator is governed by the character and color of the flame issuing from the converter. He must learn by experience to do this, as the whole matter depends upon his judgment. The open-hearth process, being slower, gives an opportunity to take samples and make analyses, and thus control the operation.

The Bessemer process, as ordinarily conducted, does not remove sulphur and phosphorus, so that whatever quantities of these undesirable elements are in the iron ore remain in the finished steel. On the other hand, the usual open-hearth practice reduces the amount of sulphur and phosphorus, these elements being removed in the slag.

For the above reasons, the product of the open-hearth furnace is considered more desirable than that of the Bessemer, when steel is to be subjected to severe use, as in the case of railway bridges.

Heretofore the question has been an economic one. The Bessemer process being the cheaper, most of the producing capacity was of that type, and a higher price was charged for open-hearth steel. Recently the situation has changed. Most of the new furnaces are open-hearth and no extra charge is demanded for steel made by this process. There is now no difficulty in securing it.

Chemical Composition. *Carbon.* The essential elements of steel are iron and carbon. All of the other elements found may be considered as impurities. The iron, of course, constitutes all but a small percentage of the total. The function of the carbon is to make the steel hard and strong. Within certain limits the tensile strength of steel increases, while the ductility decreases, with the increase in the amount of carbon used. The amount of carbon in structural steel varies from 0.10 per cent to 0.40 per cent. The smaller amount occurs in rivet steel. For structural shapes, the usual limits are 0.15 per cent to 0.25 per cent. A larger amount makes steel too hard for structural purposes.

Steel to be forged or welded needs to be low in carbon. Steel to be tempered must be high in carbon. These features do not concern structural steel.

Phosphorus. Phosphorus occurs as an impurity in the iron ore. It is not practicable or necessary to remove all of it. It increases the strength of the steel but produces brittleness. The amount of phosphorus allowed is about 0.10 per cent.

Sulphur. Sulphur is also found as an impurity in the iron ore. Its presence in the steel causes trouble in rolling. It usually amounts to less than 0.05 per cent.

Silicon. Silicon may be in the pig iron or may be absorbed from the material used in lining the steel furnace. It increases the hardness of the steel and has a beneficial effect in the process of manufacture, so that the presence of a limited quantity, about 0.20 per cent, is not objectionable.

Manganese. Manganese also may be found in the iron ore, but if not, it is added during the process of manufacture to assist in the chemical transformations. Its presence in the finished steel to the extent of about 1.0 per cent is an advantage, as it adds to the strength and improves the forging qualities. However, some authorities believe that it promotes corrosion of steel and on this account is objectionable.

Alloys of Steel. A much larger quantity of manganese is sometimes used as an alloy, but such a steel is not used for structural purposes. There are many alloys of steel, developed for special purposes. The only one used for structural work is nickel steel, and up to the present time its use has been limited to a few large bridges. Probably nickel steel will not be economical for building work for some time.

Physical Properties. The determination of the physical properties most suitable for structural steel has been a gradual development. It has been influenced by the cost of manufacture and ease of fabrication on the one hand, and uniformity and economy to the consumer on the other.

The manufacturers have required that such limits be set as would permit them to operate economically. Expensive refinements of small importance have been eliminated. The allowable range in strength has been made large enough so that it can easily be attained. The fabricating shops have encouraged the use of a material that can easily be punched and sheared.

The designing engineers representing the consumers have demanded a small range in strength and uniformity in physical properties, and at the same time as great strength as is consistent with reliability of material, with economy of manufacture, and with ease of fabrication.

As the physical properties are closely related to the uses of the steel, their requirements are much more explicit than are those relating to chemical composition. The chemical tests are of interest only to the extent that they indicate physical properties. Thus, high carbon and high phosphorus indicate high tensile strength and brittleness, but these properties can be determined more directly by the tension test, with the attendant observations of elongation and character of fracture.

Railway Bridge Grade Steel. It has been noted that the Manufacturers' Standard Specifications (a) provide for steel, which has a maximum strength five thousand pounds greater than the strength provided by specifications (b) and (c). This grade of steel was formerly very much used for building work, but now steel having the lower strength is generally used. The reason for using the lower strength steel is that it is more reliable and more uniform in quality. The higher the strength the more brittle the material, hence the greater danger of injury from careless handling and from the shop operations of fabricating. This latter condition makes the fabricating shops prefer to use the softer grade. It seems probable that this harder grade of steel will be used less and less and, therefore, more difficult to get; so it is wise to specify the railway bridge grade, which is Class A, in case Manufacturers' Standard Specifications are used.

Yield Point. The yield point indicates one of the most important properties of structural steel. When a piece of steel is subjected to a tensile stress, it elongates, the amount of the elongation within certain limits being proportional to the load applied; thus, if a piece of steel of one square inch cross section is subjected to a load of 5000 pounds, and then to a load of 10,000 pounds, the elongation in the second case will be twice as much as that in the first case. The test for the strength of the steel specimen, as described in the specifications, is made in a tension or pulling machine, to which is attached a lever arm carrying a weight, corresponding to the beam of an ordinary scale. If the load is increased at a uniform rate, the weight on the scale beam, by being moved at a certain uniform rate, will keep the beam exactly balanced until about one-half the ultimate strength of the material is reached; then the scale beam will drop, indicating that the specimen is elongating more rapidly. The stress at which this occurs is called the "yield point" of the steel.

Breaking Load. If a load somewhat larger than that which produced the above effect were applied continuously for a long time, or repeated many times, the specimen would finally break; but usually in testing, additional load is applied at the same rate as before until the specimen breaks. The breaking load, according to the specifications, should be about 60,000 pounds per square inch. This represents the load which will break the steel if applied within a relatively brief period of time, but a much smaller load will break it if applied over a long period of time.

Elastic Limit. The change in the rate of elongation does not occur just at the point where it becomes manifest by the action of the scale beam, but at a somewhat lower stress. The point where the change actually occurs is called the "elastic limit". This term formerly was used in specifications and, in fact, still is used in the Manufacturers' Standard Specifications, but as the commercial methods of testing structural steel do not clearly show the exact point of the elastic limit, the yield point is used.

Yield Point and Factor of Safety. The Standard Specifications require that the yield point shall be not less than one-half the ultimate strength. The value of the yield point is usually several thousand pounds above this amount. When the yield point is reached, the material has begun to fail. This value, therefore, instead of that for the ultimate strength, is the one which should be used in computing the *factor of safety.* If the yield point is at 32,000 pounds and the unit stress 16,000 pounds, the factor of safety is 2 instead of 4, as commonly stated. Refer to the discussion of factor of safety, p. 7.

Reduction of Area. The provision in the specifications regarding the reduction of area of the test piece at the point of fracture is of importance, as it indicates the ductility of the metal. If the piece breaks without much reduction in area, it indicates that the material is hard and probably brittle. Such material is likely to break, if subjected to shock, and may fracture in punching and shearing. The character of the fracture is indicative of the same condition. If cup-shaped and silky in appearance, it indicates toughness; but if the fracture is irregular, it indicates brittleness. The bending test also is important for determining whether the steel is tough or brittle.

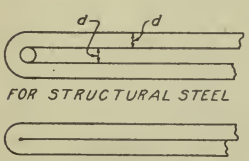

Fig. 45. Bending Tests for Steel

Inspection and Tests. In order to check the quality of the steel as it is made, tests are made of each melt. The chemical analysis is made from a sample taken from the molten metal as it comes from the furnace or converter. Sometimes a check analysis is made from drillings taken from the rolled sections.

Physical tests are made in accordance with the requirements of the standard specifications. The test specimens are cut from the finished structural steel. The bend test is made by bending the specimen around a pin whose diameter equals the thickness of the specimen, Fig. 45. Rivet rods must bend flat on themselves. These tests are made with cold steel. The work is done either by blows or by pressure. To pass the test, the specimens must show no fracture on the outside of the bent portion.

The tension-test specimen is shaped as shown in Fig. 46. It is put in a tension-testing machine and pulled until it breaks. From this are determined the total strength, yield point, elongation, and character of fracture, Fig. 47.

Records of these tests are furnished to customers if desired.

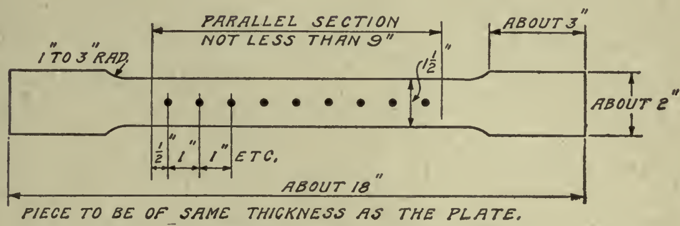

Fig. 46. Tension Test Piece

Customers may, and on important jobs do, employ inspectors to supervise the tests. These inspectors also make a surface inspection to see that the finished sections are straight and free from cracks, blisters, buckles, and slivers. Fig. 48 is a specimen report of tests.

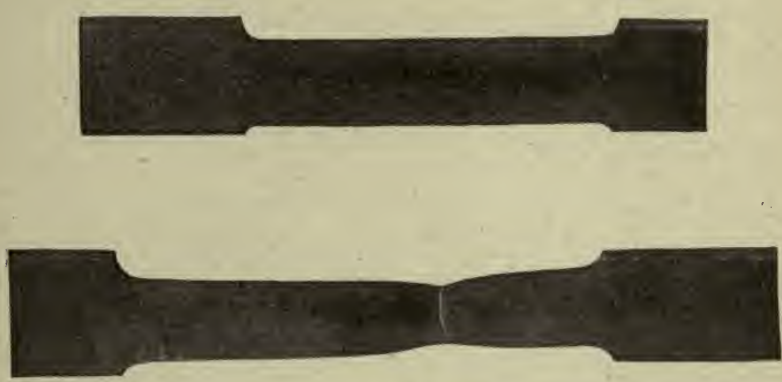

Fig. 47. Test Piece Before and After Being Broken by Tension

UNIT STRESSES

General Discussion. The unit stress, or working stress, is the stress or load that is allowed on each square inch of cross section of the metal and is expressed in pounds per square inch. There is

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SHEET 13 FILE NO. C5549 I. O. NO. S10014

DATE

Report of Tests of Material Manufactured by Illinois Steel Company

For Contract C-7762 of Gary Plant of American Bridge Company

Structure: Fort Dearborn Hotel, Van Buren & La Salle Streets, Chicago. Req. No. 73-1992.

TEST No.	MELT NUMBER	TEST PIECE REPRESENTS	Elastic Limit Per Sq. Inch.	Tensile Strength Per Sq. Inch	Elongation Per Cent In 8 Inches	Reduction of Area Per Cent	FRACTURE	Bend and Drift Tests	Accepted or Rejected	Process of Manufacture	CHEMICAL ANALYSIS Car.	Phos.	Mang.	Sul.	Sil.
		Ult. 55-65,000, Elas. 1/2Ult., Elong. 1,400,000/Ult., Bend 180° 1 Dia. Phos. .06													
		Angles													
204	15082	6×6×3/4	35910	60520	28.7	58.7	Cup	O.K.	Acc.	B.O.H.	.20	.014	.50	.031	
205	37039	5×3×8.2	35030	61080	28.7	53.9	1/2Cup	"	"	"	.17	.022	.47	.039	
		Plates													
206	4096	15×1/4	36670	60230	26.7	52.9	Ang	"	"	"	.19	.014	.53	.044	
207	8075	7×3/4	35770	60800	27.2	51.0	Ang	"	"	"	.18	.017	.45	.040	
208	9083	14×5/16	37560	61060	27.2	51.7	Ang	"	"	"	.16	.009	.36	.040	
209	10076	12×3/4	35430	60470	28.7	50.3	Ang	"	"	"	.18	.012	.45	.043	
210	10077	"	35800	61800	29.5	53.8	And	"	"	"	.17	.018	.44	.045	
211	15103	24×5/16	37220	61800	29.0	55.1	Ang	"	"	"	.17	.033	.47	.039	
212	19104	14×7/8	36460	62200	26.7	52.7	Ang	"	"	"	.18	.051	.50	.034	
213	24104	12×3/8	37200	64240	29.0	50.1	Ang	"	"	"	.17	.018	.51	.046	

Robert W. Hunt & Co

Fig. 48. Sample Report of Inspection Tests

practical agreement on the values used for the various kinds of stress. The following values can be used with assurance that they will give safe results. Note that these values are for building work; they may also be used for highway bridges but not for railroad bridges.

Structural Steel. Structural steel is so dependable and of such uniform quality that the values for unit stress are well established. The values given follow standard practice.

Maximum Allowable Stresses on Structural Steel in Pounds per Square Inch:

Axial tension net section	16,000
Bending on extreme fiber, tension	16,000
Bending on extreme fiber, compression	16,000
Bending on extreme fiber, of pins	25,000
Shear on shop-driven rivets	12,000
Shear on field-driven rivets and turned bolts	10,000
Shear on rolled-steel shapes	12,000
Shear on plate-girder webs	10,000
Bearing on shop-driven rivets and pins	25,000
Bearing on field-driven rivets and turned bolts	20,000
Axial compression on columns	$16{,}000 - 70\frac{l}{r}$

In the above, l is the length of the column in inches from center to center of bearing, and r is the least radius of gyration. The maximum value allowed is 14,000 pounds per square inch.

For wind pressure alone or combined with gravity loads, the unit stresses may be 50 per cent in excess of those given above, but the section must not be less than required for the gravity loads alone.

The discussion under "Columns" should be consulted regarding limitations of the use of the compression formula and the conditions under which higher and lower values are used.

Cast Iron. There is not such close agreement among engineers as to the unit stresses allowable on cast iron. The following values represent fairly well the current practice; they are in pounds per square inch.

Axial tension	not allowed
Bending on extreme fiber, tension	3,000
Bending on extreme fiber, compression	10,000
Shear	2,000
Compression	$10{,}000 - 60\frac{l}{r}$

The discussion of cast-iron columns should be consulted for limitations of values used and length of columns. These values are taken from the Building Ordinances of the City of Chicago.

Masonry. As the ultimate bearing of steel work is on masonry, and as the bearing values are necessary in designing the bearing plates and column bases, the values are given for the usual forms of masonry. The values below, expressed in pounds per square inch, are taken from the Building Ordinances of the City of Chicago.

Coursed rubble, Portland cement mortar	200
Ordinary rubble, Portland cement mortar	100
Coursed rubble, lime mortar	120
Ordinary rubble, lime mortar	60
First-class granite masonry, Portland cement mortar	600
First-class lime and sandstone masonry, Portland cement mortar	400
Portland cement concrete, 1-2-4 mixture, machine mixed	400
Portland cement concrete, 1-2-4 mixture, hand mixed	350
Portland cement concrete, 1-2½-5 mixture, machine mixed	350
Portland cement concrete, 1-2½-5 mixture, hand mixed	300
Portland cement concrete, 1-3-6 mixture, machine mixed	300
Portland cement concrete, 1-3-6 mixture, hand mixed	250
Natural cement concrete, 1-2-5 mixture	150
Paving brick, mortar 1 part Portland cement, 3 parts torpedo sand	350
Pressed brick and sewer brick, mortar same as above	250
Hard common select brick, same as above	200
Hard common select brick, mortar, 1 part Portland cement, 1 part lime, 3 parts sand	175
Common brick, all grades, Portland cement mortar	175
Common brick, all grades, good lime and cement mortar	125
Common brick, all grades, natural cement mortar	150
Common brick, all grades, good lime mortar	100

The American Railway Engineering Association permits a bearing of 800 pounds per square inch on concrete, provided the area of the pier is twice the area of the base plate. The writer would allow this high stress only when the concrete is properly reinforced with hooping, similar to that used in hooped columns.

RIVETS AND BOLTS

Ordinary Sizes. The sizes of rivets vary in a general way with the thickness of steel which they connect. In structural steel work the sizes commonly used are $\frac{5}{8}$ inch, $\frac{3}{4}$ inch, and $\frac{7}{8}$ inch, the $\frac{3}{4}$-inch size being used much more than any other. In very light work $\frac{1}{2}$-inch rivets are sometimes used and, in very heavy work, rivets 1 inch, $1\frac{1}{8}$ inches, and $1\frac{1}{4}$ inches are used.

Rivets smaller than $\frac{3}{4}$ inch are used when the size of the members connected requires it, or when the thickness of metal used is chiefly $\frac{1}{4}$ inch. $\frac{5}{8}$-inch rivets must be used in the flanges of 6-inch and 7-inch I-beams; 6-inch and 7-inch channels; and 2-inch angles. $\frac{3}{4}$-inch rivets can be used in all of the beams, channels, and angles larger than the above sizes. $\frac{7}{8}$-inch rivets may be used in beams 18 inches and larger, and angles 3 inches and larger.

Another consideration that sometimes affects the sizes of rivets used, and concerns particularly the sizes larger than $\frac{3}{4}$ inch, is the thickness of metal to be joined together. It is the general experience in shops that satisfactory punching cannot be done when the thickness of metal is greater than the diameter of the hole to be punched. Of course, it is possible to punch thicker material than this, but it is troublesome to do so because of the frequent breakage of punches. Consequently if most of the material to be punched is $\frac{7}{8}$ inch in thickness, $\frac{7}{8}$-inch rivets will be used.

Another approximate rule governing the size of rivets is that in general the diameter of the rivet shall be not less than one-fourth of the total thickness of metal.

The use of more than one size of rivet on a job is to be avoided as much as practicable on account of the trouble and expense of frequently changing the punches. It is especially inconvenient to punch more than one size of hole or drive more than one size of rivet in a structural member.

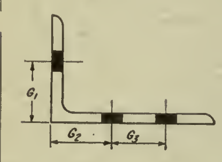
Spacing. There are a number of conditions that control the spacing of rivets. These have been developed into practical rules which are quite uniform among the various fabricating shops. Rivets spaced too close together would cut out too large a percentage of the cross section of members. Rivets spaced too far apart cause a waste of material in connecting pieces.

The specifications relating to rivet spacing,* p. 365, items 57 to 63, are in accord with usual practice and should be followed.

*From "Specifications for Structural Work of Buildings" by C. C. Schneider, M. Am. Soc. C. E., published in *Transactions of the American Society of Civil Engineers*, Vol. LIV (June, 1905), p. 498.

TABLE I

Gages for Angles



Leg	8"	7"	6"	5"	4"	$3\frac{1}{2}$"	3"
G_1	$4\frac{1}{2}$"	4"	$3\frac{1}{2}$"	3"	$2\frac{1}{2}$"	2"	$1\frac{3}{4}$"
G_2	3"	$2\frac{1}{2}$"	$2\frac{1}{2}$"	2"	...	...	...
G_3	3"	3"	$2\frac{1}{4}$"	$1\frac{3}{4}$"	...	...	...
Max. Rivet	$1\frac{1}{8}$"	1"	$\frac{7}{8}$"	$\frac{7}{8}$"	$\frac{7}{8}$"	$\frac{7}{8}$"	$\frac{7}{8}$"

Leg	$2\frac{1}{2}$"	2"	$1\frac{3}{4}$"	$1\frac{1}{2}$"	$1\frac{3}{8}$"	$1\frac{1}{4}$"	1"	$\frac{3}{4}$"
G_1	$1\frac{3}{8}$"	$1\frac{1}{8}$"	1"	$\frac{7}{8}$"	$\frac{7}{8}$"	$\frac{3}{4}$"	$\frac{5}{8}$"	$\frac{1}{2}$"
G_2	...	...	...	...	...	...	...	...
G_3	...	...	...	...	...	...	...	...
Max. Rivet	$\frac{3}{4}$"	$\frac{5}{8}$"	$\frac{1}{2}$"	$\frac{3}{8}$"	$\frac{3}{8}$"	$\frac{3}{8}$"	$\frac{1}{4}$"	$\frac{1}{4}$"

Gage. The term "gage" is used to designate the spacing of rivet lines parallel to the axis of the member. For example, Fig. 49 illustrates the gage lines of beams, channels, and angles. Standard values are assigned in the hand-books to the gage lines in the flanges of I-beams and channels, and in angles. However, as manufacturers do not agree as to the gage lines of angles, values used by the American Bridge Company are given, Table I.

Gage lines in webs of beams and channels and in plates are not standard and are located according to requirements.

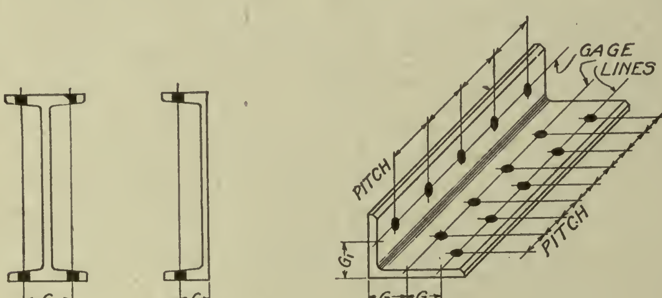


Fig. 49. Diagrams Showing Gage and Pitch Lines

Pitch. By the pitch of rivets is meant the spacing along the gage lines, Fig. 49. Some of the rules for this spacing are given in Schneider's Specifications previously referred to. Note carefully the provisions there given. The rule usually followed for the minimum pitch is three times the diameter of the rivet. But this minimum should be used only when necessary, it being preferable to use a larger spacing of rivets under ordinary conditions. Three inches is desirable for $\frac{3}{4}$-inch rivets, where this spacing does not involve the use of an excess of material in the connected pieces. Where no definite stress occurs in the rivet, as in built-up columns, or where the stress is small, as in certain portions of flanges of plate girders, six inches has been established as the maximum. In case there are two gage lines closer together than the minimum spacing allowed, the rivets in the adjacent rows must alternate so that the diagonal distance between them will exceed the minimum by 40 per cent or more.

Edge Distance. If holes are punched too close to the edge of the metal, the tendency is to bulge out the metal and perhaps to crack the edge. This necessitates maintaining a certain distance from the edge to the center of the rivet holes. This distance must be greater in the case of a sheared edge, as of a plate, than is required for a rolled edge, as the flange of a beam, an angle, or a universal mill plate. The values commonly used are given in Schneider's Specifications quoted above.

In the smaller sizes of beams and channels, the gage distances do not comply with these specifications. The width of flange is not sufficient to permit the use of the full edge distance and still allow necessary clearance from web to permit driving. On account of the danger that the metal will bulge out or crack along the edge, designers should try to avoid using smaller than 10-inch I-beams and channels in a way that will require flange punching. Instead, web connections or clips and clamps can generally be used.

Clearance. A hole cannot be punched close against the web of an I-beam or close to the leg of an angle. A certain amount of space is required for the die. Of course holes can be drilled in any position, but this is not resorted to unless there is some particular reason for so doing. However, the punching of holes is not the limiting feature in the matter of rivet clearance. The required clearance is governed by the size of the die used in forming the rivet head. The usual rule for clearance is *one-half the diameter of the rivet head plus three-eighths of an inch.* The clearances required for various conditions for several sizes of rivets are given in Fig. 50, which represents the practice of the American Bridge Company.

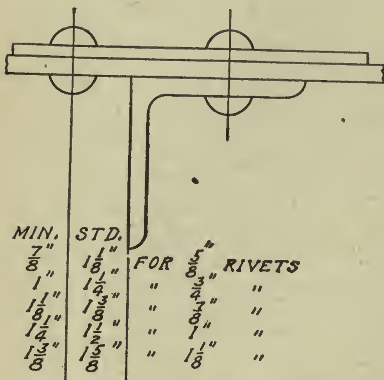


Fig. 50. Clearance Allowed for Riveting

Closely associated with the amount of clearance is the accessibility for driving the rivets, Fig. 51. For power driving, the rivet must be so situated that it can be brought between the jaws of the riveting machine. For riveting with the percussion hammer (air hammer), it must be possible to hold on to one head of the rivet with a die while the other head is formed by the riveter. For hand riveting it is necessary to be able to hold on to one head of the rivet and that the other end of it be accessible for driving with a maul. It is sometimes necessary to cut away flanges of I-beams or cut holes in the webs of box girders to make the rivets accessible for driving, Fig. 51. This matter is generally looked after in making shop drawings, but needs some attention in designing.

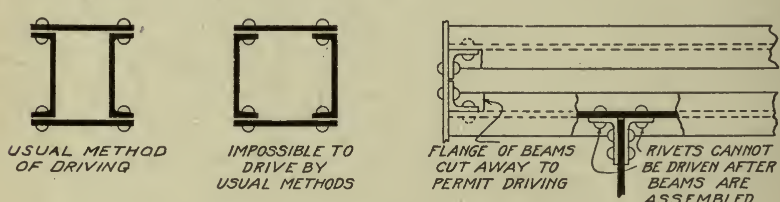


Fig. 51. Difficult Situations for Riveting

Rivet Heads. *Manufacture.* Rivets are made with one head. This is done by heating a length of rivet rod to the proper temperature and running it into the rivet machine. The machine upsets the end of the rod, making a head, and then cuts off the rivet to the desired length. It is necessary that the dies in which the heads are formed be of proper size and be kept in perfect condition in order to make good rivets. If the two halves of the die which grip the sides of the rivet do not fit closely, some of the metal will be forced between them, forming fins on the sides of the rivets, Fig. 52. If the corners of the die become rounded, a shoulder will be formed at the junction of the shank with the head. Either of these defects will prevent the rivet head from fitting up tight against the plate, thus causing unsatisfactory results when driven. This point is especially important in tank work where the rivets must be water-tight.

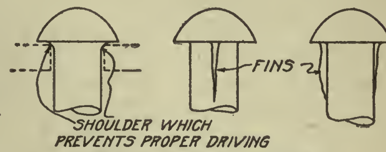
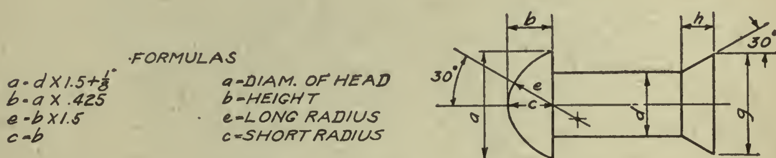


Fig. 52. Defective Rivets

Button Head. The shapes of the rivet heads vary among different makers, although these variations are slight. The type of head used in structural work is called the "button head" to distinguish it from the cone head which is used in tank and boiler work.

Flattened and Countersunk Heads. It is sometimes necessary to flatten rivet heads for special situations in order to provide the required clearance for an adjacent member. This flattening may vary from a slight reduction from the full thickness of the head down to a flush or countersunk head. The different thicknesses ordinarily used are $\frac{3}{8}$ inch, $\frac{1}{4}$ inch and $\frac{1}{8}$ inch. A countersunk rivet is one in which the head is made in the form of a truncated cone and is formed by driving in a hole which has been tapered by reaming



Diam. of Rivets	Diam. of Holes	FULL DRIVEN HEAD				COUNTERSUNK	
		Diam.	Height	Radius	Radius	Diam.	Depth
d		a	b	c	e	g	h
$\frac{3}{8}$	$\frac{7}{16}$	$\frac{11}{16}$	$\frac{19}{64}$	$\frac{19}{64}$	$\frac{7}{16}$	$\frac{19}{32}$	$\frac{3}{16}$
$\frac{1}{2}$	$\frac{9}{16}$	$\frac{7}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{9}{16}$	$\frac{25}{32}$	$\frac{1}{4}$
$\frac{5}{8}$	$\frac{11}{16}$	$1\frac{1}{16}$	$\frac{29}{64}$	$\frac{29}{64}$	$\frac{43}{64}$	1	$\frac{5}{16}$
$\frac{3}{4}$	$\frac{13}{16}$	$1\frac{1}{4}$	$\frac{17}{32}$	$\frac{17}{32}$	$\frac{51}{64}$	$1\frac{3}{16}$	$\frac{3}{8}$
$\frac{7}{8}$	$\frac{15}{16}$	$1\frac{7}{16}$	$\frac{39}{64}$	$\frac{39}{64}$	$\frac{59}{64}$	$1\frac{3}{8}$	$\frac{7}{16}$
1	$1\frac{1}{16}$	$1\frac{5}{8}$	$\frac{11}{16}$	$\frac{11}{16}$	$1\frac{1}{32}$	$1\frac{9}{16}$	$\frac{1}{2}$
$1\frac{1}{8}$	$1\frac{3}{16}$	$1\frac{13}{16}$	$\frac{49}{64}$	$\frac{49}{64}$	$1\frac{5}{32}$	$1\frac{3}{4}$	$\frac{9}{16}$

Fig. 53. Proportions of Rivets in Inches
From American Bridge Company

so that the diameter at the outside is greater than at the inside of the plate. The sizes of rivet heads are shown in Fig. 53. The conventional signs for riveting are given in the handbooks.

It is to be noted that countersunk rivets are not as strong as rivets with button heads and are much more expensive, consequently they are not used unless absolutely required by the condi-

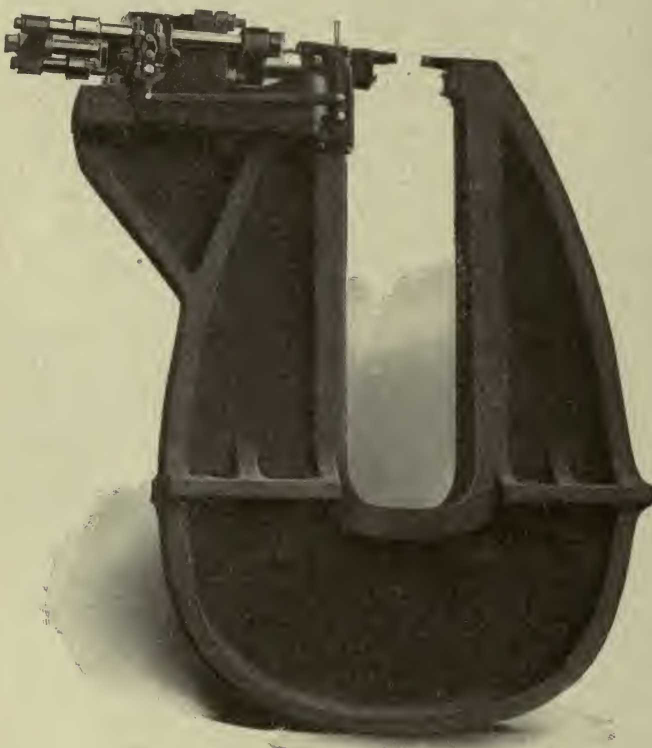


Fig. 54. 100-Ton Hydraulic Riveter, 120-inch Gap
Courtesy Mackintosh, Hemphill & Company

tions. A flattened rivet should be used in preference to a countersunk rivet; but when a smooth surface is to be obtained, the head must be countersunk and chipped flush with the plate.

Driving. Before rivets can be driven, the pieces to be joined must be assembled accurately in position and be held together with bolts. The number of bolts used for this purpose will depend to some extent on the accuracy of the punching and the straightness of the pieces. If the several pieces are not held together, the metal of the rivet will be forced out between them, or the driving of adjacent

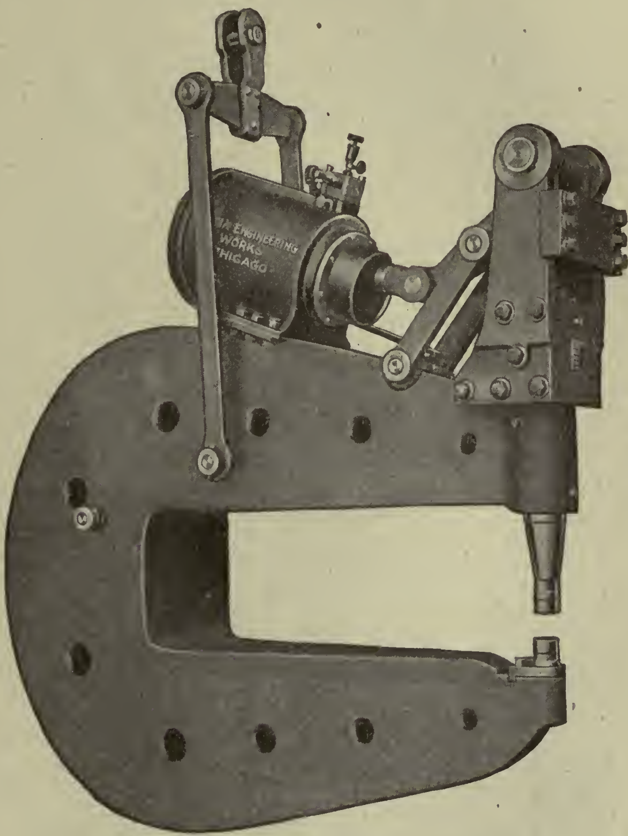


Fig. 55. Hanna Pneumatic Riveter, 24-inch Gap
Courtesy Vulcan Engineering Sales Company

rivets may draw the plates closer together and loosen the rivets previously driven.

Rivet holes are punched $\frac{1}{16}$ inch larger than the nominal size of the rivet for when the rivet is heated, it expands somewhat, making it necessary to have the larger size hole. The driving of the rivet must be done in such a way as to upset the metal of the shank so that it fills the rivet hole solidly, even to the extent of filling out any irregularities in the hole, and then the button head must be formed on the driving side. As the rivet cools, it shrinks and thus grips the steel more tightly than when first driven.

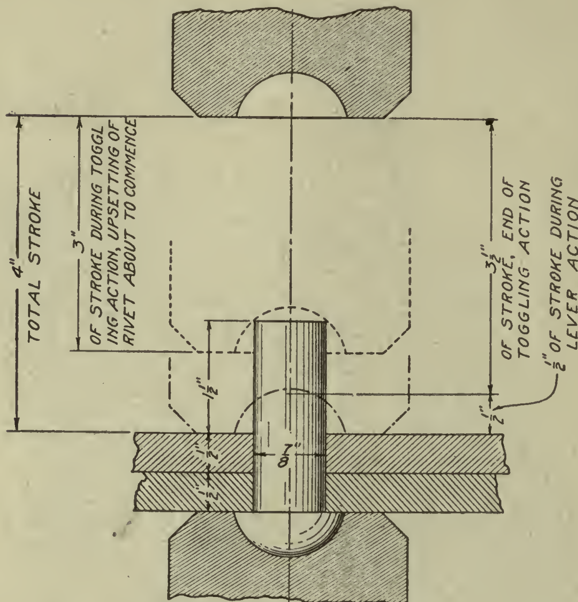


Fig. 56. Rivet Ready for Driving
Courtesy Vulcan Engineering Sales Company

Riveting Machines in Shop. In the shop, rivets are driven with an hydraulic riveter, Fig. 54, or a pneumatic riveter, Fig. 55. The

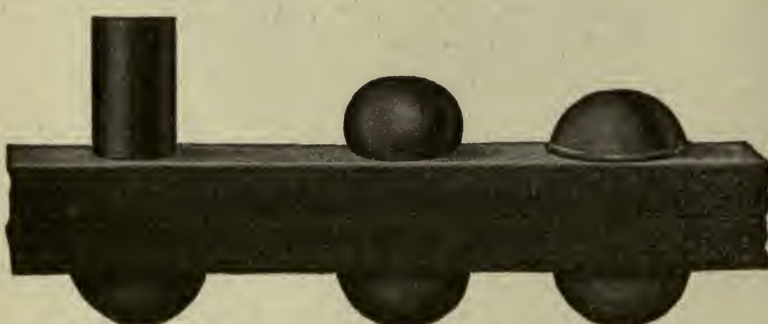


Fig. 57. Three Stages in Process of Riveting

machine consists essentially of a yoke which spans the members to be riveted, Fig. 56. On the outer arm of the yoke is a die which fits over the head of the rivet; the other arm carries a similar die, or rivet set, which pushes against the end of the rivet, upsetting the shank of the rivet and thus forming a head, Fig. 57. The power is applied by means of hydraulic or pneumatic pressure. The pressure

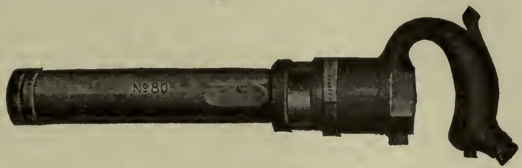


Fig. 58. Pneumatic Riveting Hammer
Courtesy Chicago Pneumatic Tool Company

is held on until the rivet is partly cooled and has acquired enough strength so that the spring of the plates will not stretch it.

Pneumatic Hammer. Whenever the rivet is in such position that it cannot be reached by means of the power riveter, it is driven

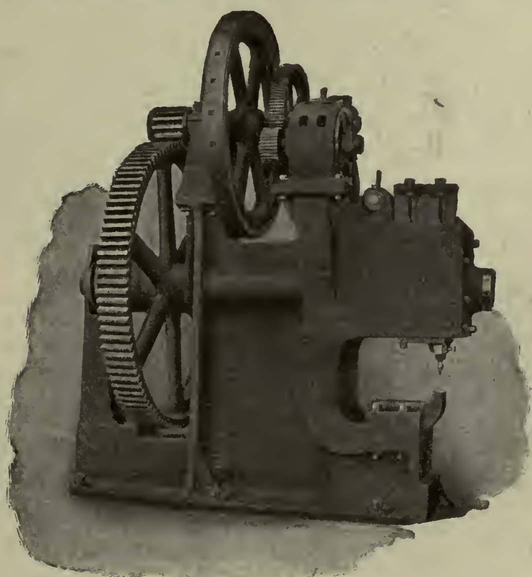


Fig. 59. Light Motor-Driven Punch
Courtesy Mackintosh, Hemphill & Company

with a pneumatic hammer. The rivet is inserted in the hole and held in place by means of a die pressed against the head, the die being held in position by hand or by a suitable arrangement of levers. The pneumatic riveter, or air gun, Fig. 58, carries a die, or set, for upsetting the rivet and forming the head. When the power is turned on, this machine delivers very rapid blows and thus performs the required work. Riveting in the field on the assembled structure is usually done by means of the pneumatic hammer.

Hand Riveting. Hand riveting is now used only on small jobs, the air gun being replaced by the sledge hammer. The rivet is first hammered down by blows from the sledges, then the rivet set is applied and sledged to form the head to its proper shape.

Perfect rivets can be driven by either of the above methods.

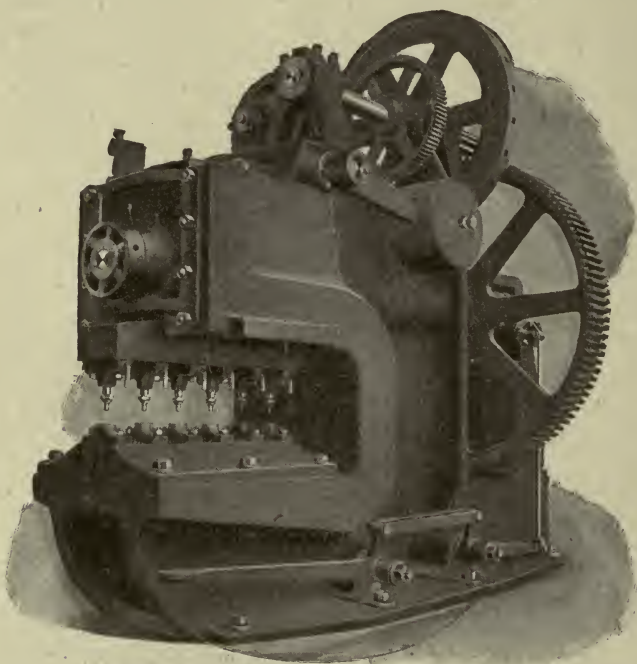


Fig. 60. Heavy Motor-Driven Multiple Punch
Courtesy of Mackintosh, Hemphill & Company

Punching and Reaming. Rivet holes in structural steel work are ordinarily punched in the metal by means of a powerful punching machine, Figs. 59 and 60 showing examples of the single and multiple types, respectively. The essential features of the machine for doing this work are a punch and a die. The die is usually about $\frac{1}{16}$ inch larger in diameter than the punch. The two are placed in the machine so that their axes are exactly in line. The plate is placed over the die and the punch is forced through, thus shearing out a round piece. This resulting hole is not perfectly smooth. The degree of roughness will depend on the condition of the punch and die, and the amount of difference in their diameters. The metal around the hole is to some extent torn and distorted.

For ordinary structural purposes the holes are accurate enough and the damage to the metal so slight that no further treatment is needed, but in railroad structures and sometimes for special cases of building work it is required that the holes be reamed. In such cases the hole is punched smaller than the size of the rivet—called "sub-punching"—and it is then enlarged to the proper size by means of a drill or reamer. In railroad bridge construction, it is customary to ream all metal over $\frac{5}{8}$ inch in thickness and to ream all holes for field connections. In structural work for buildings, reaming is not required to such a great extent. Sometimes it is required on metal thicker than $\frac{3}{4}$ inch and on field connections of very heavy members where a slight inaccuracy would occasion serious inconvenience in erecting.

Where the several pieces assembled together have a thickness of more than four times the diameter of the rivet, or where through any inaccuracy of punching the holes do not match accurately, the holes should be reamed to true them up; but in such cases they need not be sub-punched and the reaming only serves the purpose of trimming up the irregularities.

As previously stated, the diameter of the rivet hole as punched is $\frac{1}{16}$ inch larger than the diameter of the rivet; but in order to take account of the injured metal in computing the net section, the hole is figured $\frac{1}{8}$ inch larger than the rivet.

Functions of Rivets and Bolts. Rivets and bolts are used for fastening together the several sections used in building up the structural steel members and for connecting the members together in the finished structure. Rivets are always used for this purpose unless there is some special reason for using bolts. Generally speaking, rivets are cheaper than bolts and for most purposes more effective. They fill the holes full even though the holes may be slightly irregular in shape, and if driven tight will remain so; whereas bolts, unless they are turned and driven tight into reamed holes, are apt to become loose after a time.

The function of rivets is to hold one piece of steel to another and to transmit stress from one to the other. In so doing they must resist a bearing pressure and a shearing stress.

In many cases the rivets are not subjected to any definite shearing or bearing stress, but simply serve to hold the steel sections together in built-up members. They are unquestionably subjected to some stresses, but it is not possible to determine just what these are. In such situations the spacing of rivets is governed by rules resulting from practical experience.

It sometimes happens that the direction of the stress applied to the rivet is along its axis, that is, the rivet is subjected to tension. It was formerly the custom to specify that rivets should not be

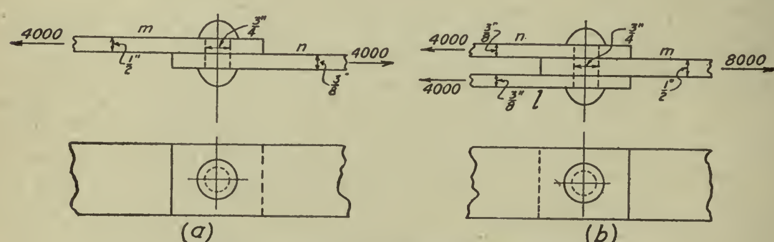


Fig. 61. Diagrams Showing Stresses in Rivets

subjected to tension; but that bolts should be used in such situations. This provision was necessary when wrought-iron rivets were in use, as their heads could be easily broken off. Steel rivets are much more reliable in this respect and, if properly driven, can be subjected to tension as safely as bolts.

Bearing. Fig. 61–a represents two pieces, *m* and *n*, riveted together, so that the stress (4000 pounds) in *m* is transmitted to *n*. Fig. 61–b represents three pieces riveted together so that the stress (8000 pounds) in the center piece *m* is transmitted to the two outside pieces *l* and *n*.

The bearing on the rivet is the pressure exerted on it by the plate through which it passes. In Fig. 61–a the bearing from plate *m* is on the right half of the rivet and from plate *n* on the left half of the rivet. Although the actual bearing is on the curved surface,

i. e., one-half the circumference of the rivet, the area used in figuring is the projected area of this surface, i. e., the thickness of the plate multiplied by the diameter of the rivet. For the plate m, the area is $\frac{1}{2}'' \times \frac{3}{4}''$ or .375 sq. in., and for plate n, $\frac{3}{8}'' \times \frac{3}{4}''$ or .281 sq. in.

The unit stress allowed in bearing is 25,000 pounds per square inch for shop-driven rivets; thus the allowed values in bearing are

for m $0.375 \times 25{,}000 = 9375\#$
for n $0.281 \times 25{,}000 = 7025\#$

The stress actually transmitted is 4000 pounds, and each bearing must be good for at least this amount, hence the bearings are sufficient.

The actual bearings per square inch are

for m $4000 \div 0.375 = 10{,}600\#$
for n $4000 \div 0.281 = 14{,}200\#$

PROBLEM

Compute from the above data the allowable bearing values for m and n and the actual bearing per square inch on m and n for field-driven rivets.

In Fig. 61–b the stress is transmitted from the plate m to the plates l and n and divided equally between them. The bearing areas are

for m $\frac{1}{2}'' \times \frac{3}{4}'' = 0.375$ sq. in.
for l and n combined $2 \times \frac{3}{8}'' \times \frac{3}{4}'' = 0.5625$ sq. in.

The allowed bearing values on shop-driven rivets are

for m $0.375 \times 25{,}000 = 9375\#$
for l and n combined $0.5625 \times 25{,}000 = 14{,}065\#$

The stress actually transmitted is 8000 pounds, so that the bearing for m is 8000 pounds and for l and n 4000 pounds each; hence, the bearings are sufficient.

The actual bearings per square inch are

for m $8000 \div 0.375 = 21{,}300\#$
for l and n combined $8000 \div 0.5625 = 14{,}200\#$

PROBLEM

Compute from the above data the allowable bearings for l, m, and n for field-driven rivets.

Shear. Referring again to Fig. 61–a, the forces acting on the two plates tend to cut, or shear, the rivet along the plane between the plates. This shearing action is resisted by the cross-section area of the rivets. This sectional area is $\frac{\pi d^2}{4}$, which in this case is

$\frac{3.1416}{4} \times \frac{3}{4} \times \frac{3}{4}$ or 0.4418 sq. in. The unit stress allowed in shear on shop-driven rivets is 12,000 pounds per square inch. Then the allowable value for one $\frac{3}{4}$-inch rivet is $12{,}000 \times 0.4418$ or 5302. This is greater than the actual stress applied and is sufficient.

The actual shear on the rivet per square inch of cross section is

$$4000 \div 0.4418 = 9054\#$$

Problem

Compute the shearing value of a $\frac{3}{4}$-inch rivet, field driven.

In Fig. 61–b there is a tendency to shear the rivet along two planes, i. e., on each side of the plate m. Consequently the shearing value of one rivet in this case is twice the value computed above, or 2×5302 or 10,604 pounds, and is sufficient to carry the actual load, which is 8000 pounds. The actual shear per square inch is the same as before, because both the actual load and the total cross-section area resisting it are twice as much as before, giving

$$8000 \div (2 \times 0.4418) = 9054\#$$

In the first case the rivet is in single shear, in the second case it is in double shear. It is clear that rivets should be used in double shear wherever possible, provided the middle plate has a bearing value more than that of a rivet in single shear.

Problems

1. Compute the shear value for shop-driven rivets of the following sizes: $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$, $\frac{7}{8}$, and 1 inch, respectively, for (a) single shear and (b) double shear.
2. Compute similar values for field-driven rivets.

Illustrative Example. In the case illustrated in Fig. 61–a, what thickness of plate n is required to make the bearing value equal the shearing value? The shearing value is 5302 pounds. The bearing area required is $\frac{5302}{25{,}000}$ or 0.212 sq. in. The diameter of rivet being 0.75 in., the thickness required to give the required area is 0.212 sq. in. $\div$ 0.75 or 0.283 in. The next higher commercial size is 0.3125 in. or $\frac{5}{16}$ in. thick.

Problems

1. In the case illustrated in Fig. 61–b compute the thickness of plate m required to make the bearing value equal the shearing value.
2. Compute the thickness of plates whose bearing values correspond to the single shear values of $\frac{1}{2}$-in., $\frac{5}{8}$-in., $\frac{7}{8}$-in., and 1-in. rivets. Compute the same for double shear values.

3. How many $\frac{7}{8}$-in. rivets, field driven, single shear, are required to transmit 175,000 pounds?

4. How many $\frac{5}{8}$-in. rivets, field driven, double shear, are required to transmit 100,000 pounds?

5. Assume shop rivets in double shear, middle plate $\frac{1}{2}$-in. thick. How many $\frac{3}{4}$-in. rivets are required to transmit 235,000 pounds? How thick must be the outside plates?

The designer can readily fix in mind the thickness of plates which give bearing values corresponding to the shear values of the rivets, then it will be necessary to compute only the shearing values.

Friction. If the plates are held together when the rivet is driven, the shrinkage in length as the rivet cools will exert considerable pressure. This makes the riveted joint develop a frictional resistance, which is additional to the shear and the bearing resistance. The amount of this friction has not been accurately determined. Furthermore, it may have no value if the rivets are not tight. Consequently, no account is taken of the friction in figuring the strength of riveted joints.

Tension. Specifications do not usually assign any value to rivets in tension. While their use in this manner is to be avoided, they may be so used when conditions require it. The unit stress allowable is the same as for shear. (See p 51).

Rivet Tables. The handbooks contain tables giving the shearing and bearing values of rivets. These tables cover several values of unit shearing stress and unit bearing stress. They give the diameter of rivet, area of cross section, single shear, double shear, and the bearing for various thicknesses of plates.

PROBLEM

Refer to the rivet tables and check all the examples and problems that have been given.

If the handbook does not contain tables based on the unit stresses given herein, prepare such tables and keep them for future use. Most handbooks have blank pages in the back part of the book for such use.*

Investigation of Riveted Joints. The theoretical strength of a riveted joint involves three elements: the bearing value of the rivets; the shearing value of the rivets; and the area of the section of metal after deducting rivet holes.

*The student should become familiar with all the tables given in the handbook relating to rivets and bolts.

In a perfect design these three elements would be equal in value, but this ideal is rarely reached. Most frequently it is the shearing value which determines the strength of the joint, next the bearing value, and least frequently the section of the metal.

Illustrative Example. Fig. 62 illustrates a splice of two plates, each $7'' \times \frac{3}{8}''$. Rivets $\frac{3}{4}''$ diameter, field driven.

(a) Using all of the ten rivets,

Shear value	10×4418	$=44,180\#$
Bearing value	10×5625	$=56,250\#$
Tension value at (1)	$6\frac{1}{8} \times \frac{3}{8} \times 16,000$	$=36,750\#$
Tension value at (2)	$5\frac{1}{4} \times \frac{3}{8} \times 16,000$	$=31,500\#$
Loss of tension value between (1) and (2)		$=5,250\#$

As this loss is more than the amount transmitted from m to n by the

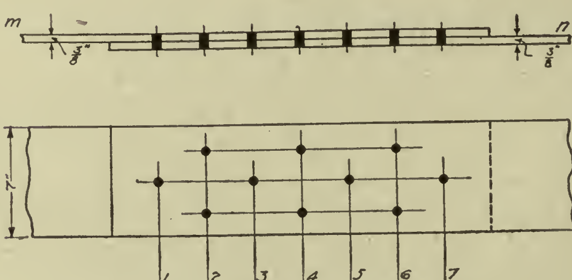


Fig. 62. Diagrammatic Views of a Riveted Joint

rivet at (1), the entire tension value at (1) is not available and the strength of the joint is the tension value at (2) plus the shear value of the rivet at (1), or $31,500+4418=35,918\#$.

(b) Now consider that the rivets at (1) and (7) are omitted.

Shear value	8×4418	$=35,344\#$
Bearing value	8×5625	$=45,000\#$
Tension value at (2)	$5\frac{1}{4} \times \frac{3}{8} \times 16,000$	$=31,500\#$

The strength of the joint is the tension value at (2), i. e., 31,500#.

(c) Next consider that the rivets (4) are omitted.

Shear value $8 \times 4418=35,344\#$

Tension value at (2) plus shear value of rivet at (1) as above $=35,918\#$

The strength of the joint is the shear value 35,344#.

(d) Finally omit the rivet at (3).

Shear value $9 \times 4418 = 39{,}762\#$

Strength of joint same as in (a) $= 35{,}918\#$

From the above it is clear that the maximum strength of the joint

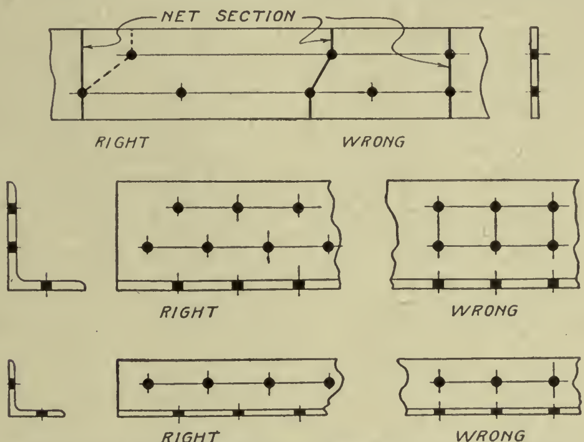
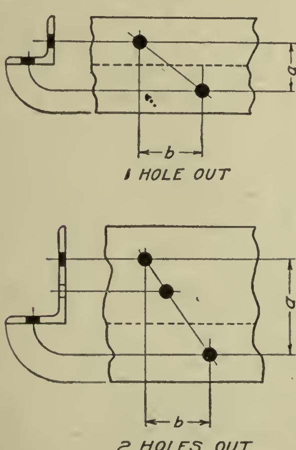


Fig. 63. Diagrams Showing Right and Wrong Arrangement of Rivet Holes in Tension Members

that can be made in this case is 35,918 pounds. It requires 9 rivets as in (d). Nearly the same strength can be secured with 8 rivets as in (c), 35,344 pounds.



	¾″ Rivet	⅞″ Rivet		¾″ Rivet	⅞″ Rivet
a	b	b	a	b	b
1	1⅝	1¾	5	$3\frac{1}{16}$	$3\frac{5}{16}$
1½	1⅞	2	5½	3¼	3½
2	$2\frac{1}{16}$	2¼	6	3⅜	3⅝
2½	2¼	$2\frac{7}{16}$	6½	3½	3¾
3	$2\frac{7}{16}$	2⅝	7	3⅝	3⅞
3½	$2\frac{9}{16}$	$2\frac{13}{16}$	7½	3¾	4
4	$2\frac{13}{16}$	3	8	3⅞	4⅛
4½	$2\frac{15}{16}$	$3\frac{3}{16}$	8½	4	4¼

a = Sum of gages minus thickness of angle

Note: ⅝″ Rivets can be taken at ⅛″ less than for ¾″ Rivets
1″ Rivets can be taken at ⅛″ more than for ⅞″ Rivets

Fig. 64. Stagger of Rivets Required to Maintain Net Section.
From "Standards for Detailing" American Bridge Company

The important point to be observed from this example is the difference between (a) and (b); the loss of section by rivet holes should be made as gradual as possible.

PROBLEM

Go through the operations of the above example on the basis of shop rivets.

Net Section. The holes in angles can usually be arranged so that only one need be deducted with two or three rows, and two with four rows. This is not always true for the large angles. Fig. 63 illustrates the right and wrong arrangement of holes in a number of cases. Fig. 64 illustrates the pitch of staggered rivets required to maintain the net section. If the joint is in compression no deduction is made in the cross section on account of rivets, and the rivets need not be staggered unless required for minimum spacing.

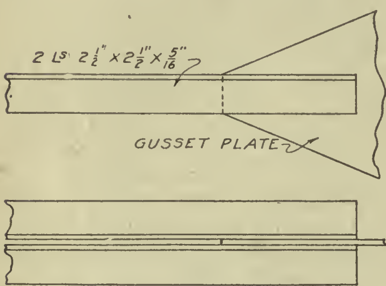


Fig. 65. Two Angles Attached to a Gusset Plate

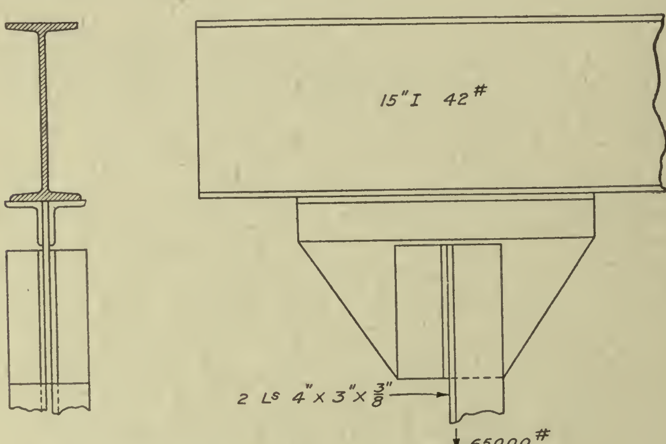


Fig. 66. Side and End View of a Riveted Hanger

PROBLEMS

1. Fig. 65 shows two angles in tension to be connected to a gusset plate with shop rivets. Determine the following:

Size of rivets
Net section of angles

Tension value of net section

Thickness of gusset plate to develop the full double shearing value of the rivets

Number of rivets

Locate gage line and space the rivets

Draw plan, elevation, and section of joint at $\frac{3}{4}$-inch scale

NOTE. The connection illustrated is poor on account of secondary stress, p. 234. It is used only for practice.

2. Fig. 66 shows a hanger connected to the underside of an I-beam. The hanger is made of 2 Ls $4'' \times 3'' \times \frac{3}{8}''$ and carries a load of 65,000 pounds. Determine the following:

Size of rivets

Total section of two angles

Net section of two angles after deducting one rivet hole from each

Whether section is sufficient for the load applied

Thickness of gusset plate to develop the double shearing value of rivets

Number of shop rivets to connect lug angles to main angles (assume

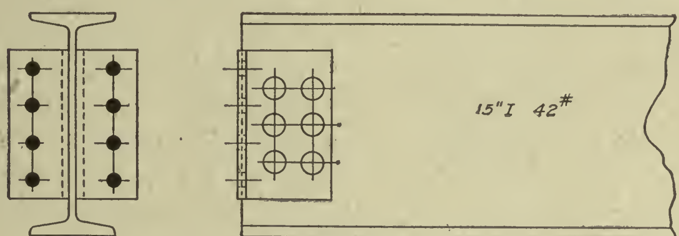


Fig. 67. Side and End View of Standard Beam Connection

that one-half of load is transmitted through the lug angles)

Number of shop rivets to connect hanger to gusset plate

Number of shop rivets to connect gusset plate to top angles

Number of field rivets (in tension) to connect top angles to I-beam

Make drawing at $\frac{3}{4}$-inch scale, showing all dimensions and rivet spacing

Give page numbers of handbook for all references used in the above operations

3. Fig. 67 shows the standard end connection for a 15" I 42#. What is the strength of the connection?

Bolts.* The foregoing discussion of rivets applies also to bolts, except as to stresses allowed and as to bolts in tension.

Turned bolts fitting tight in reamed holes have the same values as field rivets. Machine bolts should be allowed only three-fourths the unit stresses of field rivets, i. e., 7500 pounds per square inch shear and 15,000 pounds per square inch bearing.

*The student should obtain a catalogue from a bolt manufacturer and become familiar with the standard and special bolts on the market.

In general, the use of bolts in the permanent structure should be discouraged, being limited to locations where it is impracticable to drive rivets and to connections where they serve simply to hold the members in position and do not transmit stress. The cost of using turned bolts will prevent their use where rivets can be used. But machine bolts are cheaper than rivets for most field connections and their use must be forbidden except in cases where they are suitable.

Turned Bolts. Turned bolts, as the name indicates, are turned to exact diameter in a lathe. The holes for turned bolts must be reamed after the members are assembled, or both members must be reamed to fit the same template. The reamer must have the same diameter as the finished bolt so as to give a driving fit.

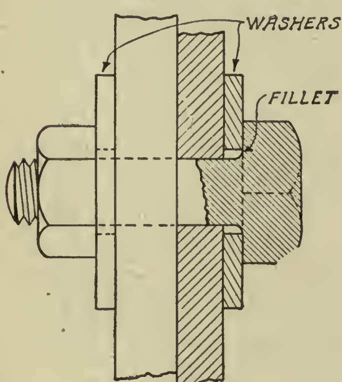


Fig. 68. Part Section of Bolted Joint Showing Fillet Under Bolt Head

Washers must be used under both the head and nut. Refer to Fig. 68 and note that there is a fillet under the head. If the washer is not used, this fillet will prevent the head from bearing against the plate. If the thread is cut long enough to allow the nut to bear against the plate, the thread will extend into the hole; hence a washer is used so that the thread need not be cut so long. After the nut is drawn up tight, the threads should be checked with a chisel so that it cannot become loosened.

Machine Bolts. Machine bolts are made from rods as they come from the rolling mill and are not finished to exact size. They do not fill the holes fully. Their principal use is for assembling material in the shop or in the structure, preparatory to riveting. They may remain in the finished structure if not subjected to shear. Such a case is a beam resting on another.

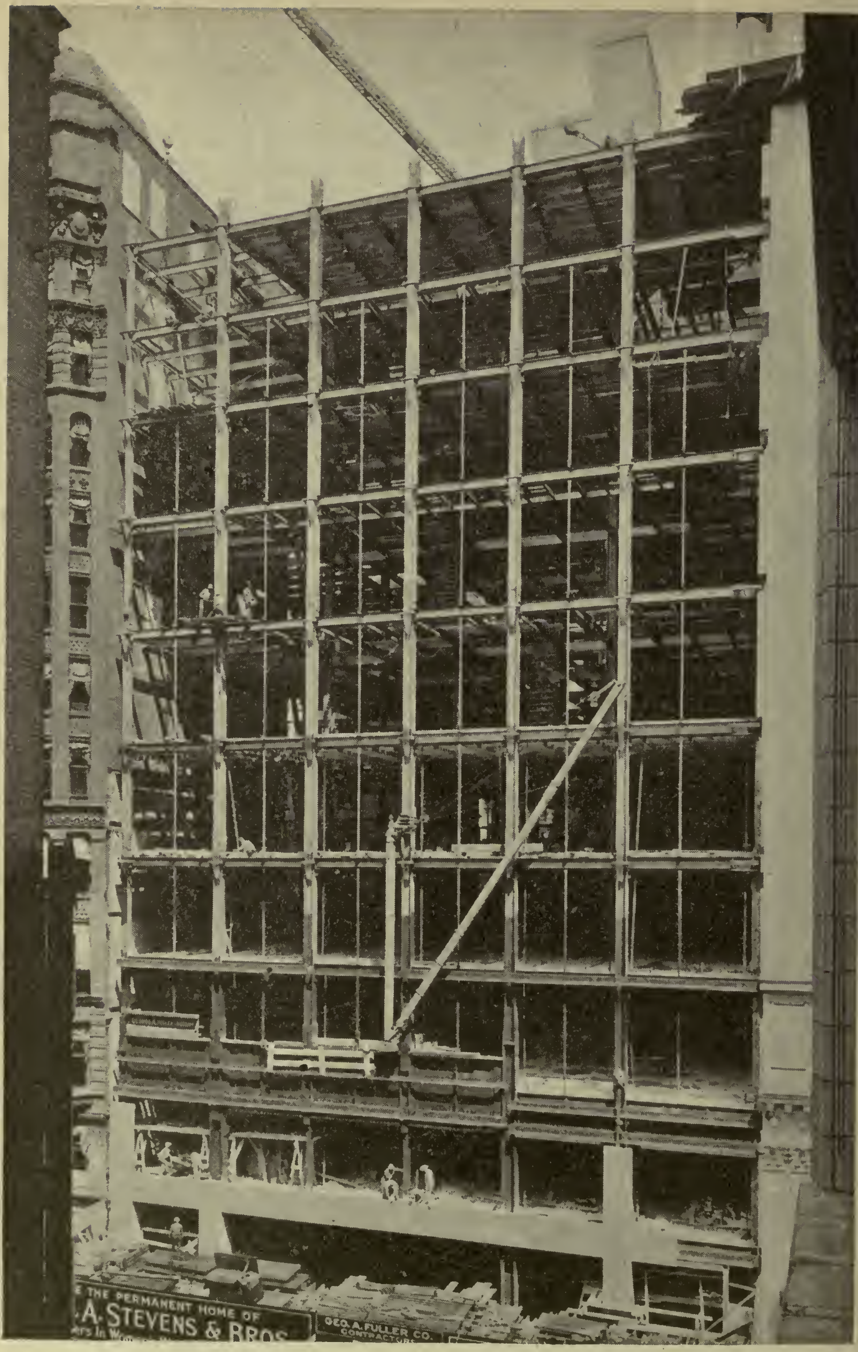
Bolts in Tension. When a bolt is used in tension, the net area available to resist the stress is the area at the root of the thread. For example, determine the tensile strength of a $\frac{3}{4}$-inch bolt. Referring to the handbook, it is found that the diameter at the root of the thread is 0.62 inches. From this, the area, if not given in the table, can be computed and is found to be 0.30 square inch. Then the tension value is $0.30 \times 10{,}000$ or 3000 pounds.

Two nuts should be used on bolts in tension to prevent stripping the threads, and the threads should be checked after the nuts are tightened.

Length of Rivets and Bolts. The grip of a rivet or bolt is the thickness of the material through which it passes. The grip estimated is the nominal thickness of metal plus $\frac{1}{16}$ inch for each piece of metal.

The length of rivet required for a given case is the grip plus the amount of stock required to form the head and for filling the hole when the rivet is upset. The lengths required for various grips are given in the handbooks.

The length of bolt required for a given case is the grip plus the thickness of the washers, plus the thickness of the nut (or two nuts if in tension), plus $\frac{1}{2}$ inch.



VIEW OF STRUCTURAL STEEL WORK FOR THE STEVENS BUILDING, CHICAGO

Note the special girder at second floor designed to carry the column load from second floor to the top.

Courtesy of George A. Fuller Company, Contractors, Chicago

STEEL CONSTRUCTION

PART II

BEAMS

Definitions. A *beam* is a structural member subjected to a load applied perpendicular to its longitudinal axis. Usually the beam is in a horizontal position and the load is applied vertically downward. It is supported at the ends (unless it is a cantilever). The space between the supports is the span.

The word beam is a general term which applies in all cases to a member subjected to bending by a transverse load, irrespective of the use to which it is put. There are a number of special terms which have reference to the position or use of the beam.

A *joist* is a beam which supports the floor or other load direct.

A *girder* is a beam which supports one or more joists or other beams.

A *lintel* is a beam which supports the wall above an opening therein.

A *spandrel beam* is one which supports the masonry spandrel between the piers of a wall.

Elevator beams, sheave beams, stair stringers, crane girders, etc., are used for the purposes indicated by their names.

Built-up beams are usually called "girders" irrespective of their uses. There are plate girders, box girders, beam box girders, etc.

The *span* of a beam is the distance between supports, or, in the case of a cantilever, the distance from the support to the end of the beam.

Classification. Beams are classified as simple and restrained. A *simple beam* is one which has a single span and merely rests on its supports, there being no rigid connection to prevent normal bending. A *restrained beam* is one which has more than one span or is rigidly connected at one or more supports, or otherwise prevented from normal bending. Fig. 69 illustrates a simple beam and several forms of restrained beams, showing in an exaggerated way the forms they assume when bending under load.

Although most beams in steel construction are somewhat restrained by their end connections, they are treated as simple beams in designing. Beams extending over more than two supports are very rarely used in building construction and are not considered in this text. Cantilever beams occur in the form of a beam projecting from a support to which it is rigidly attached, and in the form of a beam spanning from one support to another, and projecting beyond one or both supports.

Sections. The structural steel section most used as a beam is the I-beam. It is designed for this purpose and is the most efficient form in which the steel can be made for resisting bending. Channels, angles, and tees are used only to meet some special condition. The built-up or riveted girders imitate the I-beam and are used for

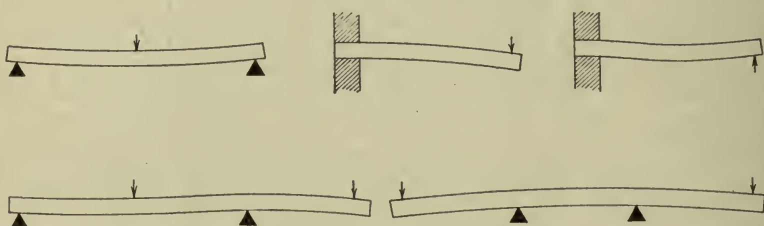


Fig. 69. Simple, Cantilever, and Restrained Beams

loads which are too great to be supported by the rolled section. This part of the text deals only with rolled sections. Riveted sections are given later.

REVIEW OF THEORY OF BEAM DESIGN

Factors Required in a Complete Design. The complete design of a beam requires the computation of the bending moments and shears resulting from the assumed loading, and of the resisting moment, shearing resistance, and deflection of the beam section which it is proposed to use. The resisting moment usually governs.

Maximum Bending Moment. The resisting moment based on the allowable unit stress must be equal to or greater than the maximum bending moment. As the section of the rolled beam is the same from end to end, its resistance is constant throughout its length. Hence, it is necessary to compute only the maximum bending moment. The position and amount of the maximum bending moment are computed later in the text for various conditions of loading.

Maximum Shear. The shearing resistance based on the allowable unit stress must be equal to or greater than the maximum shear. The shearing resistance of the rolled beam is constant throughout its length. Hence, it is necessary to compute only the maximum shear. The position of maximum shear in single span beams is always adjacent to the support which has the greater reaction.

Deflection. A beam subjected to bending stresses must have some deflection, and, under certain conditions, the amount of this deflection must be limited. For example, the floor section, Fig. 70, shows that the deflections in the joists were so great as to cause a bad crack in the marble floor above the steel girder.

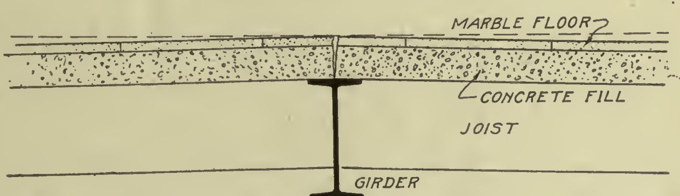


Fig. 70. Floor Section Showing Crack Over Girder, Due to Deflection of Joists

The definitions and methods of computing bending moments, shears, resisting moments, shearing resistance, and deflection are given in "Strength of Materials." The student should review those topics before proceeding with this text. The following brief discussion may help to fix in mind the important points.

Flexure. It is a matter of common observation that a loaded beam deflects or sags between the supports. This is most evident in wood beams, but is true of beams of all materials. This deflection stretches the fibers at the bottom of the beam, i. e., produces tension; and shortens the fibers at the top of the beam, i. e., produces compression. Somewhere between the top and the bottom the fibers are neither stretched nor shortened, hence there is no stress; this place is called the "neutral axis" and passes through the center of gravity. In I-beams and channels the neutral axis is at mid-depth.

This is also true of rectangular wood beams. The intensity of stress—tension or compression—corresponds to the amount of deformation—lengthening or shortening; hence, the intensity varies with distance from the neutral axis, being zero at the neutral axis and maximum at the extreme fibers* at the top and bottom. This is illustrated in Fig. 71. The stress on the extreme fiber—not the average stress—governs in designing. The working or unit stress allowed is 16,000 pounds per square inch in both the tension and the compression flanges. (See Unit Stresses, p. 51.)

In Fig. 71 assume that each arrow represents the stress on a unit area, the length of the arrow representing amount or intensity of the stress. To find the resistance of the beam to bending it must be remembered that the resisting moment is the sum of the moments of all stresses about the neutral axis. Under Strength of Beams, in "Strength of Materials," Part I, it is shown that the resisting moment is expressed by the formula

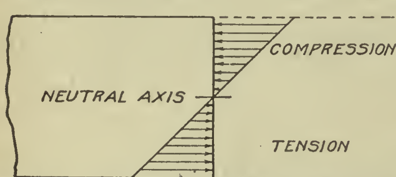


Fig. 71. Graphical Representation of Stresses in the Fibers of a Beam

$$M=\frac{S\,I}{c}$$

in which M is resisting moment in inch-pounds; I is the moment of inertia in terms of inches; c is the distance from the neutral axis to the extreme fiber in inches; and S is the maximum fiber stress, that is, the stress on the extreme fiber in pounds per square inch. From this formula the resisting moment of the beam can be computed.

Assume a 12″ I $31\frac{1}{2}$#. From the handbook the value of I is 215.8. The distance from the neutral axis to the extreme fiber is 6 inches. The allowable unit stress on the extreme fiber is 16,000 pounds per square inch. Then

$$M=\frac{S\,I}{c}=\frac{16{,}000\times 215.8}{6}=575{,}467 \text{ in.-lb.}$$

When the unit stress S, resulting from a given bending moment, is required, the formula is transposed into the form

$$S=\frac{M\,c}{I}$$

*The term extreme fiber is correctly used in relation to wooden beams as wood is a fibrous material. Steel is not a fibrous material but the term expresses the idea clearly and is generally used.

Assume that the bending moment is 500,000 inch-pounds and that the beam is 12″ I $31\frac{1}{2}$#, then,

$$S=\frac{500{,}000\times 6}{215.8}=13{,}900\#$$

Vertical Shear. Fig. 72 illustrates a beam with a heavy load applied close to one support. There is a tendency for the part on the left of the vertical plane *a a* to slide downward in relation to the part on the right. This is prevented by the shearing resistance of the beam. This shearing tendency exists throughout the length of the beam but is greatest near the supports. In this case the maximum shear is adjacent to the right support at *a a* and is assumed to be 45,000 pounds. It is resisted by the strength of the steel at this section. The average stress over this section is the total vertical shear divided by the area and is expressed by the formula

$$S_s=\frac{V}{A}$$

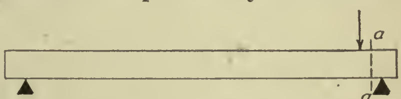


Fig. 72. Diagram Illustrating Shear on a Beam

in which S_s equals shearing stress per square inch; V equals total vertical shear; and A equals area in square inches. But it can be shown that the shear is not uniform over this area, being zero at the extreme fiber and a maximum at the neutral axis. The exact maximum value is difficult to compute, but it can be determined approximately by assuming that the entire shear is resisted by the web of the beam (see Resisting Shear, "Strength of Materials" Part II); then the above formula is used, making A equal the area of the web in square inches. In this case assume that the beam is 12″ I $31\frac{1}{2}$#. The area of the web is approximately 12″× .35″=4.2 sq. in. Then $S_s=\frac{45{,}000}{4.2}$ or 10,714 pounds per square inch. The allowable value given under Unit Stresses is 10,000 pounds per square inch, and the beam is over-stressed in shear.

If it is desired to compute the maximum resistance to shear for this beam, the formula is put in the form

$$V=S_s\times A$$

and for this case

$$V=10{,}000\times 4.2=42{,}000\#$$

A beam subjected to an excessive load would not fail by the actual shearing of the metal along the plane *a a* but by the buckling of the web. This has been taken into account in establishing the unit stress.

Deflection. As previously stated, a beam which is subjected to bending stresses must deflect a certain amount. The amount of deflection depends on the load, the length of span, and the section of the beam. It is expressed by the formulas:

(1) For a uniformly distributed load

$$d = \frac{5}{384}\,\frac{W\,l^3}{E\,I}$$

(2) For a load concentrated at center of span

$$d = \frac{1}{48}\,\frac{W\,l^3}{E\,I}$$

in which d equals deflection in inches; W equals total load in pounds; l equals span in inches; I equals moment of inertia; and E equals modulus of elasticity equals 30,000,000.

Modulus of Elasticity. The modulus of elasticity is the ratio of the unit stress to the unit deformation. If a piece of steel one inch square and ten inches long is subjected to a tensile stress of 20,000 pounds, the unit stress is 20,000 pounds per square inch. The steel is elongated about $\frac{1}{150}$ inch and, therefore, the unit deformation, or the elongation of one inch in length, is $\frac{1}{1500}$ inch. Then the ratio of unit stress to unit deformation is $\frac{20{,}000}{\frac{1}{1500}} = 30{,}000{,}000$. This ratio has been determined by experiment. It is the same for both tension and compression. Other materials have other values.

CALCULATION OF LOAD EFFECTS

Uniformly Distributed Loads. The first step in designing a beam is to determine the bending moments and shears resulting from the assumed loading. The methods of computing them are given under External Shear and Bending Moment, "Strength of Materials," Part I.

Joists. The loads on joists are usually distributed uniformly along the length of the beam. Assume that the simple beam, Fig. 73, has a span $L=17'\text{-}6''$, and supports a load of 800 pounds per lineal foot.

$$\text{Total load} = W = 17.5 \times 800$$
$$= 14{,}000\#$$

Since the load is uniformly distributed, the reactions are equal:

$$R_1 = R_2 = \tfrac{1}{2}\,W = \frac{14{,}000}{2} = 7000\#$$

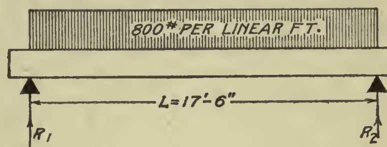


Fig. 73. Diagram of Beam Uniformly Loaded

The maximum shear occurs adjacent to each support and its amount is the same as the reaction, hence V_1 and V_2 have the same values as R_1 and R_2.

The maximum bending moment occurs at the middle of the span and has a value

$$M = R_2 \times \frac{17.5}{2} - \tfrac{1}{2}W \times \frac{17.5}{4}$$

$$R_2 \times \frac{17.5}{2} = 7000 \times 8.75 = 61{,}250 \text{ ft.-lb.}$$

$$\tfrac{1}{2}W \times \frac{17.5}{4} = 7000 \times 4.375 = 30{,}625 \text{ ft.-lb.}$$

$$M = \qquad 30{,}625 \text{ ft.-lb.} = 367{,}500 \text{ in.-lb.}$$

The formula for this bending moment is

$$M = \tfrac{1}{8}\,W\,L = \tfrac{1}{8} \times 14{,}000 \times 17.5 = 30{,}625 \text{ ft.-lb.}$$

Cantilever Beam. Fig. 74 represents a cantilever beam supporting a uniformly distributed load. Assume the length L of cantilever to be $8'\text{-}9''$, and the load, 800 pounds per lineal foot; then

$$W = 8.75 \times 800 = 7000\#$$
$$R_1 = \qquad 7000\#$$

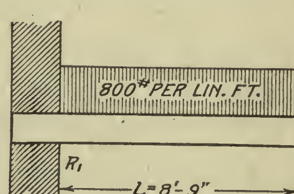


Fig. 74. Diagram of Cantilever Uniformly Loaded

The maximum bending moment is at the support, and therefore

$$M = W \times \frac{L}{2} = 7000 \times 4.375 = 30{,}625 \text{ ft.-lb.}$$

Compare these results with those obtained for the simple span having the same load per lineal foot. The span is one-half as much, while the shear and the bending moment are the same.

Combination Simple and Cantilever Beam. A beam resting on two supports, projecting beyond one of them, and supporting a uniformly distributed load is represented in Fig. 75. Assume the span L between supports to be 17′-6″, the length L' of the cantilever to be 8′-9″, and the load 800 pounds per lineal foot; then

$$W=(800\times 17.5)+(800\times 8.75)=21{,}000\#$$

The reactions must be determined by the method of moments. Take the moments about R_1. For the positive moment the lever arm is the distance from R_1 to the center of gravity of the entire beam, viz, 13.125 feet; therefore

$$\text{Positive moment}=21{,}000\times 13.125=275{,}625 \text{ ft.-lb.}$$

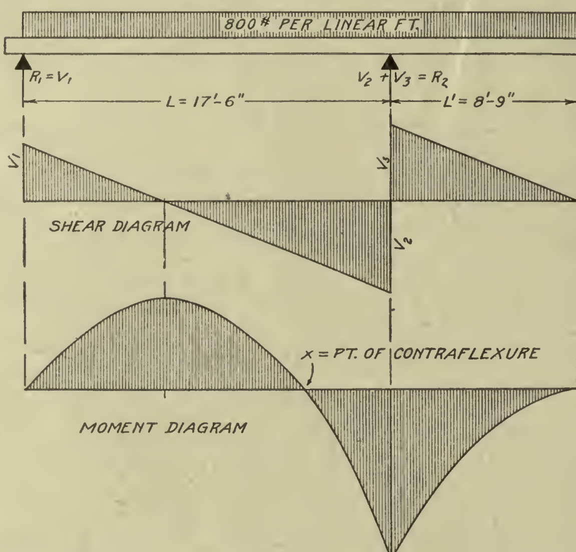


Fig. 75. An Overhanging Beam with Shear and Moment Diagrams

The negative moment must equal the positive moment; then

$$R_2\times L=275{,}625$$

and the value of R_2 is found by dividing the positive moment by the distance L between supports.

Therefore $$R_2=\frac{275{,}625}{17.5}=15{,}750\#$$

Now since the sum of the reactions must equal the total load, the value of R_1 can be determined by subtracting R_2 from W; then

$$R_1 = 21{,}000 - 15{,}750 = 5250\#$$

This value of R_1 can be checked by taking moments about R_2.

The position of the maximum shear is not self-evident so the shear values must be computed. $V_1 = 5250$. Proceeding toward the right, 800 pounds is deducted for each foot, so the shear becomes zero at 6.5625 feet from R_1; continuing to a point just to the left of R_2, the value of the shear is

$$V_2 = 5250 - (800 \times 17.5) = -8750\#$$

Continuing, to the right, add the value of R_2; then the value of the shear is

$$V_3 = -8750 + 15{,}750 = +7000\#$$

Continuing, the shear reduces at the rate of 800 pounds per lineal foot, becoming zero at the end of the cantilever. The above values are shown graphically on the shear diagram.

The maximum positive bending moment is between R_1 and R_2 at the same position as the zero shear. Its value is

$$\left\{\begin{array}{ll} +5250 \times 6.5625 & = +34{,}448 \\ -800 \times 6.5625 \times \dfrac{6.5625}{2} & = -17{,}224 \end{array}\right\} = +17{,}224 \text{ ft.-lb.}$$

The maximum negative bending moment is at R_2. Its value computed on the right is

$$-800 \times 8.75 \times \frac{8.75}{2} = -30{,}625 \text{ ft.-lb.}$$

or computed on the left is

$$\left\{\begin{array}{ll} +5250 \times 17.5 & = +\ 91{,}875 \\ -\ 800 \times 17.5 \times \dfrac{17.5}{2} & = -122{,}500 \end{array}\right\} = -30{,}625 \text{ ft.-lb.}$$

The moment diagram can be constructed by computing the values at points one foot apart and plotting the results. From this diagram it will be noted that the bending moment changes from positive to negative at the point x. This is called the "point of contraflexure" and in this case it is located 13.125 feet from R_1.

It is usually easier to compute the bending moment for simple spans uniformly loaded from the formula

$$M=\frac{W\,L}{8}$$

and for cantilevers from the formula

$$M=\frac{W\,L'}{2}$$

For the combination span illustrated above, the maximum negative moment may be computed from the cantilever formula. But the maximum positive moment cannot be expressed in a simple formula and must be computed by means of the summation of moments as illustrated.

EXAMPLES FOR PRACTICE

1. A joist has a span of 21 feet. It supports a floor area $5\frac{1}{2}$ feet wide. The floor construction weighs 115 pounds per square

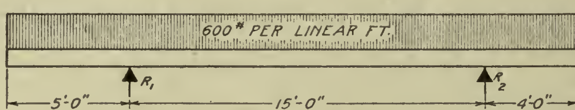


Fig. 76. Uniformly Loaded Beam Overhanging at Both Ends.

foot and the live load to be supported is 50 pounds per square foot. Compute the shear and bending moment.

2. What are the maximum shear and bending moment for a total load of 80,000 pounds uniformly distributed on a span of 8 feet; 10 feet; 12 feet; 14 feet; 16 feet? What is the ratio of the bending moments for the 8-foot, and the 16-foot spans?

3. Compute the maximum shears and bending moments for a beam supporting a uniformly distributed load of 1,000 pounds per lineal foot on a span of 8 feet; 10 feet; 12 feet; 14 feet; 16 feet. What is the ratio of the bending moments for the 8-foot and 16-foot spans?

4. Compute the maximum shears and bending moments for cantilevers from the data given for the preceding problem. Compare the results with those for the simple beam.

5. Fig. 76 represents a beam extending beyond both supports. Its load is 600 pounds per lineal foot. What is the maximum shear? What are the bending moments at R_1 and R_2? What is the maximum positive bending moment?

6. Construct the shear and moment diagrams for the preceding problems.

7. Given a span of 20 feet and a bending moment of 50,000 foot-pounds, what is the total uniformly distributed load?

$$\frac{50,000 \times 8}{20} = 20,000\#$$

8. Given a span of 18 feet and a bending moment of 72,000 foot-pounds, what is the load per lineal foot?

Concentrated Loads. Girders in floor construction usually receive their loads at points where joists connect.

Simple Beam. Fig. 77 represents a simple beam supporting the concentrated loads P_1, P_2, P_3, and P_4. The loads are

$$P_1 = 60,000\#$$
$$P_2 = 80,000\#$$
$$P_3 = 80,000\#$$
$$P_4 = 50,000\#$$
$$\text{Total load} = 270,000\#$$

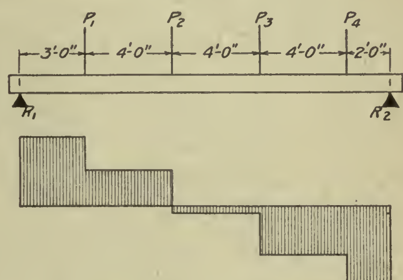


Fig. 77. Simple Beam with Concentrated Loads. Shear Diagram

To determine the reaction R_2, take moments about R_1:

$$3 \times 60,000 = 180,000$$
$$7 \times 80,000 = 560,000$$
$$11 \times 80,000 = 880,000$$
$$15 \times 50,000 = 750,000$$
$$2,370,000 \text{ ft.-lb.}$$

$$R_2 = \frac{2,370,000}{17} = 139,412\#$$

Similarly, to determine the reaction R_1, take moments about R_2:

$$2 \times 50,000 = 100,000$$
$$6 \times 80,000 = 480,000$$
$$10 \times 80,000 = 800,000$$
$$14 \times 60,000 = 840,000$$
$$2,220,000 \text{ ft.-lb.}$$

$$R_1 = \frac{2,220,000}{17} = 130,588\#$$

Therefore $\qquad R_1 + R_2 = 130,588 + 139,412 = 270,000\#$

which checks with the total load.

The maximum shear occurs at the left of R_2 and is 139,412 pounds. By constructing the shear diagram, it is found that the shear passes from positive to negative at P_2. This position of zero shear establishes the point of maximum moment. Computing the moment from the loads and reaction on the left gives

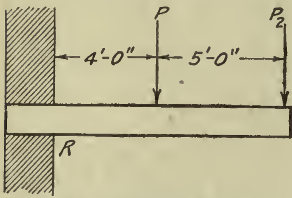


Fig. 78. Cantilever Beam with Concentrated Loads

$$\begin{aligned} +7\times130{,}588 &= +914{,}116 \\ -4\times\ 60{,}000 &= \underline{-240{,}000} \\ &\ +674{,}116 \text{ ft.-lb.} \end{aligned}$$

Computing on the right gives

$$\begin{array}{lr} +10\times139{,}412= & +1{,}394{,}120 \\ -\ 4\times\ 80{,}000=320{,}000 & \\ -\ 8\times\ 50{,}000=\underline{400{,}000} & -\ \underline{720{,}000} \\ & +\ 674{,}120 \text{ ft.-lb.} \end{array}$$

Cantilever Beam. Fig. 78 represents a cantilever supporting the concentrated loads P_1 and P_2.

$$R=P_1+P_2=30{,}000+40{,}000=70{,}000\#$$

The maximum shear is 70,000 at the right of R. Zero shear is at the right of P_2.

The maximum bending moment is at R. It is

$$\begin{aligned} -4\times30{,}000 &= -120{,}000 \\ -9\times40{,}000 &= \underline{-360{,}000} \\ &\ -480{,}000 \text{ ft.-lb.} \end{aligned}$$

Simple Beams on Two Supports and Projecting at Both Ends. Fig. 79 represents a beam resting on two supports and projecting beyond both of them. It supports concentrated loads as shown. The loads are

$$\begin{aligned} P_1 &= 15{,}000\# \\ P_2 &= 15{,}000\# \\ P_3 &= 15{,}000\# \\ P_4 &= 15{,}000\# \\ P_5 &= 15{,}000\# \\ P_6 &= \underline{30{,}000\#} \\ \text{Total load} &= 105{,}000\# \end{aligned}$$

To determine the reaction R_2, take moments about R_1:

$$\begin{array}{rlr}
0\times 15{,}000\ (P_2) & = 00{,}000 & \\
4\times 15{,}000\ (P_3) & = 60{,}000 & \\
8\times 15{,}000\ (P_4) & = 120{,}000 & \\
12\times 15{,}000\ (P_5) & = 180{,}000 & \\
16\times 30{,}000\ (P_6) & = \underline{480{,}000} & 840{,}000 \text{ ft.-lb.} \\
-\ 4\times 15{,}000\ (P_1) & = & -\ \underline{60{,}000} \\
\text{Moment of reaction } R_2 = & & 780{,}000 \text{ ft.-lb.}
\end{array}$$

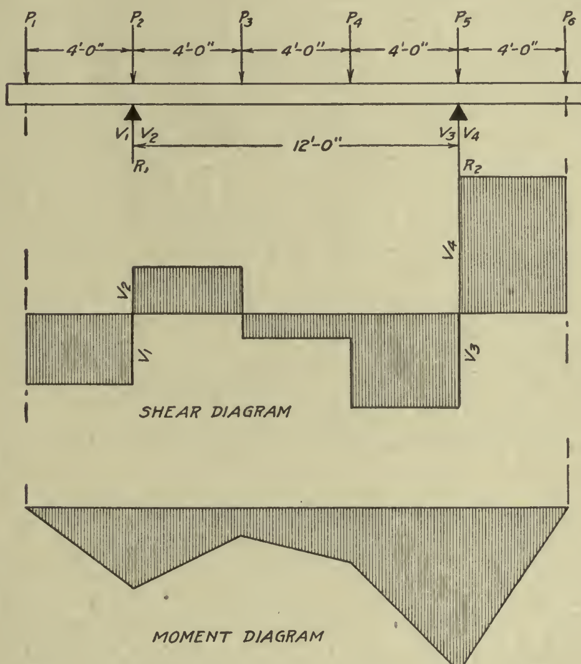


Fig. 79. Overhanging Beam with Concentrated Loads. Shear and Moment Diagrams

$$R_2 = \frac{780{,}000}{12} = 65{,}000\#$$

To determine the reaction R_1, take moments about R_2:

$$\begin{array}{rlr}
0\times 15{,}000\ (P_5) & = 00{,}000 & \\
4\times 15{,}000\ (P_4) & = 60{,}000 & \\
8\times 15{,}000\ (P_3) & = 120{,}000 & \\
12\times 15{,}000\ (P_2) & = 180{,}000 & \\
16\times 15{,}000\ (P_1) & = \underline{240{,}000} & 600{,}000 \text{ ft.-lb.} \\
-\ 4\times 30{,}000\ (P_6) & = & -\underline{120{,}000} \\
\text{Moment of reaction } R_1 = & & 480{,}000 \text{ ft.-lb.}
\end{array}$$

Therefore

$$R_1=\frac{480{,}000}{12}=40{,}000\#$$

The shear values are

$$V_1=15{,}000$$
$$V_2=10{,}000$$
$$V_3=20{,}000$$
$$V_4=30{,}000$$

Zero shear occurs at P_3.

The bending moments are maximum negative at R_1 and R_2 and maximum positive at P_3. Their values are

$$\text{at } R_1 \quad M= \ -4\times15{,}000= \qquad -\ 60{,}000 \text{ ft.-lb.}$$
$$\text{at } R_2 \quad M= \ -4\times30{,}000= \qquad -120{,}000 \text{ ft.-lb.}$$

$$\text{at } P_3 \quad M=\begin{cases}+4\times40{,}000=+160{,}000\\-4\times15{,}000=-\ \ 60{,}000\\-8\times15{,}000=-\underline{120{,}000}-20{,}000 \text{ ft.-lb.}\end{cases}$$

From the last result, it develops that the bending moment at P_3 is minimum negative (not considering the ends of the cantilevers) and not maximum positive. Hence there is no reversal of moment in this case. The moment diagram shows this.

EXAMPLES FOR PRACTICE

1. Solve the preceding case for the following loads: $P_1=10{,}000\#$; $P_2=10{,}000\#$; $P_3=15{,}000\#$; $P_4=20{,}000\#$; $P_5=20{,}000\#$; $P_6=15{,}000\#$. Construct the shear and moment diagrams.

2. What are the maximum shear and bending moment for a load of 40,000 pounds at the center of an 8-foot span? Of a 10-foot span? Of a 12-foot span? Of a 14-foot span? Of a 16-foot span? What is the ratio of the bending moments for the 8-foot and 16-foot spans?

Compare these results with those from the second problem under uniformly distributed loads and note that the bending moments are the same though the uniformly distributed load is twice the concentrated load.

3. Compute the shear and bending moment for two loads of 40,000 pounds each, placed at the third points of a 16-foot span; at the quarter points. Compare with the preceding problem.

4. A load at the center of a 20-foot span produces a bending moment of 200,000 foot-pounds. What is the load?

5. Two equal loads at the quarter points of a 20-foot span produce a bending moment of 100,000 foot-pounds. What are the loads?

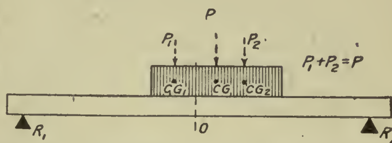


Fig. 80. Simple Beam with Uniformly Distributed Load over Part of Span

Combined Loads. Under "combined loads" are considered the combinations of uniformly distributed and concentrated loads, and of uniformly distributed loads on parts of spans. In computing moments in these cases, the uniformly distributed load may be considered as concentrated at its center of gravity, Fig. 80, unless the center of moments is within the space occupied by the load; in which case the parts of the load to the right and to the left of the center of moments must be considered as concentrated at their respective centers of gravity. Thus, if the center of moments is at R_1 or R_2, the concentrated load P is used; but if the center of moments is at O the concentrated loads P_1 and P_2 are used. The same principle applies if the distributed load is variable instead of uniformly distributed, Fig. 81.

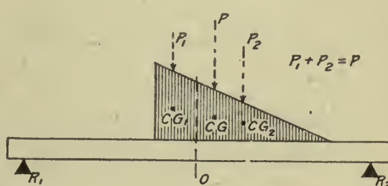


Fig. 81. Simple Beam with Variable Load

Full Length Distributed Load and Concentrated Load. Fig. 82 illustrates a beam with a uniformly distributed load full length and a concentrated load, as shown.

Total load = 10,000# (u.d.) + 10,000# (con.) = 20,000#

Moments about R_1 are

10×10,000 = 100,000
15×10,000 = 150,000

250,000 ft.-lb.

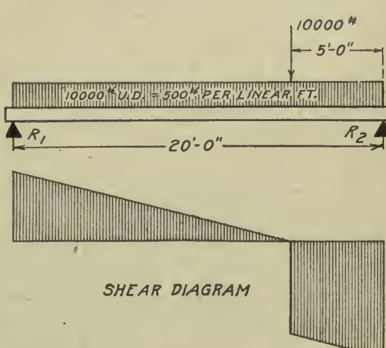


Fig. 82. Simple Beam with Uniformly Distributed Load over Entire Length and One Concentrated Load

Therefore $R_2 = \dfrac{250{,}000}{20} \qquad = 12{,}500\#$

$R_1 = 20{,}000 - 12{,}500 = \quad 7500\#$

Maximum shear is 12,500#. Zero shear occurs under the load P. Hence this is the point of maximum bending moment.

The bending moment computed on the right is

$$\begin{aligned} +5 \times 12{,}500 &= +62{,}500 \\ -2\tfrac{1}{2} \times 5 \times 500 &= -\ \ 6{,}250 \\ \hline & \quad\ \ 56{,}250 \text{ ft.-lb.} \end{aligned}$$

The bending moment computed on the left is

$$\begin{aligned} +15 \times 7{,}500 &= +112{,}500 \\ -\ 7\tfrac{1}{2} \times 15 \times 500 &= -\ \ 56{,}250 \\ \hline & \quad\ \ 56{,}250 \text{ ft.-lb.} \end{aligned}$$

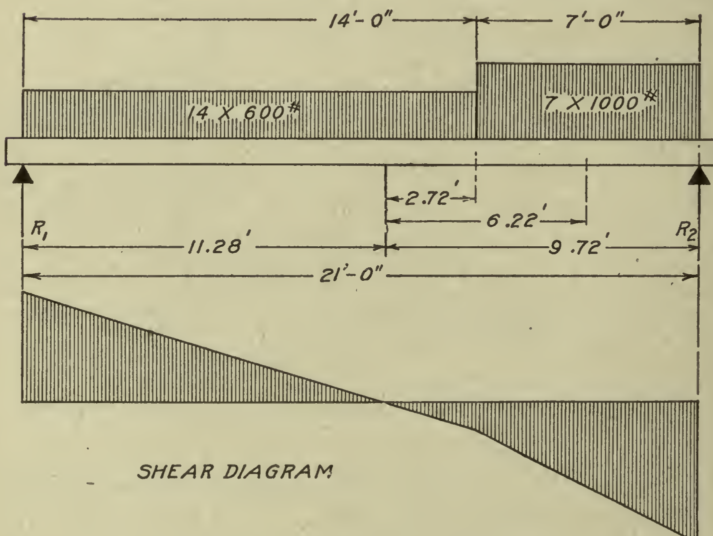


Fig. 83. Simple Beam with Two Rates of Uniformly Distributed Load

Two Uniformly Distributed Loads Not Overlapping. Fig. 83 illustrates a beam with one uniformly distributed load on part of its length and another load on the remainder, as shown. The total load on the beam is

$$\begin{aligned} 14 \times 600 &= 8400 \\ 7 \times 1000 &= 7000 \\ \hline \text{Total load} &= 15{,}400\# \end{aligned}$$

Moments about R_1 are

$$7 \times 8400 = 58{,}800$$
$$17\tfrac{1}{2} \times 7000 = \underline{122{,}500}$$
$$181{,}300 \text{ ft.-lb.}$$

Therefore $$R_2 = \frac{181{,}300}{21} = 8633\#$$

Moments about R_2 are

$$3\tfrac{1}{2} \times 7000 = 24{,}500$$
$$14 \times 8400 = \underline{117{,}600}$$
$$142{,}100 \text{ ft.-lb.}$$

Therefore $$R_1 = \frac{142{,}100}{21} = 6767\#$$

$$R_1 + R_2 = 6767 + 8633 = 15{,}400\#$$

Maximum shear is 8633. Zero shear occurs at a point $\frac{6767}{600}$ or 11.28 feet to the right of R_1. Hence this is the point of maximum bending moment.

The bending moment computed on the right is

$$\begin{array}{llr} +9.72 \times 8633 & = & +83{,}913 \\ -\dfrac{2.72}{2} \times 2.72 \times 600 = - \ 2220 & & \\ -6.22 \times 7000 & = \underline{-43{,}540} & \underline{-45{,}760} \\ & & +38{,}153 \text{ ft.-lb.} \end{array}$$

The bending moment computed on the left is

$$+11.28 \times 6767 = +76{,}332$$
$$-\frac{11.28}{2} \times 11.28 \times 600 = \underline{-38{,}166}$$
$$+38{,}166 \text{ ft.-lb.}$$

Two Distributed Loads and Concentrated Loads. Fig. 84 illustrates a beam with one uniformly distributed load for part of its length, another load for the remainder, and a concentrated load as shown. The total load on the beam is

u.d.	$12 \times 1{,}000$	$= 12{,}000$
u.d.	4×500	$= 2{,}000$
concentrated		$= 10{,}000$
Total load		$= 24{,}000\#$

Moments about R_1 are

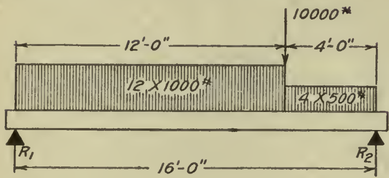


Fig. 84. Simple Beam with Mixed Loads

$$\begin{aligned} 6\times 12{,}000 &= 72{,}000 \\ 12\times 10{,}000 &= 120{,}000 \\ 14\times 2{,}000 &= \underline{28{,}000} \\ & \quad 220{,}000 \text{ ft.-lb.} \end{aligned}$$

Therefore

$$R_2 = \frac{220{,}000}{16} = 13{,}750\#$$

Moments about R_2 are

$$\begin{aligned} 2\times 2{,}000 &= 4{,}000 \\ 4\times 10{,}000 &= 40{,}000 \\ 10\times 12{,}000 &= \underline{120{,}000} \\ & \quad 164{,}000 \text{ ft.-lb.} \end{aligned}$$

Therefore $$R_1 = \frac{164{,}000}{16} = 10{,}250\#$$

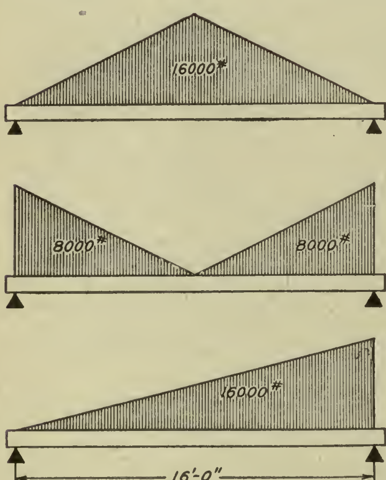


Fig. 85. Simple Beams with Variable Loads

Maximum shear is 13,750#. Zero shear occurs at 10.25 feet from R_1. Hence this is the point of maximum bending moment.

The bending moment computed on the left is

$$\begin{aligned} +10.25\times 10{,}250 &= +105{,}062 \\ -\ 5.125\times 10{,}250 &= -\ \underline{52{,}531} \\ & \quad 52{,}531 \text{ ft.-lb.} \end{aligned}$$

This value is to be checked by computing the bending moment on the right.

EXAMPLES FOR PRACTICE

1. Compute the bending moments for the loads illustrated in Fig. 85. Compare results with a uniformly distributed load.

2. A beam 20 feet long supports a load of 250 pounds on the first 5 feet, 400 pounds on the second 5 feet, and 350 pounds on the

remainder. What is the maximum shear? What is the position of the maximum bending moment?

3. What is the bending moment on an I-beam $15'' \times 42\# \times 30$ feet long, due to its own weight and to a load of 14,000 pounds concentrated at mid-span?

Typical Loadings. *Tabular Data.* When the shear and the bending moment can be expressed in simple formulas, it is easier to compute from the formulas than from the detailed calculations just illustrated. Table II has been compiled for this purpose. It gives the common arrangements of loading and the formulas for end reactions and maximum bending moment for each case.

Column 1 gives diagrams of the arrangement of the loading.

Columns 2 and 3 give the end reactions which, for all the cases given, are the same as the end shears. When the loading is symmetrical, the reaction is the same at both ends and is one-half the total load. When not symmetrical, the values differ at the two ends and both are given.

Column 4 gives the maximum bending moment.

Column 5 gives the distance in feet from the left support to the point of maximum bending moment.

The symbols used are:

W = total uniformly distributed or variable loads in pounds
P = single concentrated load in pounds
L = span in feet
L_1 = distance from support to center of gravity of load on cantilever beams
M = bending moment in foot-pounds
R_1 = reaction at left support
R_2 = reaction at right support
X = distance from left support to position of maximum bending moment

Simple Loads. When a load on a simple beam is symmetrically placed, whether uniformly distributed or concentrated, the reactions are equal, and the maximum bending moment is at the center of the span.

For a simple beam, irrespective of the manner of loading, the maximum bending moment and zero shear occur at the same point.

TABLE II

Reactions and Bending Moments for Typical Loadings

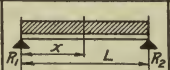
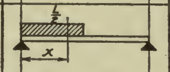
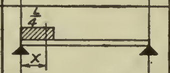
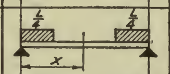
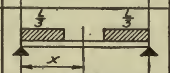
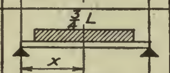
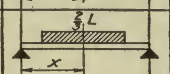
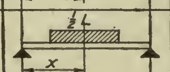
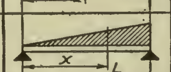
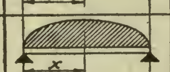
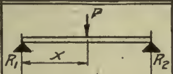
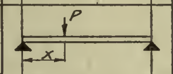
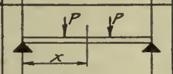
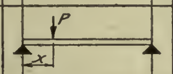
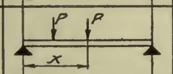
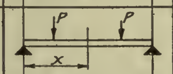
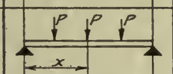
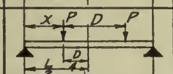
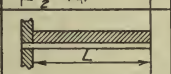
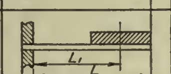
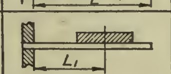
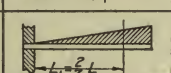
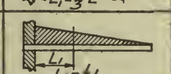
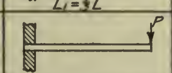
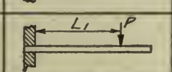
	FORM OF LOAD	R_1 Lbs.	R_2 Lbs.	MAXIMUM BENDING MOMENT Foot-Lbs.	x	REMARKS
1		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{1}{8}WL$	$\frac{1}{2}L$	
2		$\frac{3}{4}W$	$\frac{1}{4}W$	$\frac{9}{64}WL$	$\frac{3}{8}L$	
3		$\frac{5}{6}W$	$\frac{1}{6}W$	$\frac{25}{216}WL$	$\frac{5}{18}L$	
4		$\frac{7}{8}W$	$\frac{1}{8}W$	$\frac{49}{512}WL$	$\frac{7}{32}L$	
5		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{1}{16}WL$	$\frac{1}{4}L$ to $\frac{3}{4}L$	BENDING MOMENT CONSTANT OVER UNLOADED PART
6		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{1}{12}WL$	$\frac{1}{3}L$ to $\frac{2}{3}L$	DO
7		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{5}{32}WL$	$\frac{1}{2}L$	
8		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{1}{6}WL$	$\frac{1}{2}L$	
9		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{3}{16}WL$	$\frac{1}{2}L$	
10		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{5}{24}WL$	$\frac{1}{2}L$	
11		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{7}{32}WL$	$\frac{1}{2}L$	
12		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{1}{6}WL$	$\frac{1}{2}L$	
13		$\frac{1}{3}W$	$\frac{2}{3}W$	$0.128\,WL$	$0.57\,L$	
14		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{1}{12}WL$	$\frac{1}{2}L$	
15		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{3}{20}WL$	$\frac{1}{2}L$	THE CURVE IS A PARABOLA. THE B.M. IS APPROX. CORRECT FOR CIRCULAR SEGMENT

TABLE 11—(Continued)

Reactions and Bending Moments for Typical Loadings

	FORM OF LOAD	R_1	R_2	MAXIMUM BENDING MOMENT	x	REMARKS
		Lbs.	Lbs.	Foot-Lbs.		
16		$\frac{1}{2}P$	$\frac{1}{2}P$	$\frac{1}{4}PL$	$\frac{1}{2}L$	POSITION OF ONE CONCENTRATED LOAD FOR MAXIMUM BENDING MOMENT
17		$\frac{2}{3}P$	$\frac{1}{3}P$	$\frac{2}{9}PL$	$\frac{1}{3}L$	
18		P	P	$\frac{1}{3}PL$	$\frac{1}{3}L$ to $\frac{2}{3}L$	BENDING MOMENT CONSTANT BETWEEN LOADS
19		$\frac{3}{4}P$	$\frac{1}{4}P$	$\frac{3}{16}PL$	$\frac{1}{4}L$	
20		$\frac{5}{4}P$	$\frac{3}{4}P$	$\frac{3}{8}PL$	$\frac{1}{2}L$	
21		P	P	$\frac{1}{4}PL$	$\frac{1}{4}L$ to $\frac{3}{4}L$	BENDING MOMENT CONSTANT BETWEEN LOADS
22		$\frac{3}{2}P$	$\frac{3}{2}P$	$\frac{1}{2}PL$	$\frac{1}{2}L$	
23		$P\left(1-\frac{1}{2}\frac{D}{L}\right)$	$P\left(1+\frac{1}{2}\frac{D}{L}\right)$	$\frac{P}{2}\left(L-D+\frac{1}{4}\frac{D^2}{L}\right)$	$\frac{L}{2}-\frac{D}{4}$	POSITION OF TWO CONCENTRATED LOADS FOR MAXIMUM BENDING MOMENT
24		W		$\frac{WL}{2}$	0	
25		W		WL_1	0	
26		W		WL_1	0	
27		W		$\frac{2}{3}WL$	0	
28		W		$\frac{1}{3}WL$	0	
29		P		$P.L$	0	
30		P		PL_1	0	

The point of zero shear is important only as the easiest means of locating the place of maximum bending moment.

For a cantilever beam, irrespective of the manner of loading, the maximum bending moment and maximum shear occur at the support.

Illustrative Examples. To illustrate the use of Table II, assume a beam 18 feet long to be loaded from the left support to the middle at 320 pounds per lineal foot.

$$W = 9 \times 320 = 2880\#$$
$$V_1 = R_1 = \tfrac{3}{4}W = \tfrac{3}{4} \times 2880 = 2160\#$$
$$V_2 = R_2 = \tfrac{1}{4}W = \tfrac{1}{4} \times 2880 = 720\#$$
$$M = \tfrac{9}{64}WL = \tfrac{9}{64} \times 2880 \times 18 = 7290 \text{ ft.-lb.}$$

Moving Loads. It is sometimes necessary to know what position of a *moving load* will produce the maximum bending moment in a beam. If it is a single concentrated load, the maximum occurs when the load is at the center of the span, as in item 16. Compare items 16, 17, and 19. If there are two concentrated loads, as the wheels of a traveling crane, the position producing the maximum is shown in item 23. As there indicated, one load is $\frac{1}{4}$ D distant on one side of the center of the span and the other is $\frac{3}{4}$ D distant on the other side. The maximum bending moment is at the load nearer to the center.

Illustrative Example. Assume two crane wheels spaced 8 feet centers, each loaded with 10,000 pounds, span 20 feet, to find maximum bending moment. From the formulas

$$R_1 = 8{,}000\# \text{ and } R_2 = 12{,}000\#; \quad X = 8 \text{ ft.}$$

$$\text{Max. } M = 8 \times 8000 = 64{,}000 \text{ ft.-lb.}$$

Beam with Two or More Loadings. A beam may have two or more of the loadings illustrated. The respective reactions for the combined loads are the sums of the corresponding reactions for the separate loadings. This applies in all cases. The maximum bending moment for the combined loads is the sum of the moments for the separate loadings, provided the *positions of the maximums for the separate loadings are the same.* Generally this condition occurs only when all the loads are symmetrical about the center of the span, or for cantilever beams.

EXAMPLES FOR PRACTICE

1. What is the bending moment of a concentrated load of 89,000 pounds at the center of a span 21′-6″ long?

2. What are the shear and bending moment of a load of 21,000 pounds at the quarter point of a span 19 feet long?

3. A beam is loaded at 750 pounds per lineal foot on the two end-thirds. What is the bending moment?

4. A beam carries a uniformly distributed load of 18,000 pounds and a center load of 9000 pounds. Span 16 feet. What are the reactions and maximum bending moment?

5. A crane girder has a span of 20 feet. The wheel load is 30,000 pounds. The wheel base is 10 feet. What is the position of loads for maximum bending moment? What is the amount of the maximum bending moment?

CALCULATION OF RESISTANCE

Factors Considered. Having determined the shear and bending moment to which a beam is subjected, the next step, logically, is to determine the dimensions of the section which will resist them. The *resistance to bending* is first provided for, as this usually governs in the design of the rolled beam section. Then the *shearing resistance* is compared with the shearing stress to make sure that it is sufficient. To investigate the *resisting moment* in complete detail would require the following operations:

(1) Assume maximum unit stress on extreme fiber
(2) Assume section of beam, and compute its moment of inertia
(3) From these values compute the resisting moment of the assumed section
(4) Compare this resisting moment with the bending moment
(5) Repeat the operation until a resisting moment is found which equals or slightly exceeds the bending moment

This procedure, with some additional steps, is followed in the case of riveted beams, but for rolled beams the tables in the handbooks and elsewhere give resisting moments and various other properties of the sections so that the operations are much simplified.

Resisting Moment. The resisting moment of any beam is determined from the formula

$$M = S\frac{I}{c}$$

as stated on p. 78 and demonstrated under Resisting Moment in "Strength of Materials" Part I. This formula may be changed to the form

$$\frac{I}{c}=\frac{M}{S}$$

which stated in words is

$$\frac{\textit{moment of inertia}}{\textit{one-half the depth}}=\frac{\textit{resisting moment}}{\textit{unit stress}}$$

Section Modulus. In the expression just given $\frac{I}{c}$ is called the "section modulus," (p. 39). Its values for I-beams, channels, and angles are given in the handbook. Since the resisting moment must be equal to or greater than the bending moment and, since the value of the unit stress has been established, the value of the section modulus can be computed and the section selected from the tables. For example, the allowable unit stress in bending on the extreme fiber is 16,000 pounds per square inch; assume a beam subjected to a bending moment of 100,000 foot-pounds; since the section modulus is in terms of inches, the bending moment must be expressed in inch-pounds and for this case becomes 1,200,000 inch-pounds; then the section modulus required is

$$\frac{I}{c}=\frac{M}{S}=\frac{1{,}200{,}000}{16{,}000}=75.0$$

Referring to the tables for I-beams it is found that the section having the nearest higher value of the section modulus is

15″ I 60#

Expressed in simple words the operations are:

(1) Multiply the bending moment of the beam by 12 to reduce it to inch-pounds.
(2) Divide this by 16,000 to determine the required section modulus.
(3) From the tables select a section whose section modulus is equal to or greater than the required value.

Tabular Values for Resisting Moments. For a given unit stress each section has a definite resisting moment which is computed from the formula

$$M=S\,\frac{I}{c}$$

The values of the resisting moment are not given in all of the handbooks. They are given in Table III, based on a unit stress of 16,000 pounds per square inch, and expressed in *foot-pounds*. This shortens the operation of selecting a section, it being necessary only to choose a section whose resisting moment is equal to or greater than the bending moment produced by the load on the beam.

For example, assume a bending moment of 30,625 foot-pounds. Referring to Table III, the beam having the nearest higher resisting moment is 10″ I 25#, whose resisting moment is 32,500 foot-pounds.

If the load on the beam is uniformly distributed, the computations may be still further shortened by means of tables given in the handbooks. These tables give the safe loads uniformly distributed for various lengths of spans. The Carnegie handbook has formerly given these values for I-beams, channels, angles, tees, and zees in *tons* but in the 1913 edition they are given in *thousands of pounds*. The Cambria handbook gives the values for I-beams and channels only and expresses them in *pounds*. For example, a beam 20 feet long supports a load of 700 pounds per lineal foot. The total load is $20 \times 700 = 14{,}000\#$. From the tables the size of beam is found to be 10″ I 30#.

EXAMPLES FOR PRACTICE

1. Two angles are required to support a load of 4200 pounds uniformly distributed on a span of 6 feet. Determine the section. by means of the section modulus.

2. A channel having a span 12′-6″ long is required to support a concentrated load of 17,900 pounds at the middle point. What section is required?

3. Determine the sizes of beams required for the conditions given in the problems on p. 97. Use the simplest of the three methods given above, and check the results by one of the other methods.

Application of Tables to Concentrated Loads. By careful study of the moment factors given in Table II, the designer can adapt the tables in the handbooks for uniformly distributed loads to other forms of loading. Thus a concentrated load at the center of a span produces the same bending moment as a uniformly distributed load of twice the amount; then to use the table select a beam whose capacity is twice the amount of the concentrated load.

TABLE III

Strength of Beams

I-Beams; H-Sections; Channels; Angles; and Tees

SECTION	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Shearing Resistance of Web at 10,000 Lb. per Sq. Inch	Strength of Standard End Connections American Bridge Co., 1911	Extreme Length for Deflection for Plastered Ceilings Limit 1-360 Span: For Uniformly Distributed Load	Extreme Length for Deflection for Plastered Ceilings Limit 1-360 Span: For Center Load	Extreme Length for Beams without Lateral Support: When Loaded to Full Capacity	Extreme Length for Beams without Lateral Support: When Loaded to Half Capacity
	$(In.)^4$	$(In.)^3$	Ft.-Lb.	Pounds	Pounds	Ft. In.	Ft. In.	Ft. In.	Ft. In.
27″ I 83#	2888.6	214.0	285,300	114,500		54–0	36–0	12– 6	37–6
24″ I 115#	2955.5	246.3	328,400	180,000	53,000	48–0	32–0	13– 4	40–0
110	2883.5	240.3	320,400	165,100	"	"	"	13– 3	39–0
105	2811.5	234.3	312,400	150,000	"	"	"	13– 1	39–4
100	2380.3	198.4	264,500	181,000	"	"	"	12– 1	36–3
95	2309.6	192.5	256,700	166,100	"	"	"	12– 0	36–0
90	2239.1	186.5	248,800	151,400	"	"	"	11–11	35–8
85	2168.6	180.7	240,900	136,800	"	"	"	11– 9	35–4
80	2087.9	174.0	232,000	120,000	"	"	"	11– 8	35–0
69½	1928.0	160.7	214,300	93,600	43,900	"	"	11– 8	35–0
21″ I 57½#	1227.5	116.9	155,900	75,000	33,400	42–0	28–0	10–10	32–6
20″ I 100#	1655.8	165.6	220,800	176,800	44,200	40–0	26–8	12– 2	36–5
95	1606.8	160.7	214,300	162,000	"	"	"	12– 0	36–1
90	1557.8	158.5	207,700	147,400	"	"	"	11–11	35–8
85	1508.7	150.9	201,200	132,600	"	"	"	11– 9	35–4
80	1466.5	146.7	195,600	120,000	"	"	"	11– 8	35–0
75	1268.9	126.9	169,200	129,800	"	"	"	10– 8	32–0
70	1219.9	122.0	162,700	115,000	"	"	"	10– 7	31–8
65	1169.6	117.0	156,000	100,000	"	"	"	10– 5	31–3
18″ I 90#	1260.4	140.0	186,700	145,300	43,100	36–0	24–0	12– 1	36– 3
85	1220.7	135.6	180,800	130,500	"	"	"	11–11	35–10
80	1181.0	131.2	175,000	115,900	"	"	"	11–10	35– 5
75	1141.3	126.8	169,100	101,200	"	"	"	11– 8	35– 0
70	921.3	102.4	136,500	129,400	"	"	"	10– 5	31– 4
65	881.5	97.9	130,500	114,700	"	"	"	10– 4	30–11
60	841.8	93.5	124,700	99,900	"	"	"	10– 2	30– 6
55	795.6	88.4	117,900	82,800	"	"	"	10– 0	30– 0
46	733.2	81.5	108,700	58,000	30,200	"	"	11– 8	35– 0
15″ I 100#	900.5	120.1	160,100	177,600	35,400	30–0	20–0	11– 3	33–10
95	872.9	116.4	155,200	162,800	"	"	"	11– 1	33– 4
90	845.4	112.7	150,300	148,000	"	"	"	11– 0	32–11
85	817.8	109.0	145,300	133,400	"	"	"	10– 9	32– 4
80	795.5	106.1	141,500	121,500	"	"	"	10– 8	32– 0
75	691.2	92.2	122,900	132,300	"	"	"	10– 6	31– 5
70	663.6	88.5	118,000	117,600	"	"	"	10– 4	31– 0
65	636.0	84.8	113,100	102,900	"	"	"	10– 2	30– 6
60	609.0	81.2	108,300	88,500	"	"	"	10– 0	30– 0
55	511.0	68.1	90,800	98,400	"	"	"	9– 7	28– 9
50	483.4	64.5	86,000	83,700	"	"	"	9– 5	28– 3
45	455.8	60.8	81,100	69,000	"	"	"	9– 3	27– 9
42	441.7	58.9	78,500	61,500	"	"	"	9– 2	27– 6
36	405.1	54.1	72,000	43,400	32,500	"	"	9– 2	27– 6

TABLE III (Continued)

Strength of Beams

I-Beams; H-Sections; Channels; Angles; and Tees

SECTION	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Shearing Resistance of Web at 10,000 Lb. per Sq. Inch	Strength of Standard End Connections American Bridge Co., 1911	Extreme Length for Deflection for Plastered Ceilings Limit 1–360 Span: For Uniformly Distributed Load	Extreme Length for Deflection for Plastered Ceilings Limit 1–360 Span: For Center Load	Extreme Length for Beams without Lateral Support: When Loaded to Full Capacity	Extreme Length for Beams without Lateral Support: When Loaded to Half Capacity
	(In.)^4	(In.)^3	Ft.-Lb.	Pounds	Pounds	Ft. In.	Ft. In.	Ft. In.	Ft. In.
12″ I 55#	321.0	53.5	71,300	98,600	26,500	24–0	16–0	9– 4	28– 1
50	303.3	50.6	67,500	83,900	"	"	"	9– 2	27– 5
45	285.7	47.6	63,500	69,100	"	"	"	8–11	26–10
40	268.9	44.8	59,700	55,200	"	"	"	8– 9	26– 3
35	228.3	38.0	50,700	52,300	"	"	"	8– 6	25– 5
31½	215.8	36.0	48,000	42,000	"	"	"	8– 4	25– 0
27½	199.6	33.3	44,400	38,200	23,900	"	"	8 –4	25– 0
10″ I 40#	158.7	31.7	42,300	74,900	17,700	20–0	13–4	8–6	25–6
35	146.4	29.3	39,100	60,200	"	"	"	8–3	24–9
30	134.2	26.8	35,700	45,500	"	"	"	8–0	24–0
25	122.1	24.4	32,500	31,000	"	"	"	7–9	23–4
22	113.9	22.8	30,400	23,200	17,400	"	"	7–9	23–4
9″ I 35#	111.8	24.8	33,100	65,900	17,700	18–0	12–0	7–11	23–10
30	101.9	22.6	30,100	51,200	"	"	"	7– 8	23– 0
25	91.9	20.4	27,200	36,500	"	"	"	7– 5	22– 3
21	84.9	18.9	25,200	26,100	"	"	"	7– 3	21– 8
8″ I 25¼#	68.4	17.1	22,800	43,300	17,700	16–0	10–8	7– 1	21– 4
23	64.5	16.1	21,400	35,900	"	"	"	7– 0	20–11
20½	60.6	15.1	20,100	28,600	"	"	"	6–10	20– 5
18	56.9	14.2	18,900	21,600	"	"	"	6– 8	20– 0
17½	58.3	14.6	19,500	16,800	15,800	"	"	7– 3	21– 8
7″ I 20#	42.2	12.1	16,100	32,100	17,700	14–0	9–4	6–5	19– 4
17½	39.2	11.2	14,900	24,700	"	"	"	6–3	18–10
15	36.2	10.4	13,900	17,500	"	"	"	6–1	18– 4
6″ I 17¼#	26.2	8.7	11,600	28,500	8,800	12–0	8–0	7–2	21–5
14¾	24.0	8.0	10,700	21,100	"	"	"	5–9	17–3
12¼	21.8	7.3	9,700	13,800	8,600	"	"	5–7	16–8
5″ I 14¾#	15.2	6.1	8,100	25,200	8,800	10–0	6–8	5–6	16–6
12¼	13.6	5.4	7,200	17,800	"	"	"	5–3	15–9
9¾	12.1	4.8	6,400	10,500	7,900	"	"	5–0	15–0
4″ I 10½#	7.1	3.6	4,800	16,400	8,800	8–0	5–4	4–10	14–5
9½	6.7	3.4	4,500	13,500	"	"	"	4– 8	14–0
8½	6.4	3.2	4,300	10,500	"	"	"	4– 7	13–8
7½	6.0	3.0	4,000	7,600	7,100	"	"	4– 5	13–4
3″ I 7½#	2.9	1.9	2,500	10,800	8,800	6–0	4–0	4– 2	12–7
6½	2.7	1.8	2,400	7,900	"	"	"	4– 0	12–1
5½	2.5	1.7	2,270	5,100	6,400	"	"	3–11	11–8
H-8″-34.0#	115.4	28.9	38,500	30,000	—	13–4	10–8	—	—
6″-23.8	45.1	15.0	20,000	18,800	—	12–0	8–0	—	—
5″-18.7	23.8	9.5	12,700	15,600	—	10–0	6–8	—	—
4″-13.6	10.7	5.3	7,100	12,500	—	8–0	5–4	—	—

TABLE III (Continued)

Strength of Beams

I-Beams; H-Sections; Channels; Angles; and Tees

SECTION	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Shearing Resistance of Web at 10,000 Lb. per Sq. Inch	Strength of Standard End Connections American Bridge Co., 1911	Extreme Length for Deflection for Plastered Ceilings Limit 1-360 Span: For Uniformly Distributed Load	Extreme Length for Deflection for Plastered Ceilings Limit 1-360 Span: For Center Load	Extreme Length for Beams without Lateral Support: When Loaded to Full Capacity	Extreme Length for Beams without Lateral Support: When Loaded to Half Capacity
	$(In.)^4$	$(In.)^3$	Ft.-Lb.	Pounds	Pounds	Ft. In.	Ft. In.	Ft. In.	Ft. In.
15″ [55#	430.2	57.4	76,500	122,700	35,400	30–0	20–0	6– 4	19–1
40	402.7	53.7	71,600	108,000	"	"	"	6– 2	18–6
45	375.1	50.0	66,700	93,300	"	"	"	6– 0	18–1
40	347.5	46.3	61,700	78,600	"	"	"	5–11	17–7
35	320.0	42.7	56,900	63,900	"	"	"	5– 9	17–2
33	312.6	41.7	55,600	60,000	"	"	"	5– 8	17–0
12″ [40#	197.0	32.8	43,700	91,000	26,500	24–0	16–0	5– 8	17– 1
35	179.3	29.9	39,900	76,300	"	"	"	5– 6	16– 6
30	161.7	26.9	35,900	61,600	"	"	"	5– 3	15–10
25	144.0	24.0	32,000	46,800	"	"	"	5– 1	15– 3
20½	128.1	21.4	28,500	33,600	26,200	"	"	4–11	14– 8
10″ [35#	115.5	23.1	30,800	82,300	17,700	20–0	13–4	5– 4	15–11
30	103.2	20.6	27,500	67,600	"	"	"	5– 1	15– 2
25	91.0	18.2	24,300	52,900	"	"	"	4–10	14– 5
20	78.7	15.7	20,900	38,200	"	"	"	4– 7	13– 9
15	66.9	13.4	17,900	24,000	"	"	"	4– 4	13– 0
9″ [25#	70.7	15.7	20,900	55,400	17,700	18–0	12–0	4–8	14–1
20	60.8	13.5	18,000	40,700	"	"	"	4–5	13–3
15	50.9	11.3	15,100	25,900	"	"	"	4–2	12–5
13¼	47.3	10.5	14,000	20,700	17,200	"	"	4–1	12–2
8″ [21¼#	47.8	11.9	15,900	46,600	17,700	16–0	10–8	4– 4	13–1
18¾	43.8	11.0	14,700	39,200	"	"	"	4– 3	12–8
16¼	39.9	10.0	13,300	31,900	"	"	"	4– 1	12–2
13¾	36.0	9.0	12,000	24,600	"	"	"	3–11	11–9
11¼	32.3	8.1	10,800	17,600	16,500	"	"	3– 9	11–4
7″ [19¾#	33.2	9.5	12,700	44,300	17,700	14–0	9–4	4– 2	12–7
18¾	30.2	8.6	11,500	37,000	"	"	"	4– 0	12–0
14¾	27.2	7.8	10,400	29,600	"	"	"	3–10	11–6
12¼	24.2	6.9	9,200	22,300	"	"	"	3– 8	11–0
9¾	21.1	6.0	8,000	14,700	15,800	"	"	3– 6	10–5
6″ [15½#	19.5	6.5	8,700	33,800	8,800	12–0	8–0	3–10	11– 5
13	17.3	5.8	7,700	26,400	"	"	"	3– 7	10–10
10½	15.1	5.0	6,700	19,100	"	"	"	3– 5	10– 2
8	13.0	4.3	5,700	12,000	7,500	"	"	3– 2	9– 7
5″ [11½#	10.4	4.2	5,600	23,800	8,800	10–0	6–8	3– 5	10–2
9	8.9	3.5	4,700	16,500	"	"	"	3– 2	9–5
6½	7.4	3.0	4,000	9,500	7,100	"	"	2–11	8–9
4″ [7¼#	4.6	2.3	3,100	12,000	8,800	8–0	5–4	2–10	8– 7
6¼	4.2	2.1	2,800	10,100	"	"	"	2– 9	8– 3
5¼	3.8	1.9	2,500	7,200	6,800	"	"	2– 8	7–11
3″ [6#	2.1	1.4	1,870	10,900	8,800	6–0	4–0	2–8	8–0
5	1.8	1.2	1,600	7,900	"	"	"	2–6	7–6
4	1.6	1.1	41,70	5,100	6,400	"	"	2–4	7–1

TABLE III (Continued)

Strength of Beams

I-Beams; H-Sections; Channels; Angles; and Tees

SECTION	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Extreme length for Deflection for Plastered Ceilings, Limit 1–480 Span: For Uniformly Distributed Load	Extreme length for Deflection for Plastered Ceilings, Limit 1–480 Span: For Center Load
	$(In.)^4$	$(In.)^3$	Ft.-Lb.	Ft. In.	Ft. In.
L-8x8x$1\frac{1}{8}$	97.97	17.53	23,400	17–0	11–0
$1\frac{1}{16}$	93.53	16.67	22,200	"	"
1	88.98	15.80	21,100	"	"
$\frac{15}{16}$	84.33	14.91	19,900	"	"
$\frac{7}{8}$	79.58	14.01	18,700	"	"
$\frac{13}{16}$	74.71	13.11	17,500	"	"
$\frac{3}{4}$	69.74	12.18	16,200	"	"
$\frac{11}{16}$	64.64	11.25	15,000	"	"
$\frac{5}{8}$	59.42	10.30	13,700	"	"
$\frac{9}{16}$	54.09	9.34	12,500	"	"
$\frac{1}{2}$	48.63	8.37	11,200	"	"
L-6x6x1	35.46	8.57	11,400	12–6	8–6
$\frac{15}{16}$	33.72	8.11	10,800	"	"
$\frac{7}{8}$	31.92	7.64	10,200	"	"
$\frac{13}{16}$	30.06	7.15	9,500	"	"
$\frac{3}{4}$	28.15	6.66	8,900	"	"
$\frac{11}{16}$	26.19	6.17	8,200	"	"
$\frac{5}{8}$	24.16	5.66	7,500	"	"
$\frac{9}{16}$	22.07	5.14	6,800	"	"
$\frac{1}{2}$	19.91	4.61	6,100	"	"
$\frac{7}{16}$	17.68	4.07	5,400	"	"
$\frac{3}{8}$	15.39	3.53	4,700	"	"
L-5x5x1	19.64	5.80	7,700	10–6	7–0
$\frac{15}{16}$	18.71	5.49	7,300	"	"
$\frac{7}{8}$	17.75	4.17	6,900	"	"
$\frac{13}{16}$	16.77	4.85	6,500	"	"
$\frac{3}{4}$	15.74	4.53	6,000	"	"
$\frac{11}{16}$	14.68	4.20	5,600	"	"
$\frac{5}{8}$	13.58	3.86	5,100	"	"
$\frac{9}{16}$	12.44	3.51	4,700	"	"
$\frac{1}{2}$	11.25	3.15	4,200	"	"
$\frac{7}{16}$	10.02	2.79	3,700	"	"
$\frac{3}{8}$	8.74	2.42	3,200	"	"
L-4x4x $\frac{7}{8}$	8.59	3.20	4,300	8–3	5–6
$\frac{13}{16}$	8.14	3.01	4,000	"	"
$\frac{3}{4}$	7.67	2.81	3,700	"	"
$\frac{11}{16}$	7.17	2.61	3,500	"	"
$\frac{5}{8}$	6.66	2.40	3,200	"	"
$\frac{9}{16}$	6.12	2.19	2,900	"	"
$\frac{1}{2}$	5.56	1.97	2,600	"	"
$\frac{7}{16}$	4.97	1.75	2,300	"	"
$\frac{3}{8}$	4.36	1.52	2,000	"	"
$\frac{5}{16}$	3.71	1.29	1,700	"	"
L-$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{7}{8}$	5.53	2.39	3,200	7–0	4–9
$\frac{13}{16}$	5.25	2.25	3,000	"	"
$\frac{3}{4}$	4.96	2.11	2,800	"	"
$\frac{11}{16}$	4.65	1.96	2,600	"	"
$\frac{5}{8}$	4.33	1.81	2,400	"	"

SECTION	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Extreme length for Deflection for Plastered Ceilings, Limit 1–480 Span: For Uniformly Distributed Load	Extreme length for Deflection for Plastered Ceilings, Limit 1–480 Span: For Center Load
	$(In.)^4$	$(In.)^3$	Ft.-Lb.	Ft. In.	Ft. In.
L-$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{9}{16}$	3.99	1.65	2,200	7–0	4–9
$\frac{1}{2}$	3.64	1.49	1,990	"	"
$\frac{7}{16}$	3.26	1.32	1,760	"	"
$\frac{3}{8}$	2.87	1.15	1,530	"	"
$\frac{5}{16}$	2.45	0.98	1,310	"	"
L-3x3x $\frac{11}{16}$	2.81	1.40	1,870	6–0	4–0
$\frac{5}{8}$	2.62	1.30	1,730	"	"
$\frac{9}{16}$	2.43	1.19	1,590	"	"
$\frac{1}{2}$	2.22	1.07	1,430	"	"
$\frac{7}{16}$	1.99	0.95	1,270	"	"
$\frac{3}{8}$	1.76	0.83	1,110	"	"
$\frac{5}{16}$	1.51	0.71	950	"	"
$\frac{1}{4}$	1.24	0.58	770	"	"
L-$2\frac{3}{4}$x$2\frac{3}{4}$x$\frac{1}{2}$	1.67	0.89	1,190	5–9	3–9
$\frac{7}{16}$	1.51	0.79	1,050	"	"
$\frac{3}{8}$	1.33	0.69	920	"	"
$\frac{5}{16}$	1.15	0.59	790	"	"
$\frac{1}{4}$	0.93	0.48	640	"	"
L-$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{9}{16}$	1.34	0.80	1,070	5–0	3–4
$\frac{1}{2}$	1.23	0.73	970	"	"
$\frac{7}{16}$	1.11	0.65	870	"	"
$\frac{3}{8}$	0.98	0.57	760	"	"
$\frac{5}{16}$	0.85	0.48	640	"	"
$\frac{1}{4}$	0.70	0.40	530	"	"
$\frac{3}{16}$	0.55	0.30	400	"	"
L-$2\frac{1}{4}$x$2\frac{1}{4}$x$\frac{1}{2}$	0.87	0.58	770	4–6	3–0
$\frac{7}{16}$	0.79	0.52	690	"	"
$\frac{3}{8}$	0.70	0.45	600	"	"
$\frac{5}{16}$	0.61	0.39	520	"	"
$\frac{1}{4}$	0.51	0.32	430	"	"
$\frac{3}{16}$	0.39	0.24	320	"	"
L-2x2x$\frac{1}{2}$	0.59	0.45	600	4–0	2–9
$\frac{7}{16}$	0.54	0.40	530	"	"
$\frac{3}{8}$	0.48	0.35	470	"	"
$\frac{5}{16}$	0.42	0.30	400	"	"
$\frac{1}{4}$	0.35	0.25	330	"	"
$\frac{3}{16}$	0.28	0.19	250	"	"

TABLE III (Continued)

Strength of Beams

I-Beams; H-Sections; Channels; Angles; and Tees

SECTION	LONG LEG VERTICAL: Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Extreme Length for Deflection for Plastered Ceilings Limit 1-360 Span: For Uniformly Distributed Load	For Center Load	SHORT LEG VERTICAL: Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Extreme Length for Deflection for Plastered Ceilings Limit 1-360 Span: For Uniformly Distributed Load	For Center Load
	(In.)^4	(In.)^3	Ft.-Lb.	Ft. In.	Ft. In.	(In.)^4	(In.)^3	Ft.-Lb.	Ft. In.	Ft. In.
L-8x6x1	80.78	15.11	20,100	16–3	10–9	38.78	8.92	11,900	13–3	8–9
$\frac{15}{16}$	76.59	14.27	19,000	"	"	36.85	8.43	11,200	"	"
$\frac{7}{8}$	72.32	13.41	17,900	"	"	34.86	7.94	10,600	"	"
$\frac{13}{16}$	67.92	12.55	16,700	"	"	32.82	7.44	9,900	"	"
$\frac{3}{4}$	63.42	11.67	15,600	"	"	30.72	6.92	9,200	"	"
$\frac{11}{16}$	58.82	10.77	14,400	"	"	28.56	6.40	8,500	"	"
$\frac{5}{8}$	54.10	9.87	13,200	"	"	26.33	5.88	7,800	"	"
$\frac{9}{16}$	49.26	8.95	11,900	"	"	24.04	5.34	7,100	"	"
$\frac{1}{2}$	44.31	8.02	10,700	"	"	21.68	4.79	6,400	"	"
L-7x3$\frac{1}{2}$x1	45.37	10.58	14,100	12–3	8–9	7.53	2.96	3,900	7–9	5–3
$\frac{15}{16}$	43.13	10.00	13,300	"	"	7.18	2.80	3,700	"	"
$\frac{7}{8}$	40.82	9.42	12,600	"	"	6.83	2.64	3,500	"	"
$\frac{13}{16}$	38.45	8.82	11,800	"	"	6.46	2.48	3,300	"	"
$\frac{3}{4}$	35.99	8.22	11,000	"	"	6.08	2.31	3,100	"	"
$\frac{11}{16}$	33.47	7.60	10,100	"	"	5.69	2.14	2,900	"	"
$\frac{5}{8}$	30.86	6.97	9,300	"	"	5.28	1.97	2,600	"	"
$\frac{9}{16}$	28.18	6.33	8,400	"	"	4.86	1.80	2,400	"	"
$\frac{1}{2}$	25.41	5.68	7,600	"	"	4.41	1.62	2,200	"	"
$\frac{7}{16}$	22.56	5.01	6,700	"	"	3.95	1.47	1,960	"	"
L-6x4x1	30.75	8.02	10,700	11–9	7–9	10.75	3.79	5,000	8–6	5–9
$\frac{15}{16}$	29.26	7.59	10,100	"	"	10.26	3.59	4,800	"	"
$\frac{7}{8}$	27.73	7.15	9,500	"	"	9.75	3.39	4,500	"	"
$\frac{13}{16}$	26.15	6.70	8,900	"	"	9.23	3.18	4,200	"	"
$\frac{3}{4}$	24.51	6.25	8,300	"	"	8.68	2.97	4,000	"	"
$\frac{11}{16}$	22.82	5.78	7,700	"	"	8.11	2.76	3,700	"	"
$\frac{5}{8}$	21.07	5.31	7,100	"	"	7.52	2.54	3,400	"	"
$\frac{9}{16}$	19.26	4.83	6,400	"	"	6.91	2.31	3,100	"	"
$\frac{1}{2}$	17.40	4.33	5,800	"	"	6.27	2.08	2,800	"	"
$\frac{7}{16}$	15.46	3.83	5,100	"	"	5.60	1.85	2,500	"	"
$\frac{3}{8}$	13.47	3.32	4,400	"	"	4.90	1.60	2,100	"	"
L-6x3$\frac{1}{2}$x1	29.24	7.83	10,400	11–8	7–9	7.21	2.90	3,900	7–9	5–3
$\frac{15}{16}$	27.84	7.41	9,900	"	"	6.88	2.74	3,700	"	"
$\frac{7}{8}$	26.38	6.98	9,300	"	"	6.55	2.59	3,500	"	"
$\frac{13}{16}$	24.89	6.55	8,700	"	"	6.20	2.43	3,200	"	"
$\frac{3}{4}$	23.34	6.10	8,100	"	"	5.84	2.27	3,000	"	"
$\frac{11}{16}$	21.74	5.65	7,500	"	"	5.47	2.11	2,800	"	"
$\frac{5}{8}$	20.08	5.19	6,900	"	"	5.08	1.94	2,600	"	"
$\frac{9}{16}$	18.37	4.72	6,300	"	"	4.67	1.77	2,400	"	"
$\frac{1}{2}$	16.59	4.24	5,700	"	"	4.25	1.59	2,100	"	"
$\frac{7}{16}$	14.76	3.75	5,000	"	"	3.81	1.41	1,880	"	"
$\frac{3}{8}$	12.86	3.25	4,300	"	"	3.34	1.23	1,640	"	"
L-5x4x $\frac{7}{8}$	16.42	4.99	6,700	10–0	6–8	9.23	3.31	4,500	8–6	5–9
$\frac{13}{16}$	15.54	4.69	6,300	"	"	8.74	3.11	4,100	"	"
$\frac{3}{4}$	14.60	4.37	5,800	"	"	8.23	2.90	3,900	"	"
$\frac{11}{16}$	13.62	4.05	5,400	"	"	7.70	2.69	3,600	"	"
$\frac{5}{8}$	12.61	3.73	5,000	"	"	7.14	2.48	3,300	"	"

TABLE III (Continued)

Strength of Beams

I-Beams; H-Sections; Channels; Angles; and Tees

SECTION	LONG LEG VERTICAL					SHORT LEG VERTICAL				
	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Extreme Length for Deflection for Plastered Ceilings Limit 1-480 Span: For Uniformly Distributed Load	Extreme Length for Deflection for Plastered Ceilings Limit 1-480 Span: For Center Load	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Extreme Length for Deflection for Plastered Ceilings Limit 1-480 Span: For Uniformly Distributed Load	Extreme Length for Deflection for Plastered Ceilings Limit 1-480 Span: For Center Load
	(In.)^4	(In.)^3	Ft.-Lb.	Ft. In.	Ft. In.	(In.)^4	(In.)^3	Ft.-Lb.	Ft. In.	Ft. In.
L-5x4x $\frac{9}{16}$	11.55	3.39	4,500	10–0	6–8	6.56	2.26	3,100	8–6	5–9
$\frac{1}{2}$	10.46	3.05	4,100	″	″	5.96	2.04	2,700	″	″
$\frac{7}{16}$	9.32	2.70	3,600	″	″	5.32	1.81	2,400	″	″
$\frac{3}{8}$	8.14	2.34	3,100	″	″	4.67	1.57	2,100	″	″
L-5x3½x $\frac{7}{8}$	15.67	4.88	6,500	9–9	6–6	6.21	2.52	3,400	7–6	5–0
$\frac{13}{16}$	14.81	4.58	6,100	″	″	5.89	2.37	3,200	″	″
$\frac{3}{4}$	13.92	4.28	5,700	″	″	5.55	2.22	3,000	″	″
$\frac{11}{16}$	12.99	3.97	5,300	″	″	5.20	2.06	2,700	″	″
$\frac{5}{8}$	12.03	3.65	4,900	″	″	4.83	1.90	2,500	″	″
$\frac{9}{16}$	11.03	3.32	4,400	″	″	4.45	1.73	2,300	″	″
$\frac{1}{2}$	9.99	2.99	4,000	″	″	4.05	1.56	2,100	″	″
$\frac{7}{16}$	8.90	2.64	3,500	″	″	3.63	1.39	1,850	″	″
$\frac{3}{8}$	7.78	2.29	3,100	″	″	3.18	1.21	1,610	″	″
$\frac{5}{16}$	6.60	1.94	2,600	″	″	2.72	1.02	1,360	″	″
L-5x3x $\frac{13}{16}$	13.98	4.45	5,900	9–8	6–6	3.71	1.74	2,300	6–8	4–6
$\frac{3}{4}$	13.15	4.16	5,500	″	″	3.51	1.63	2,200	″	″
$\frac{11}{16}$	12.28	3.86	5,100	″	″	3.29	1.51	2,000	″	″
$\frac{5}{8}$	11.37	3.55	4,700	″	″	3.06	1.39	1,850	″	″
$\frac{9}{16}$	10.43	3.23	4,300	″	″	2.83	1.27	1,690	″	″
$\frac{1}{2}$	9.45	2.91	3,900	″	″	2.58	1.15	1,530	″	″
$\frac{7}{16}$	8.43	2.58	3,400	″	″	2.32	1.02	1,360	″	″
$\frac{3}{8}$	7.37	2.24	3,000	″	″	2.04	0.89	1,190	″	″
$\frac{5}{16}$	6.26	1.89	2,500	″	″	1.75	0.75	1,000	″	″
L-4½x3x $\frac{13}{16}$	10.33	3.62	4,800	8–9	5–9	3.60	1.71	2,300	6–6	4–4
$\frac{3}{4}$	9.73	3.38	4,500	″	″	3.40	1.60	2,100	″	″
$\frac{11}{16}$	9.10	3.14	4,200	″	″	3.19	1.49	1,990	″	″
$\frac{5}{8}$	8.44	2.89	3,900	″	″	2.98	1.37	1,830	″	″
$\frac{9}{16}$	7.75	2.64	3,500	″	″	2.75	1.25	1,670	″	″
$\frac{1}{2}$	7.04	2.37	3,200	″	″	2.51	1.13	1,510	″	″
$\frac{7}{16}$	6.29	2.10	2,800	″	″	2.25	1.01	1,350	″	″
$\frac{3}{8}$	5.50	1.83	2,400	″	″	1.98	0.88	1,180	″	″
$\frac{5}{16}$	4.69	1.54	2,100	″	″	1.73	0.76	1,010	″	″
L-4x3½x $\frac{13}{16}$	7.77	2.92	3,900	8–0	5–6	5.49	2.30	3,100	7–6	5–0
$\frac{3}{4}$	7.32	2.75	3,700	″	″	5.18	2.15	2,900	″	″
$\frac{11}{16}$	6.86	2.56	3,400	″	″	4.86	2.00	2,700	″	″
$\frac{5}{8}$	6.37	2.35	3,100	″	″	4.52	1.84	2,500	″	″
$\frac{9}{16}$	5.86	2.15	2,900	″	″	4.17	1.68	2,200	″	″
$\frac{1}{2}$	5.32	1.93	2,600	″	″	3.79	1.52	2,000	″	″
$\frac{7}{16}$	4.76	1.72	2,300	″	″	3.40	1.35	1,800	″	″
$\frac{3}{8}$	4.18	1.50	2,000	″	″	2.99	1.18	1,570	″	″
$\frac{5}{16}$	3.56	1.26	1,680	″	″	2.59	1.01	1,350	″	″
L-4x3x $\frac{13}{16}$	7.34	2.87	3,800	8–0	5–4	3.47	1.68	2,200	6–6	4–4
$\frac{3}{4}$	6.93	2.68	3,600	″	″	3.28	1.57	2,100	″	″
$\frac{11}{16}$	6.49	2.49	3,300	″	″	3.08	1.46	1,950	″	″
$\frac{5}{8}$	6.03	2.30	3,100	″	″	2.87	1.35	1,800	″	″
$\frac{9}{16}$	5.55	2.09	2,800	″	″	2.66	1.23	1,640	″	″

TABLE III (Continued)

Strength of Beams

I-Beams; H-Sections; Channels; Angles; and Tees

SECTION	LONG LEG VERTICAL					SHORT LEG VERTICAL				
	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Extreme Length for Deflection for Plastered Ceilings Limit 1–480 Span: For Uniformly Distributed Load	Extreme Length for Deflection for Plastered Ceilings Limit 1–480 Span: For Center Load	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. Inch	Extreme Length for Deflection for Plastered Ceilings Limit 1–480 Span: For Uniformly Distributed Load	Extreme Length for Deflection for Plastered Ceilings Limit 1–480 Span: For Center Load
	$(In.)^4$	$(In.)^3$	Ft.-Lb.	Ft. In.	Ft. In.	$(In.)^4$	$(In.)^3$	Ft.-Lb.	Ft. In.	Ft. In.
L-4x3x $\frac{1}{2}$	5.05	1.89	2,500	8–0	5–4	2.42	1.12	1,490	6–6	4–4
$\frac{7}{16}$	4.52	1.68	2,200	"	"	2.18	0.99	1,320	"	"
$\frac{3}{8}$	3.96	1.46	1,940	"	"	1.92	0.87	1,160	"	"
$\frac{5}{16}$	3.38	1.23	1,640	"	"	1.65	0.74	990	"	"
L-3½x3x $\frac{13}{16}$	4.98	2.20	2,900	7–0	4–9	3.33	1.65	2,200	6–4	4–3
$\frac{3}{4}$	4.70	2.05	2,700	"	"	3.15	1.54	2,100	"	"
$\frac{11}{16}$	4.41	1.91	2,500	"	"	2.96	1.44	1,920	"	"
$\frac{5}{8}$	4.11	1.76	2,300	"	"	2.76	1.33	1,770	"	"
$\frac{9}{16}$	3.79	1.61	2,100	"	"	2.55	1.21	1,610	"	"
$\frac{1}{2}$	3.45	1.45	1,930	"	"	2.33	1.10	1,470	"	"
$\frac{7}{16}$	3.10	1.29	1,720	"	"	2.09	0.98	1,310	"	"
$\frac{3}{8}$	2.72	1.13	1,510	"	"	1.85	0.85	1,130	"	"
$\frac{5}{16}$	2.33	0.96	1,280	"	"	1.58	0.72	960	"	"
L-3½x2½x $\frac{11}{16}$	4.13	1.85	2,500	6–4	4–6	1.72	0.99	1,320	5–4	3–6
$\frac{5}{8}$	3.85	1.71	2,300	"	"	1.61	0.92	1,230	"	"
$\frac{9}{16}$	3.55	1.56	2,100	"	"	1.49	0.84	1,120	"	"
$\frac{1}{2}$	3.24	1.41	1,880	"	"	1.36	0.76	1,010	"	"
$\frac{7}{16}$	2.91	1.26	1,680	"	"	1.23	0.68	910	"	"
$\frac{3}{8}$	2.56	1.09	1,450	"	"	1.09	0.59	790	"	"
$\frac{5}{16}$	2.19	0.93	1,240	"	"	0.94	0.50	670	"	"
$\frac{1}{4}$	1.80	0.75	1,000	"	"	0.78	0.41	550	"	"
L-3¼x2x $\frac{9}{16}$	2.64	1.30	1,730	6–4	4–3	0.75	0.53	710	4–4	3–0
$\frac{1}{2}$	2.42	1.17	1,560	"	"	0.69	0.48	640	"	"
$\frac{7}{16}$	2.18	1.05	1,400	"	"	0.62	0.43	570	"	"
$\frac{3}{8}$	1.92	0.91	1,210	"	"	0.55	0.37	490	"	"
$\frac{5}{16}$	1.65	0.77	1,030	"	"	0.48	0.32	430	"	"
$\frac{1}{4}$	1.36	0.63	840	"	"	0.40	0.26	350	"	"
L-3x2½x $\frac{9}{16}$	2.28	1.15	1,530	6–0	4–0	1.42	0.82	1,090	4–4	3–6
$\frac{1}{2}$	2.08	1.04	1,390	"	"	1.30	0.74	990	"	"
$\frac{7}{16}$	1.88	0.93	1,240	"	"	1.18	0.66	880	"	"
$\frac{3}{8}$	1.66	0.81	1,080	"	"	1.04	0.58	770	"	"
$\frac{5}{16}$	1.42	0.69	920	"	"	0.90	0.49	650	"	"
$\frac{1}{4}$	1.17	0.56	750	"	"	0.74	0.40	530	"	"
L-3x2x $\frac{1}{2}$	1.92	1.00	1,330	5–9	3–9	0.67	0.47	630	4–4	3–0
$\frac{7}{16}$	1.73	0.89	1,190	"	"	0.61	0.42	560	"	"
$\frac{3}{8}$	1.53	0.78	1,040	"	"	0.54	0.37	490	"	"
$\frac{5}{16}$	1.32	0.66	880	"	"	0.47	0.32	430	"	"
$\frac{1}{4}$	1.09	0.54	720	"	"	0.39	0.25	330	"	"
L-2½x2x $\frac{1}{2}$	1.14	0.70	930	5–0	3–4	0.64	0.46	610	4–3	2–9
$\frac{7}{16}$	1.03	0.62	830	"	"	0.58	0.41	550	"	"
$\frac{3}{8}$	0.91	0.55	730	"	"	0.51	0.36	480	"	"
$\frac{5}{16}$	0.79	0.47	630	"	"	0.45	0.31	410	"	"
$\frac{1}{4}$	0.65	0.38	510	"	"	0.37	0.25	330	"	"
$\frac{3}{16}$	0.51	0.29	390	"	"	0.29	0.20	270	"	"

TABLE III (Continued)

Strength of Beams

I-Beams; H-Sections; Channels; Angles; and Tees

SECTION	Moment of Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. In.	Extreme Length for Deflection for Plastered Ceilings Limit 1–480 Span: For Uniformly Distributed Load	Extreme Length for Deflection for Plastered Ceilings Limit 1–480 Span: For Center Load
Flange × Stem × Weight	(In.)^4	(In.)^3	Foot-pounds	Ft. In.	Ft. In.
T-5 x3 –13.6	2.6	1.18	1,570	6–9	4–6
T-5 x2½–11.0	1.6	0.86	1,150	5–6	3–8
T-4½x3½–15.9	5.1	2.13	2,840	7–2	4–9
T-4½x3 – 8.6	1.8	0.81	1,080	6–9	4–6
–10.0	2.1	0.94	1,250	"	"
T-4½x2½– 8.0	1.1	0.56	750	5–9	3–10
9.3	1.2	0.65	870	"	"
T-4 x5 –15.7	10.7	3.10	4,140	10–6	7–0
12.3	8.5	2.43	3,240	"	"
T-4 x4½–14.8	8.0	2.55	3,400	9–6	6–3
11.6	6.3	1.98	2,640	"	"
T-4 x4 –13.9	5.7	2.02	2,690	8–6	5–8
10.9	4.7	1.64	2,190	"	"
T-4 x3 – 9.3	2.0	0.88	1,170	6–8	4–6
T-4 x2½– 8.7	1.2	0.62	830	5–8	3–9
7.4	1.0	0.55	730	"	"
T-4 x2 – 7.9	0.6	0.40	530	4–6	3–0
6.7	0.54	0.34	450	"	"
T-3½x4 –12.8	5.5	1.98	2,640	8–4	5–6
10.0	4.3	1.55	2,070	"	"
T-3½x3½–11.9	3.7	1.52	2,030	7–6	5–0
9.3	3.0	1.19	1,590	"	"
T-3½x3 –11.0	2.4	1.13	1,510	6–6	4–4
8.7	1.9	0.88	1,170	"	"
7.7	1.6	0.72	960	"	"
T-3 x4 –11.9	5.2	1.94	2,590	8–0	5–4
10.6	4.8	1.78	2,370	"	"
9.3	4.3	1.57	2,100	"	"
T-3 x3½ 11.0	3.5	1.49	1,990	7–2	4–9
9.8	3.3	1.37	1,830	"	"
8.6	2.9	1.21	1.610	"	"

SECTION	Moment for Inertia I	Section Modulus $\frac{I}{c}$	Resisting Moment Based on Unit Stress of 16,000 Lb. per Sq. In.	Extreme Length for Deflection for Plastered Ceilings Limit 1–480 Span: For Uniformly Distributed Load	Extreme Length for Deflection for Plastered Ceilings Limit 1–480 Span: For Center Load
Flange × Stem × Weight	(In.)^4	(In.)^3	Foot-pounds	Ft. In.	Ft. In.
T-3 x3 –10.1	2.3	1.10	1,470	6–3	4–3
9.0	2.1	1.01	1,350	"	"
7.9	1.8	0.86	1,150	"	"
6.8	1.6	0.74	990	"	"
T-3 x2½– 7.2	1.1	0.60	800	5–4	3–6
6.2	0.94	0.52	690	"	"
T-2¾x2 – 7.4	1.1	0.75	1,000	4–6	3–0
T-2½x3 – 7.2	1.8	0.87	1,160	6–0	4–0
6.2	1.6	0.76	1,010	"	"
T-2½x2¾– 6.8	1.4	0.73	970	5–8	3–9
5.9	1.2	0.60	800	"	"
T-2½x2½– 6.5	1.0	0.59	790	3–6	5–3
5.6	0.87	0.50	670	"	"
T-2½x1¼– 3.0	0.094	0.09	120	3–0	2–0
T-2¼x2¼– 5.0	0.66	0.42	560	4–9	3–0
4.2	0.51	0.32	430	"	"
T-2 x2 – 4.4	0.45	0.33	440	4–0	2–9
3.7	0.36	0.25	330	"	"
T-2 x1½– 3.2	0.16	0.15	200	3–3	2–2
T-1¾x1¾– 3.2	0.23	0.19	250	3–8	2–6
T-1½x1½– 2.6	0.15	0.14	190	3–3	2–2
2.0	0.11	0.11	150	"	"
T-1¼x1¼– 2.1	0.08	0.10	130	2–6	1–9
1.7	0.06	0.07	93	"	"
T-1 x1 – 1.3	0.03	0.05	67	2–0	1–4
1.0	0.02	0.03	40	"	"

This can be applied to designing girders for floor panels. Fig. 86 shows a section of floor with several arrangements of joists. When the girder length is divided by the joists into an even number of spaces as 2, 4, and 6 in (a), (b), and (c), respectively, Fig. 86, the bending moment on the girder is the same as if the entire panel load were uniformly distributed over the length of the girder. When the girder length is divided by the joists into an odd number of spaces as 3, 5, and 7 in (d), (e), and (f), respectively, the bending moment is *less* than if the entire panel load were uniformly distributed over the length of the girder.

PROBLEM

To prove the foregoing statements, assume panels 20 feet square and a load of 100 pounds per square foot. Compute the bending moments on the girder for all the cases illustrated in Fig. 86.

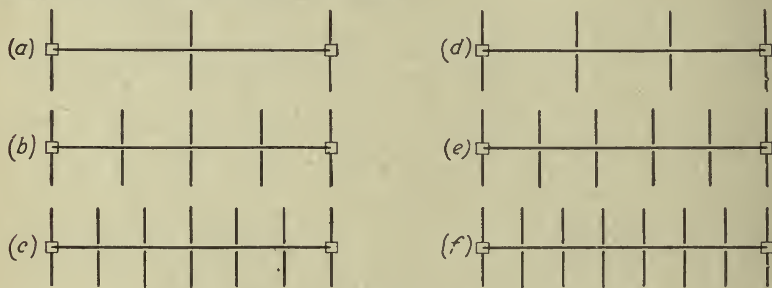


Fig. 86. Diagrams of Girders Showing Types of Joist Spacing

Shearing Resistance. It has been stated, p. 79, that the maximum shear in a beam section can be determined approximately by assuming that the entire shear is resisted by the web of the beam. For this purpose the area of the web may be taken as the total depth of the beam multiplied by the thickness of the web. Then the total resistance V is the area of the web A multiplied by the allowable unit shear S_s and is expressed by the formula

$$V = A \times S_s$$

The unit stress allowed is 10,000 pounds per square inch. For example, to determine the shearing resistance of a 12″ I 40#:

$$A = 12 \times 0.46 = 5.52 \text{ sq. in.}$$

then $$V = 5.52 \times 10{,}000 = 55{,}200\#$$

PROBLEM

Refer to the problems given under bending resistance. Compute the shearing resistance of the beams and compare with the maximum shearing stress.

The shearing resistance is usually much in excess of the amount required. It need not be investigated unless the span is short or unless a heavy load is applied near a support so that it produces a small bending moment and high shear. The values of the shearing resistance of beams are given in Table III. By the use of this table the shearing resistance of the beam which has been selected can be compared with the computed maximum shear on the beam.

Of more importance is the *strength of the standard end connections* for beams. These are discussed in a later section of this text. Their values are given in Table III. In all cases the strength of the connection is less than the shearing strength of the beam. Hence, the strength of the connection must be compared with the maximum shear on beams. If the standard connection is not strong enough, a special one must be devised and the strength of the web investigated.

Deflection. The deflection of a beam may be of as much importance as its strength. If its amount is noticeable, it gives the impression of weakness. This is especially true when it shows a definite change under the application and removal of live load. If the beam deflects unduly, it will cause cracks in the supported material. The most common results of too much deflection are cracks in plaster under the middle of joist spans and cracks in tile or concrete floors over the ends of joists where they connect to girders. This is shown in an exaggerated way in Fig. 70. It is not uncommon to find such unsightly cracks in the tile or marble floors of high-grade buildings. It has been determined experimentally that plaster will crack when the deflection is $\frac{1}{360}$ of the span, i. e., 1 inch in 30 feet; but a much lower value should be used for masonry and for marble floors and ceilings.

Deflection Formulas. Deflection formulas (p. 80) are as follows:

for uniformly distributed load $d=\frac{5}{384}\frac{Wl^3}{EI}$

for load concentrated at center $d=\frac{1}{48}\frac{Wl^3}{EI}$

in which d is deflection in inches; W is total load; l is length in inches; E is modulus of elasticity; and I is moment of inertia.

To illustrate their use, assume a 12″ I$31\frac{1}{2}$#, span 15 feet, or 180 inches, load u. d. 25,000 pounds. The value of I for this beam is 215.8. Then

$$d=\frac{5}{384}\times\frac{25{,}000\times180\times180\times180}{30{,}000{,}000\times215.8}=0.29''$$

If we change the load from u. d. to concentrated

$$d=\frac{1}{48}\times\frac{25{,}000\times180\times180\times180}{30{,}000{,}000\times215.8}=0.47''$$

A comparison of the results shows that the deflection is 1.6 times as much for the concentrated load as for the uniformly distributed load. If both the above loads are applied at the same time, the total deflection is the sum of the two amounts computed above, i. e.,

$$d=0.29''+0.47''=0.76''$$

Formulas are given in the handbooks for other forms of loading, but as they are not used often they are not given here. Concentrated loads within the middle third may be treated as if at the center, and if outside the middle third, as if uniformly distributed. The results from this approximate method will be reasonably close to the correct values.

Safe Span Length. Based on a maximum deflection of $\frac{1}{360}$ of the span, and on a unit stress of 16,000 pounds per square inch, the permissible span is *25 times the depth* for a uniformly distributed load and *15.6 times the depth* for a center load. These relations are correct for sections symmetrical about the neutral axis, as I-beams and channels. They err on the safe side for unsymmetrical sections, as angles and tees, and may be used for them. These values should be considered the extreme lengths for beams loaded to their full capacity. It is preferred that shorter lengths be used for several reasons: viz, noticeable deflection is objectionable; the greatest practicable stiffness is desired; deflection causes secondary stresses in the connections.

The handbooks, in their tables of "Safe Loads Uniformly Distributed for I-beams", limit the span length for deflection to 24 times the depth. The designer must use his judgment in this matter, giving consideration to the conditions of loading. A convenient rule for a u. d. load is 2 feet of length for each inch of depth (24 times the depth); and for a center load $1\frac{1}{3}$ feet of length for each inch of depth (16 times the depth). Table III gives the maximum allowable spans for these ratios, based on a unit stress of 16,000 pounds per square inch. If, however, the unit stress is less than 16,000 pounds, longer spans may be used.

In most cases the beam section required to resist the bending moment comes well within the limiting length for deflection. It is only when a long span has a relatively light load that deflection must be considered. This condition occurs most frequently in joists. Girders rarely have excessive deflection.

To illustrate such a case, assume a beam of 30-foot span supporting a load of 8000 pounds u. d. The bending moment is 30,000 foot-pounds, which requires 10″ I 25#. The length of this beam is 36 times its depth, therefore the deflection will be excessive. If it is decided arbitrarily to make the depth of beam $\frac{1}{24}$ of the span, the section required is 15″ I 42#. This beam, if loaded to full capacity, would deflect just to the allowed limit. But the resisting moment of 15″ I 42# is 79,000 foot-pounds, more than twice the bending moment computed above, hence its deflection being in direct proportion to the load is less than half that allowed. Assume that the deflection must not exceed 1 inch, i. e., $\frac{1}{360}$ of the span. Then try 12″ I $31\frac{1}{2}$# and compute the deflection from the formula

$$d=\frac{5}{384}\ \frac{W\,l^3}{E\,I}=\frac{5}{384}\times\frac{8000\times360\times360\times360}{30{,}000{,}000\times215.8}=0.75''$$

As the computed deflection is less than the allowed amount, the 12-inch I-beam is satisfactory.

The problem can be solved directly instead of by trial. Transform the equation to the form

$$I=\frac{5}{384}\times\frac{W\,l^3}{Ed}=\frac{5}{384}\times\frac{8000\times360\times360\times360}{30{,}000{,}000\times1}=162$$

The beam having a value I next higher than 162 is 12″ I $31\frac{1}{2}$#. The handbooks give explanations and tables for aiding the solution of this problem.

Attention is called to the fact that usually a joist receives a considerable percentage of its load (the floor construction) before the plastering is done. It has already deflected in proportion to the load it has received. It is only the subsequent loading and the resulting deflection that may crack the plaster. Consequently, the total deflection might be much greater than $\frac{1}{360}$ times the span and still not cause trouble. Nevertheless it is best to keep within this limit.

The situation regarding marble or concrete floors is quite different. Fig. 70 illustrates in an exaggerated way the joists in two panels, connecting to a cross girder. It takes but little deflection to cause cracks in the floor over the girder. No definite limit of deflection has been determined for this case. The writer has observed an instance where the deflection appeared to be less than $\frac{3}{8}$ inch in a span of 24 feet (about $\frac{1}{800}$). No definite suggestion can be made for taking care of this difficulty other than to make the joists as stiff as practicable within a reasonable cost. Probably this trouble can best be eliminated by the use of elastic joints in the floor over the girder.

PROBLEM

What I-beam is required to support a u. d. load of 4500 pounds on a span of 24 feet, the permissible deflection being ½ inch?

Lateral Support. If the top flange of a beam is not supported laterally, it is in much the same condition as a column. It is then not capable of supporting the full load given by the beam formula. In many cases where the lateral support is not furnished by the floor construction, connecting beams, or otherwise, it can be supplied by means of tie rods or struts inserted for that purpose. When no such lateral support can be provided, the allowable load must be reduced.

The handbooks contain tables which give the proportion of the total load that may be used for various ratios of length to width of flange. They permit the full load when the unsupported length is less than 20 times the width.

To illustrate the use of these tables assume a 12″ I 31½# 20 feet long, supported laterally at the center. The unsupported length is

10 feet, or 120 inches. The width of flange is 5 inches. Then the ratio of length to width of flange is $\frac{120}{5}=24$. In the Cambria handbook, the allowable load is 94 per cent of that given by the beam formula.

In Table III, the extreme lengths are given for beams without lateral support when loaded to full capacity and when loaded to half capacity. Intermediate values can be interpolated. The lengths given are, respectively, 20 and 60 times the flange width. *In all cases beams must have lateral support at the end bearings.*

PROBLEMS

1. What is the safe resisting moment of an 8″ I 18# on a 12-foot span when the top flange has no lateral support?

2. The required resisting moment of a beam is 42,000 foot-pounds; its unsupported length is 12 feet. What I-beam is required?

PRACTICAL APPLICATIONS

Panel of Floor Framing. Fig. 87 illustrates a typical floor panel in a building. It is desired to investigate the various possible arrangements of framing for this panel. Assume that the dead load on the joists is 80 pounds per square foot including the weight of joists (but not the weight of the girders and their fireproofing); assume that the live load is 100 pounds per square foot on joists, and 85 pounds per square foot on girders.

Scheme (*a*). Scheme (a) places the girders on the longer span and divides the panel into 4 parts. The joists are spaced 5′-4½″ *c. c.*

Area supported by one joist $16\times 5\frac{3}{8}=86$ sq. ft.

$$\begin{array}{lr} \text{dead load on one joist } 86\times\ 80= & 6880\# \\ \text{live load on one joist } 86\times 100= & 8600\# \\ \hline \text{Total load} & 15{,}480\# \end{array}$$

This total load, 15,480 pounds, is uniformly distributed on a span 16 feet. The table of safe loads in the handbook indicates 10″ I 25#.

The girder carries the reaction of the joists on each side and the weight of itself and of its fireproofing (assumed at 200 pounds per lineal foot). On the theory that the whole floor will not be loaded at one time, the live load on the girder is taken at 85 pounds per

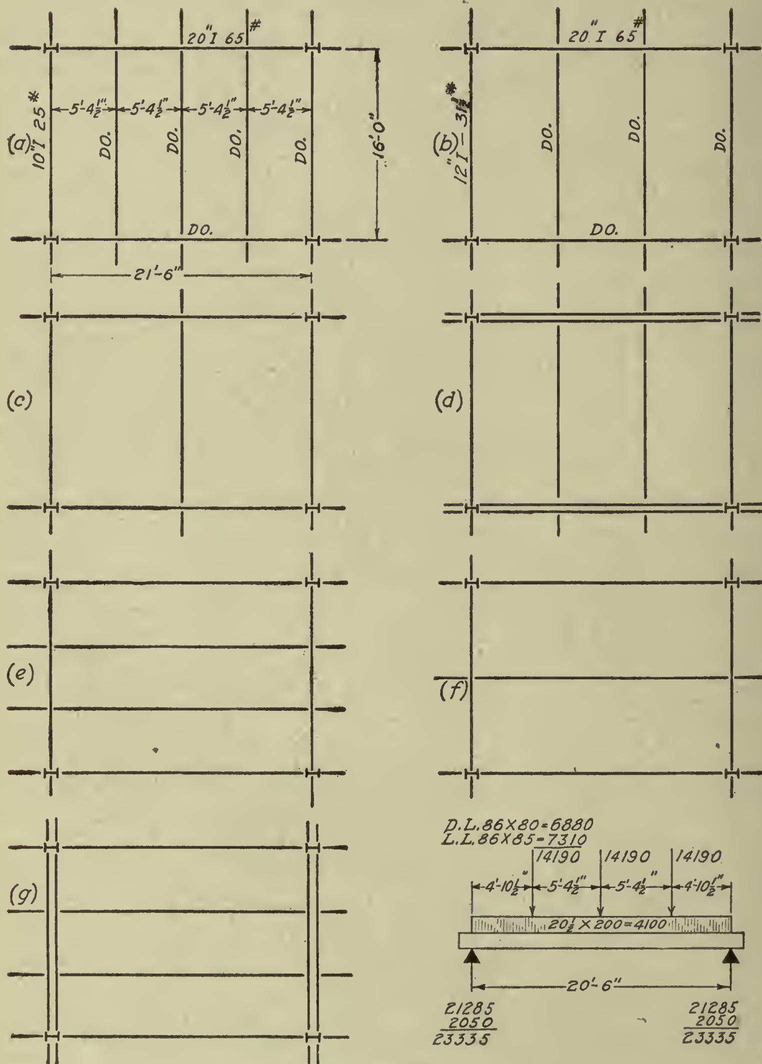


Fig. 87. A Panel of Floor Framing

square foot. The length of span is taken at 20′-6″ (allowance being made for the width of the column). Then the loads on the girder are as indicated in the figure and the bending moments are

for u. d. load $\dfrac{4100\times 20\frac{1}{2}}{8}$ = 10,500 ft.-lb.

for concentrated loads $\begin{cases} +21{,}285\times 10\frac{1}{4}=218{,}171 \\ -14{,}190\times\ 5\frac{3}{8}=-76{,}271 \end{cases}$

$\qquad\qquad =141{,}900$

Total bending moment $=152{,}400$ ft.-lb.

From the table of resisting moments, p. 100, 20″ I 65# is indicated.

Scheme (*b*). Scheme (b) places the girders on the longer span and divides the panel into 3 parts. This requires for the joists 12″ I 31½#; and for the girders 20″ I 65#.

EXAMPLES FOR PRACTICE

1. Determine the sizes of joists and girders required for scheme (c).

2. Determine the sizes of joists and girders required for scheme (d). Note that the girders are to be made of two I-beams. This makes the span of the joists 15′-4″.

3 In scheme (e) the girder is placed on the shorter span, as shown. Its net length is 15′-0″. Determine the sizes of joists and girders.

4. Determine the sizes of I-beams required for scheme (f).

5. In scheme (g) it is desired to make the joists and girders the same depth. This makes it necessary to use two I-beams for the girder. What sections are required?

6. Investigate all the beams in the foregoing problems as to shear, deflection, and strength of standard end connections.

7. Compute the weight of the I-beams required for one panel for each of the above schemes. There is one girder for each panel, and one joist for each division of the panel, i. e., four joists for scheme (a), three for scheme (b), etc. The weights for scheme (a) are

4 10″ I 25#×15′-11″=1592#
1 20″ I 65#×20′- 6″=1333#

8. Which scheme requires the least weight of steel?

Choice of Scheme. A number of considerations will affect the final decision as to the scheme to be adopted. The character of the floor construction will limit the spacing of the joists. It might eliminate schemes (b), (c), (d), and (f). The thickness of floor construction may be important, in which case scheme (a) would be preferred as to joists and scheme (g) as to girders. The thickness of floor may affect its cost and also the dead load to be carried by joists, girders, and columns, making the thinner floor preferable on this account. A flat ceiling may be required over the entire area, in which case scheme (g) is applicable.

PROBLEM

A space 14 feet wide and 100 feet long is to be floored over. This floor is to be supported by joists resting on brick side walls. The floor construction is such that the joists may be spaced not more than 8 feet *c. c.* Total load 200 pounds per square foot. Determine the most economical size and spacing of joists.

Lintels. Flat-topped openings through brick walls require lintels to support the masonry above. Brickwork, after it has hardened, will arch over such openings, the part of the brickwork below the thrust line of the arch being held in place by adhesion of the mortar. But there must be some support while the mortar is green, or the arch action may be destroyed by settlement, making a permanent support necessary. The amount of the load on lintels is uncertain. Each case must be decided according to the conditions.

Types of Construction. In Fig. 88 several cases are illustrated.

Case *a* is an opening with a solid wall above and at the sides. A satisfactory rule in this case is to figure the weight of brickwork within the triangle whose base equals the width of opening and whose slopes are 45 degrees.

In case *b* the shaded area might be entirely supported on the lintel over the lower opening.

Case *c* represents a spandrel wall between piers. The height of the brickwork is less than the width of the opening. The entire weight of the spandrel should be supported on the lintel.

In addition to the weight of the brickwork, the lintel may have to support the end of a girder as in case *d*, or it may have to support some floor area as in case *e*.

Case *f* shows a section through a wall in which the outer course of brickwork is supported by a lintel and the remainder by an arch.

In the following problems assume the weight of brickwork to be 120 pounds per cubic foot. Then for each superficial foot of wall the weight is 10 pounds for each inch of thickness.

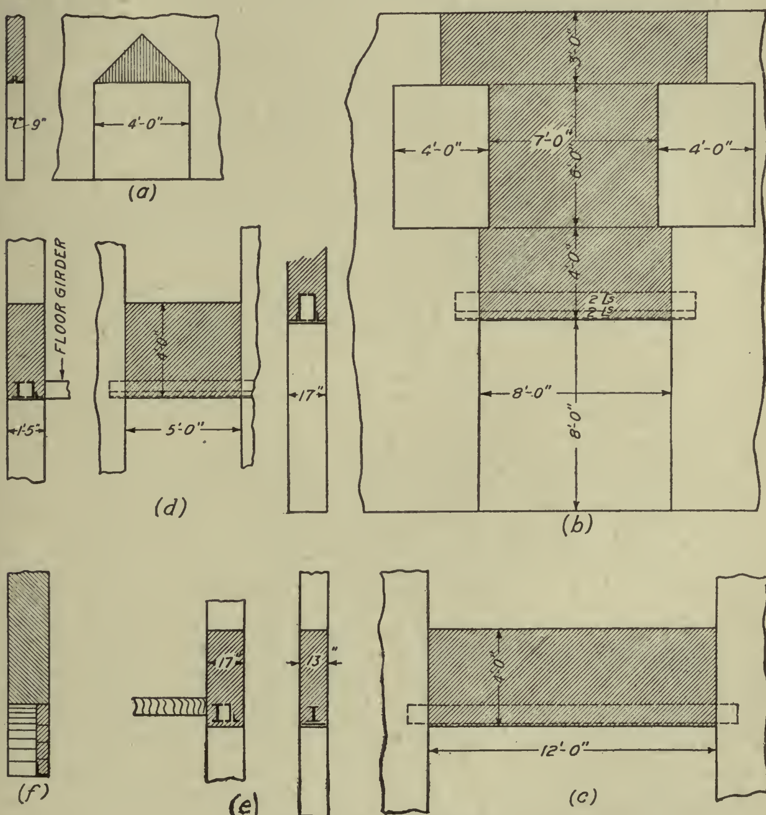


Fig. 88. Types of Lintels

EXAMPLES FOR PRACTICE

1. Design lintel for case *a*, span 4 feet, wall thickness 9 inches. Use 2 Ls. The horizontal legs of the angles should be $3\frac{1}{2}$ or 4 inches wide to support the brickwork properly. See Table II for formula for bending moment for this condition of loading.

2. Design the lintel required for conditions given for case *b*. Assume that the channels carry the entire load.

3. What section of I-beam is required for the lintel in case *c*? Neglect the value of the plate on the bottom of the beam.

4. In case *d* assume a load of 20,000 pounds from the girder in addition to the weight of brickwork. What section of I-beam and channel are required? Neglect the value of the angle.

5. In case *e* assume a load of 2000 pounds per lineal foot in addition to the weight of the wall. What section of I-beam and channel are required? The span is the same as for case *c*.

6. Determine the angle required to support the face brick across a 5-foot opening. (Case *f*). (The back is supported by brick arches.)

Cantilevers. Fig. 89 shows a beam projecting beyond the wall of a building, that is, a cantilever beam. The projection is 6 feet

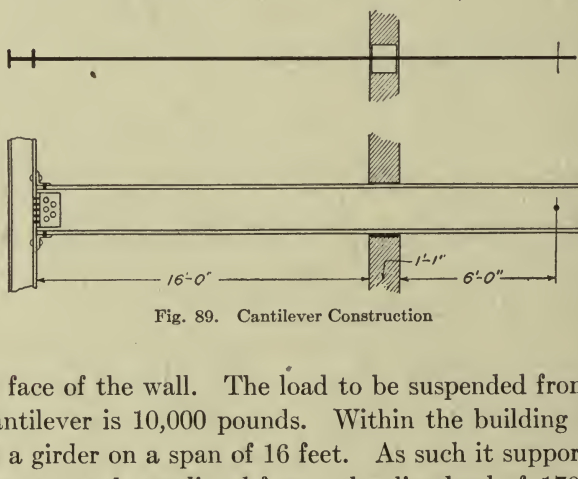


Fig. 89. Cantilever Construction

from the face of the wall. The load to be suspended from the end of the cantilever is 10,000 pounds. Within the building the beam serves as a girder on a span of 16 feet. As such it supports a dead load of 1600 pounds per lineal foot and a live load of 1700 pounds per lineal foot.

PROBLEM

Compute, from the data given above, the reactions and construct the moment and shear diagrams for each of the three following combinations of loading and determine the I-beam required:

(1) Dead load and live load
(2) Dead load and suspended load
(3) Dead load, live load, and suspended load

Tank Support. Fig. 90 illustrates the framework for supporting a wood water tank. The tank rests on 4″×6″ wood sub-joists spaced about 18 inches center to center. These in turn rest on steel joists. The load on the steel joists may be considered as uniformly distributed.

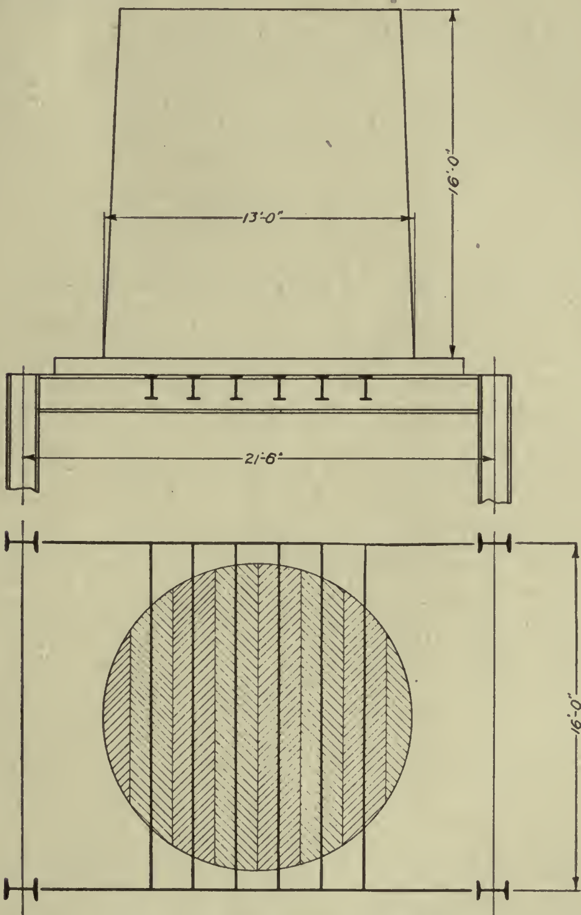


Fig. 90. Plan and Elevation of Tank Support

To compute the volume and weight, use the outside dimensions of the tank. (Assume the weight of water to be 62.5 pounds per cubic foot.) This will give some excess which will be sufficient to cover the weight of the steel beams. On this basis

$$\text{volume} = \frac{3.1416 \times 13 \times 13}{4} \times 16 = 2125 \text{ cu. ft.}$$

$$\text{weight} = 2125 \times 62.5 = 132{,}800\,\#$$

This can be used as a check on the sum of the partial loads. The load per square foot for 16 feet of water is 16×62.5 or 1000 pounds.

PROBLEMS

1. Lay out an assumed plan of the framework and the outline of the tank accurately to scale. Determine the area supported by each beam by measurements from the scale drawings as indicated by the shaded areas in the figure.

2. Compute the bending moment and shear for the several joists and the girders, and select the required I-beams. Check for strength of end connections.

DETAILS OF CONSTRUCTION

Connection of Beams to Beams. When one beam bears on top of another, the only connection required is rivets or bolts through the flange, as shown in Fig. 91. No stress is transmitted by these

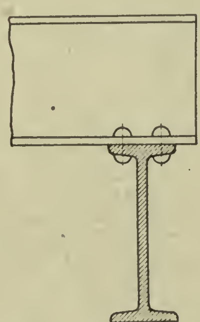


Fig. 91. Riveted Connection of Beam to Beam

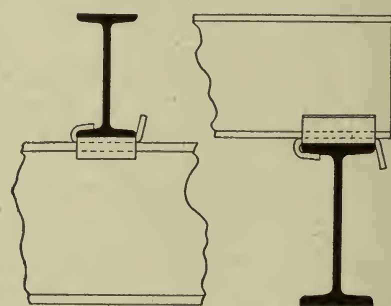


Fig. 92. Beam Connections by Means of Sheet Steel Clips

rivets or bolts. They serve simply to hold the beams in position. Steel clips are sometimes used for this purpose, Fig. 92, but as they are not positive in holding the beams in position they are not as good, especially when lateral support is required. When this is not important, the clips can be used and may effect a saving in cost. These clips are most useful for attaching tees and angles to beams in ceiling and roof construction.

Angle Connections. The most common method of connecting one beam to another is by means of angles riveted to the web. There are several sets of standard connections, various concerns having their own standards. Those of the American Bridge Company are given in Fig. 93.* The values given in Table III are based on these. The two-angle connection is generally used, but when beams are used in pairs or when for any reason the two-angle con-

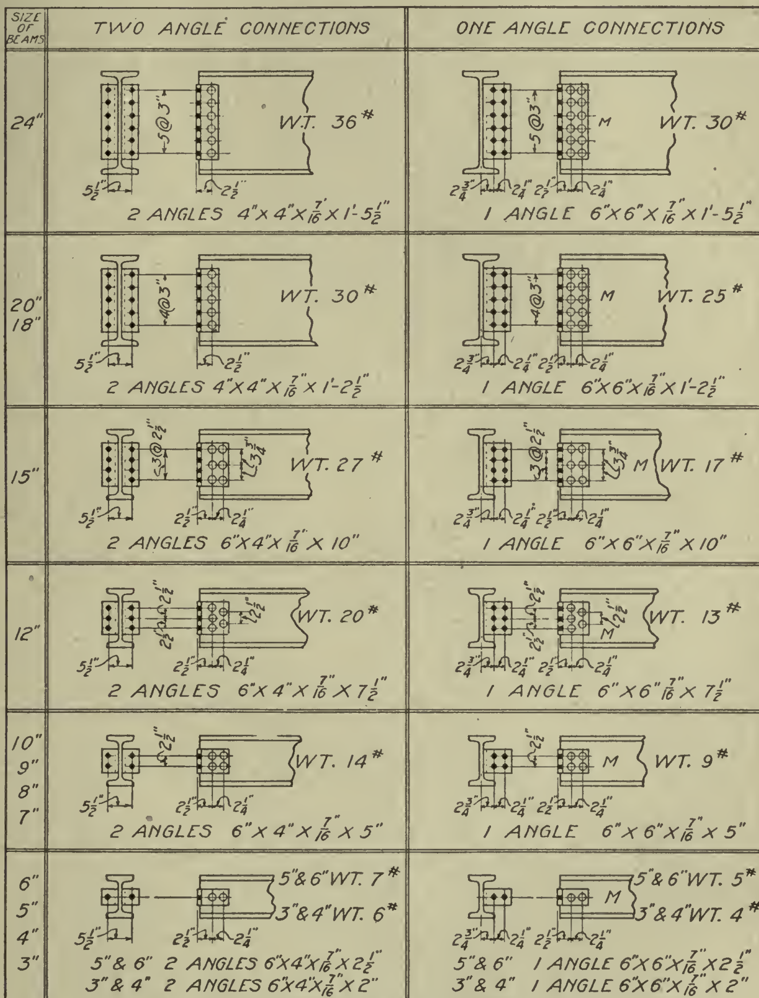


Fig. 93. Beam Connection Angles
Used by American Bridge Company

nection cannot be used, the one-angle connection is used. The rivets used in the standard connections are ¾ inch in diameter.

*Subsequently a different set of standards has been adopted. See Carnegie Pocket Companion, 1913 edition.

The strength of the two-angle connection may oe limited by

(1) Shop rivets in double shear
(2) Field rivets in single shear
(3) Shop rivets in bearing in web of joist
(4) Field rivets in bearing in web of girder

For example, take the connection for a 15″ I 42#:

(1) 6 shop rivets in double shear

$$6 \times 10{,}300 \qquad = 61{,}800\#$$

(2) 8 field rivets in single shear

$$8 \times 4420 \qquad = 35{,}360\#$$

(3) 6 shop rivets in bearing in web of joist

$$6 \times .41 \times .75 \times 25{,}000 = 46{,}125\#$$

(4) 8 field rivets in web of girder; the thickness of the web is not given. It must be at least 0.30 inch for a connection on one side only, or of twice this thickness if an equal connection is on the opposite side, in order to have the same strength as the field rivets in shear.

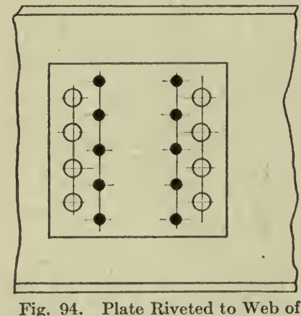


Fig. 94. Plate Riveted to Web of I-Beam to give Additional Bearing

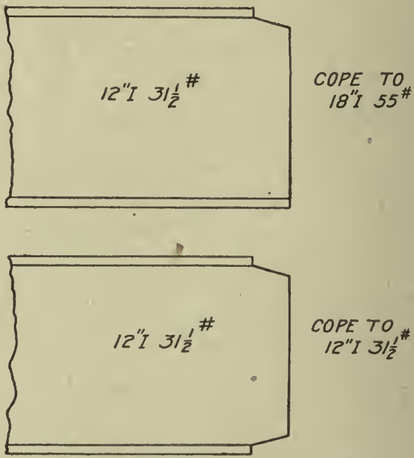


Fig. 95. Diagrams of Coped Beams

The shearing strength of this connection, 35,360 pounds, corresponds to the maximum safe u. d. load on a span of about 9 feet. It is less than the shearing strength of the web of the beam. It rarely happens that the] strength of the connection is less than required, and occurs only when the beam is short and heavily loaded or when a heavy load is applied near the end. Lack of bearing in the web of the girder is more likely to occur, but this is not fre-

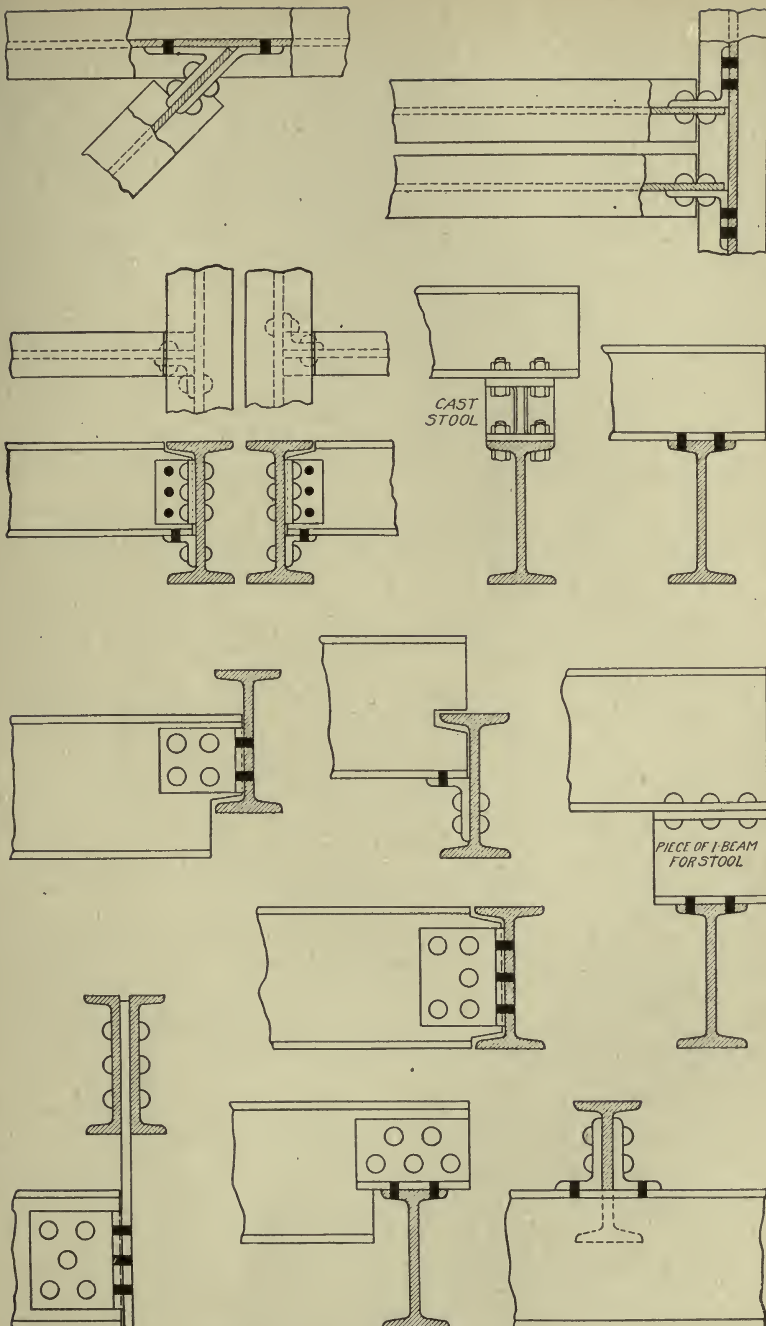


Fig. 96. Types of Beam Connections

quent. If it does happen, however, angles with 6-inch legs may be used to provide space for more rivets, or a reinforcing plate may be riveted to the web of the girder, Fig. 94.

Special Connections. When beams on the two sides of a girder do not come opposite or are of different sizes so that the standard connections do not match, it is necessary to devise a special connection. If a beam is flush on the top or on the bottom with the one to which it connects, the flange must be coped, Fig. 95. A number of special connections are shown in Fig. 96 and need no explanation.

Connections of Beams to Columns. A beam may connect to a column by means of a seat or by means of angles on the web. The great variety of conditions that may be encountered make it impracticable to have standards for these connections, though the work of each shop is standardized to some extent.

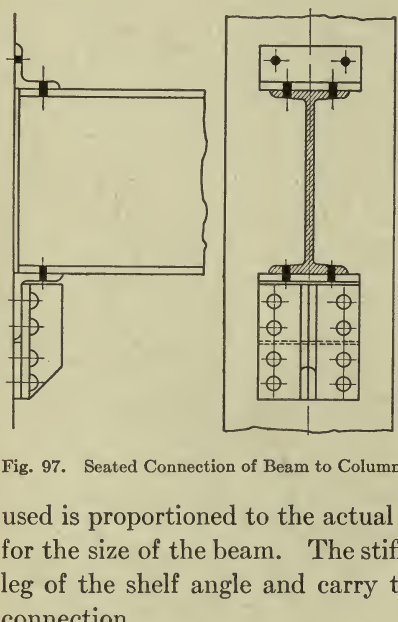


Fig. 97. Seated Connection of Beam to Column

Seat Connections. The seat connection is shown in Fig. 97. This seat or bracket is made up of a shelf angle, one or two stiffener angles, and a filler plate. The load is transmitted by the rivets, acting in single shear, which connect the bracket to the column. The number of rivets used is proportioned to the actual load instead of being standardized for the size of the beam. The stiffener angles support the horizontal leg of the shelf angle and carry the load to the lower rivets of the connection.

Shelf angles are 6 inches, 7 inches, or 8 inches vertical and 4 inches or 6 inches horizontal, having a thickness of $\frac{7}{16}$ inch to $\frac{3}{4}$ inch, depending on the size of beam and the load. The leg of the stiffener angle parallel to the web of the beam is usually $\frac{1}{2}$ inch or 1 inch less than the horizontal leg of the shelf. The leg against the column is governed by the gage line of the rivets in the column. The filler is the same thickness as the shelf angle. An angle connecting the top flange of the beam to the column is generally used. It is not counted as carrying any of the load, but serves to hold the top of the beam in position and stiffens the connection. The rivets connecting the bottom flange of the beam to the shelf serve only to hold the mem-

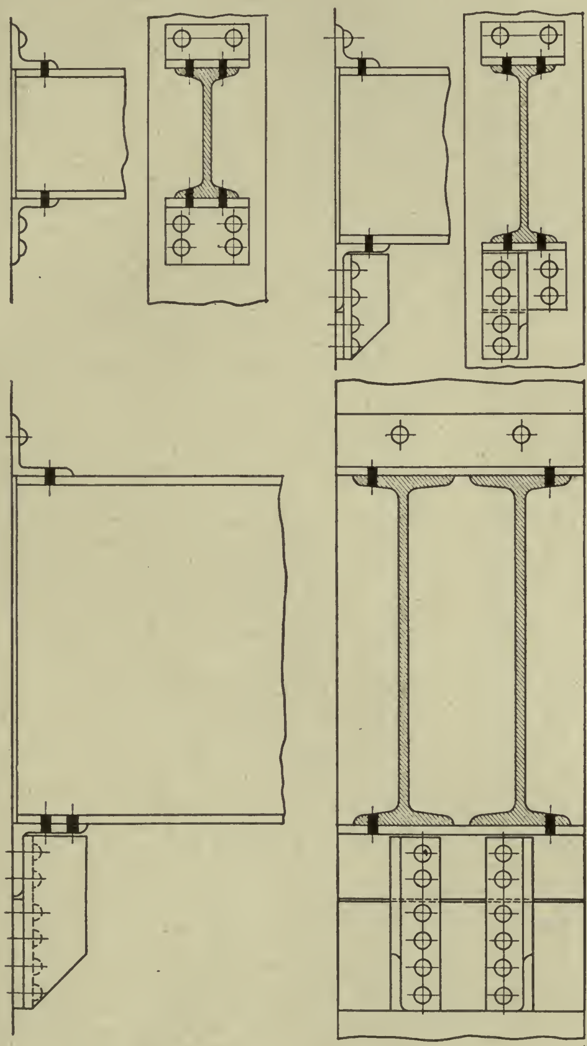


Fig. 98. Types of Seat Connections

bers together and make a stiff connection. Usually there are only two rivets in each flange but sometimes larger angles and more rivets are used to develop resistance to wind stresses. Fig. 98 gives a number of examples of seat connections.

The advantages of the seat connection are

(1) All shop riveting is on the column which is a riveted member. No shop riveting is required on the beam which thus needs only to be punched
(2) The seat is a convenience in erecting
(3) The rivets which carry shear are shop driven
(4) The number of field rivets is small

Web Connections. The web connection is made by means of two angles, Fig. 99. The legs parallel to the beam rivet to the web and the outstanding legs to the columns. The connection to the web of the beam is governed by the same conditions as the standard beam connection. The length of the outstanding leg is governed by the gage lines of the rivets in the column or the space available for them. Usually the angles are shop riveted to the beam and field riveted to the column. If the angles were shop riveted to the column, it would be difficult or impossible to erect the beam. However, one angle may be shop riveted to the column and the other furnished loose. In this case the number of field rivets generally will be the same as if the

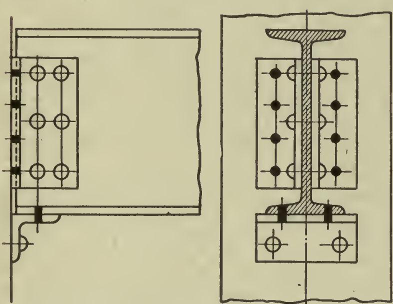


Fig. 99. Web Connection of Beam to Column

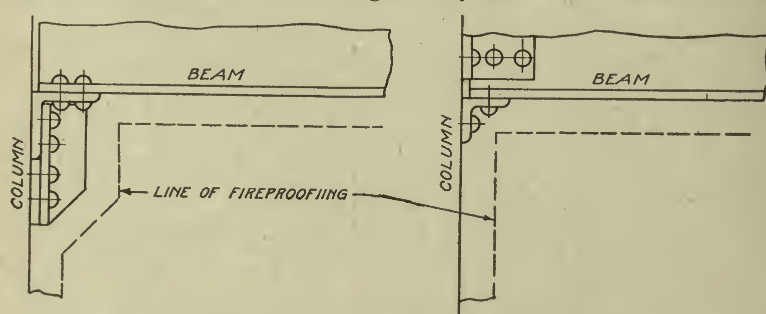


Fig. 100. Diagrams Showing Disadvantage of Seat Connection for Fireproofing

angles were shop riveted to the beam, but the shop riveting on the beam will be eliminated, which is an advantage. When this connection is used, a small seat angle is provided for convenience in erecting.

The advantage of the web connection is the compactness of the parts, keeping within the limits of the fireproofing and plaster, whereas the seat connection may necessitate special architectural treatment to fireproof it or conceal it, Fig. 100.

Combination Connections. A combination of web and seat connections may be used to meet special conditions. For example, the load may be too great for a web connection, and at the same time a seat connection may be objectionable. The combination will reduce the seat connection to a minimum, perhaps eliminating the stiffener angles. Another case is where top and bottom angles are required for wind bracing but stiffener angles are not permitted; there the combination can be used.

The objection to the combination is that there are two groups of rivets for supporting the load. If the connection is not accurately made, the entire load may be carried by one group of rivets. A number of miscellaneous connections are illustrated later in the text under column details.

Separators. When beams are used in pairs or groups, some connection is usually made between them at short intervals. The connecting piece is called a "separator". If the purpose to be served is merely to tie the beams together and keep them properly spaced, the gas-pipe separator is used, Fig. 101. This consists of a piece of gas pipe with a bolt running through it. This form is used in lintels and in grillage beams. For beams 6 inches or less in depth, one separator and bolt may be used; for greater depth, two should be used.

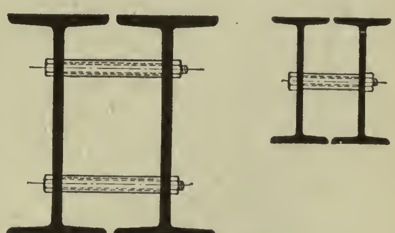


Fig. 101. Gas-Pipe Separators

The separator most commonly used is made of cast iron, Fig. 102. It not only serves as a spacer but it stiffens the webs of the beams and, to a limited extent, transmits the load from one beam to the other in case one is loaded more heavily. It seldom fits exactly to the beam so it cannot be relied upon to transmit much load. One bolt is used for beams less than 12 inches deep and two bolts for 12-inch and deeper beams. The dimensions and weights of separators and the bolts for them are given in the handbooks. They can be made for any spacing of beams and special shapes can

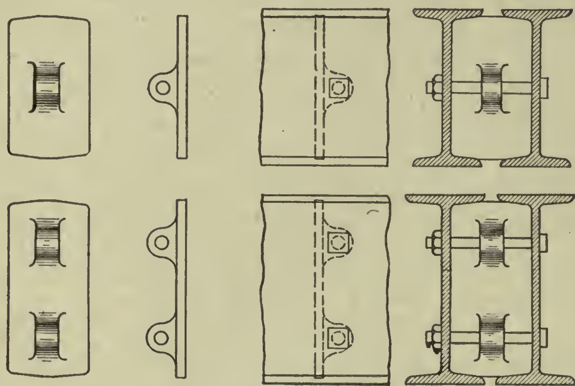


Fig. 102. Cast-Iron Separators

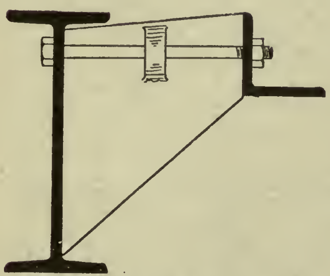


Fig. 103. Special Type of Cast-Iron Separators

be made for beams of different sizes, Fig. 103.

The individual beams of a pair or group should be designed for the actual loads which they carry, if it is practicable to do so. If it is necessary to transfer some load from one to the other, a steel separator or diaphragm should be used. This may be made of a

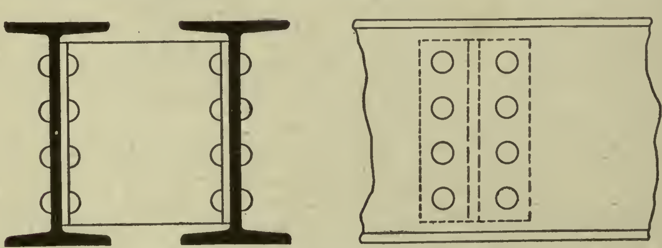


Fig. 104. Steel Separator or Diaphragm

plate and four angles or of a short piece of I-beam or channel, Fig. 104. If the beams are set close together, the holes must be reamed and turned bolts must be used in order to get an efficient connection. If the beams are set with four inches or more clearance between the flanges, the separator can be riveted to the beams.

Specifications usually require that separators be spaced not further than five feet apart. They should be placed at points of concentrated loads and over bearings.

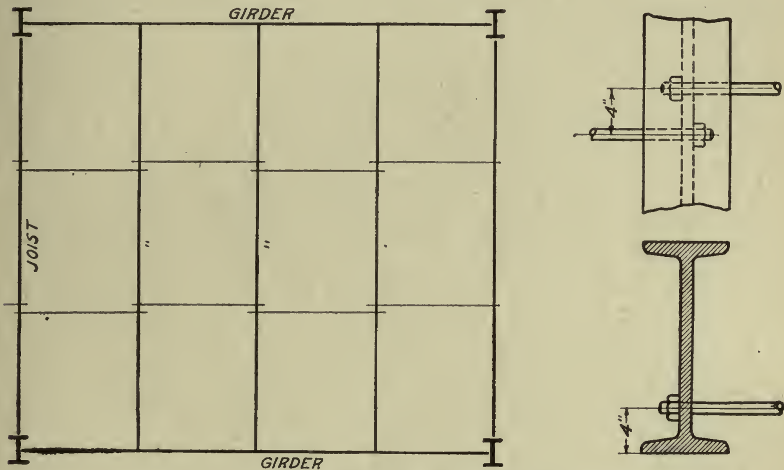


Fig. 105. Layout Showing Tie-Rod Connections Between Joists

Tie-Rods. A common form of fireproof floor construction is the hollow tile arch between steel joists spaced from 5 feet to 7 feet apart. The arch exerts a thrust sidewise on the beams and would spread the beams apart and cause the arch to fall, if they were not tied together. Rods $\frac{3}{4}$ inch in diameter are used for these ties. They are spaced about 6 feet apart and placed 3 or 4 inches above the bottom of the beams. After the arch construction is in place, the thrusts on the two sides of a beam would balance if equally

SEGMENTAL TERRA COTTA ARCH CONSTRUCTION
Fig. 106. Tie-Rod Connections for Segmental Arches

loaded so that under these conditions the rods would be needed only in the outside panels. However, they are needed in all panels during construction and as the loads on the several panels may be unequal, they are retained throughout the floor construction, Fig. 105.

If long span segmental arches are used, the thrust is much greater. Its amount must be computed and the tie-rods proportioned for the actual stress, Fig. 106.

Bearings. *Dimensions of Bearing Plates.* Under Unit Stresses are given the safe bearing values on masonry. The end of a beam resting on masonry usually does not have sufficient bearing area, and a bearing plate is required. The area of the plate is determined by dividing the load (the end reaction of the beam) by the allowed unit pressure on the masonry. For example, assume a 15″ I 42# bearing on a wall of hard brick in cement mortar, the reaction at the bearing being 18,000 pounds. The allowable pressure is 200 pounds per square inch. Then the required area of the plate is $\frac{18{,}000}{200}$ or 90 square inches. A plate 8″×12″ or one 10″×10″ would be used.

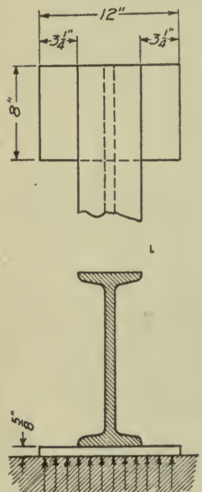


Fig. 107. Diagram Showing Bearing Plate

The required thickness of the bearing plate depends on the pressure per square inch on the masonry and the projection of the plate beyond the flange of the beam. This projecting portion of the plate acts as an inverted cantilever with a u. d. load. Thus in Fig. 107 the beam is a 15″ I 42#, the plate 8″×12″. The projection of the plate is $3\frac{1}{4}$ inches and the upward pressure per square inch is 200 pounds. To determine the thickness, assume a strip 1 inch wide; then there is a cantilever $3\frac{1}{4}$ inches long with a load of 200 pounds per inch. The bending moment is

$$3.25\times200\times\frac{3.25}{2}=1056 \text{ in.-lb.}$$

From the bending moment the required section modulus $\frac{I}{c}$ can be obtained by the formula given on p.–98; and from it the thickness t of the plate can be obtained by the formula given on p.–37, thus

$$\frac{I}{c}=\frac{M}{S}=\frac{1056}{16{,}000}=.066$$

From the section modulus the thickness t can be computed by the reverse of the method previously given for computing I, thus

$$I=\frac{1}{12}bt^3 \qquad b=1'' \qquad c=\frac{t}{2}$$

$$\frac{I}{c}=\frac{1}{12}\,\frac{t^3}{\frac{t}{2}}=\frac{1}{6}t^2$$

$$t^2=6\times\frac{I}{c}=6\times .066=.396$$

$$t=\sqrt{.396}=0.63'', \text{ or } \tfrac{5}{8}'' \text{ thick}$$

The square root can be figured by the usual rules but can be obtained more easily from tables in the handbook.

Graphical Diagram for Designing Bearing Plates. Fig. 108 is a graphical diagram for designing bearing plates. Along the left side

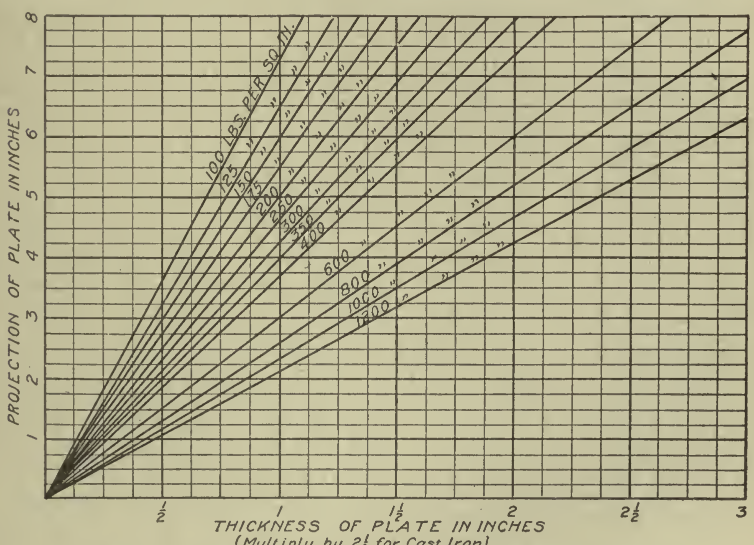


Fig. 108. Diagram for Determining Thickness of Steel Bearing Plates

is given the projection of the plate in inches; along the bottom is the thickness in inches; the diagonal lines represent the several allowable pressures for different classes of masonry. Having computed the size of plate needed for bearing, find the amount of its projection beyond the flange of the beam. Enter the diagram at the left on the horizontal line corresponding to the projection; trace to the right to the diagonal line representing the pressure; then vertically downward to the bottom of the diagram and read the thickness. For example, assume a projection of $3\frac{1}{4}$ inches and an allowable bearing of 200 pounds per square inch; the required thickness is $\frac{5}{8}$ inch.

Standard Bearing Plates. In the handbooks are given standard bearing plates for the various sizes of beams. One size of plate is given for each size of beam, hence these standard plates are designed for the heaviest loads likely to be carried by the heaviest beam section and, consequently, are larger than needed for most cases. In the example given above, the Cambria standard plate is $12'' \times 15'' \times \frac{3}{4}''$. It is larger than required, thus showing that it is economical to design the plates for the actual loads and the allowable bearing pressures. In this same example, if the bearing is on concrete at 400 pounds per square inch, no plate is required as the beam flange alone gives the necessary area.

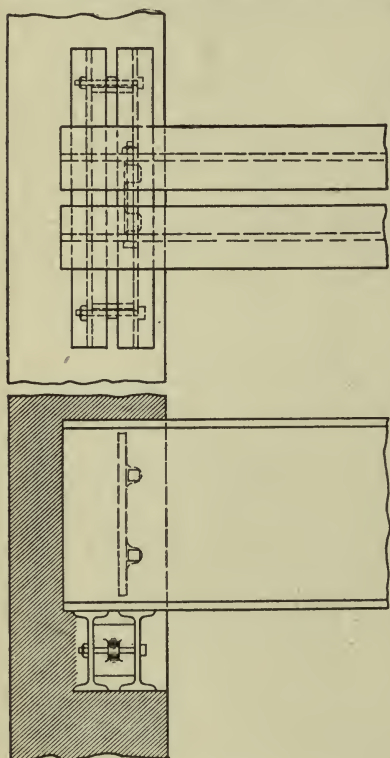


Fig. 109. I-Beams Used For Bearing

Penetration into Wall. The penetration of beams into the wall, if the thickness of wall permits, should be not less than the following:

for 3-inch, 4-inch, 5-inch, and 6-inch beams and channels	6 inches
for 7-inch, and 8-inch beams and channels	8 inches
for 9-inch, and 10-inch beams and channels	10 inches
for 12-inch, and 15-inch beams and channels	12 inches
for 18-inch, 20-inch, 21-inch, and 24-inch beams and channels	15 inches

When the thickness of the wall does not permit the penetration recommended above, the allowable bearing stress should be reduced. The reduction should be 50 per cent for heavy beams on an 8-inch bearing. A penetration less than 8 inches should never be used for beams 8 inches or more in depth. Because all beams deflect under load their bearing plates should be set with a slight slope downward toward the face of the wall, $\frac{1}{8}$ inch per foot being a satisfactory slope. This prevents the whole load from being concentrated on the front edge of the plate.

Plates thicker than 1 inch are difficult to get. When this thickness is not enough for the projection desired, one or more

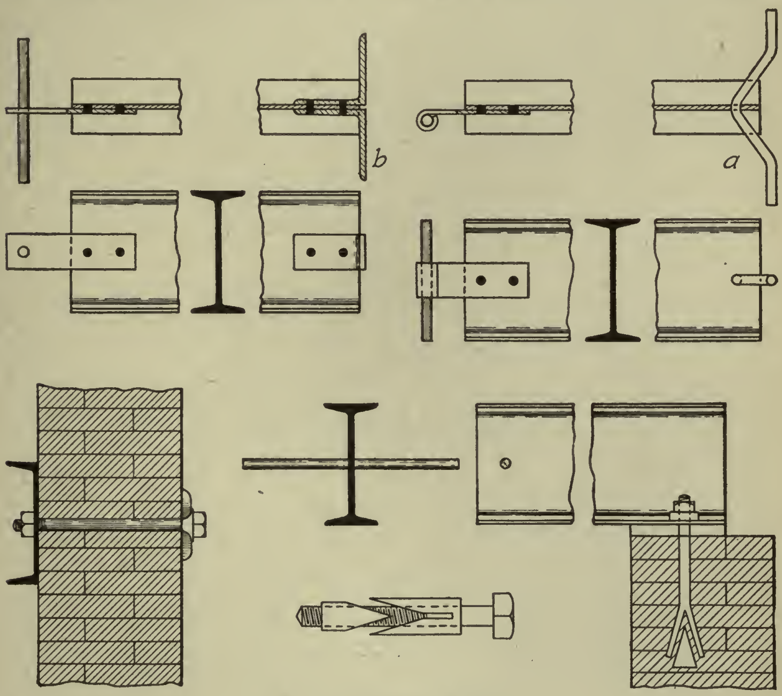


Fig. 110. Anchors for Beams

I-beams or channels should be used for the bearing, Fig. 109. These are designed as inverted cantilevers in the regular way.

Cast-Iron Plates. The foregoing discussion relates to steel plates. Cast-iron plates may be used. The method of designing them is the same as for steel plates, except that the allowable fiber stress is 3000 pounds per square inch. On account of this difference in the allowable stress, the thickness of the cast-iron plate is $2\frac{1}{4}$ times the thickness of the steel plate. The diagram, Fig. 108, may be used for cast iron by first determining the thickness for steel and multiplying the result by $2\frac{1}{4}$. In most localities the cast iron costs more than steel on account of the additional weight.

Anchors. Beams bearing on masonry are usually anchored to it to give greater stability to the structure as a whole. Fig. 110 shows the common forms of anchors used for this purpose. The bent rod *a* is the cheapest. The angle lugs *b* are the most efficient. The other forms are used for the special conditions indicated. The thickness of metal used is arbitrary, usually $\frac{3}{4}$ inch for rods and $\frac{3}{8}$ inch for angles and plates.

Miscellaneous Details. Almost every structure presents some conditions requiring special details of the beams. The relative position of the steel members may require a special form of connection, or the other materials of construction may necessitate special details for their support. A number of such details will be shown in connection with the practical designs later in this text.

RIVETED GIRDERS

Definition. The term "riveted girder" is here used to apply to all riveted beams, i. e., beams made of two or more steel sections

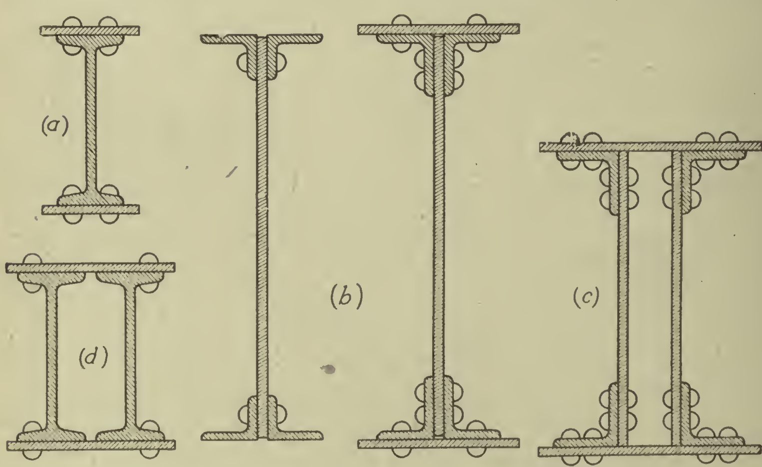


Fig. 111. Types of Riveted Girders

riveted together. The most common forms of riveted girders are illustrated in Fig. 111 as follows:

(a) I-beam with flange plates
(b) Plate girder
(c) Plate box girder
(d) Beam box girder

THEORY OF DESIGN

Determination of Resisting Moment. All that was stated under Review of Theory of Beam Design applies as well to riveted girders as to rolled beams, provided the sections are so riveted together that they act as a single piece. However, there are two methods of determining the resisting moment, viz, by *moment of inertia* and by *chord stress*, Fig. 112.

Moment of Inertia Method. The procedure for determining the resisting moment of a beam, or girder, by means of the moment of inertia has been fully explained. The value of I for the single

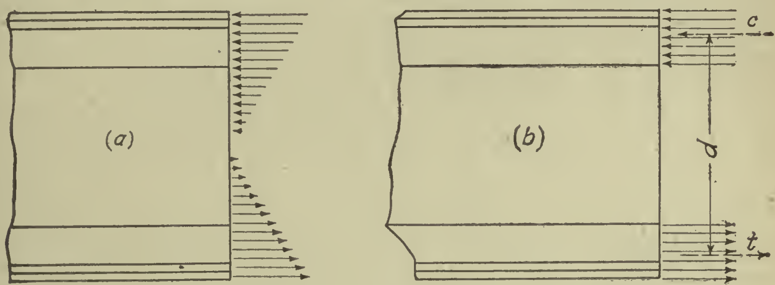


Fig. 112. Diagram of Bending Stresses in a Riveted Girder. (a) Moment of Inertia Method; (b) Chord Method

rolled section, such as the I-beam, is taken from the tables in the handbook, but for the riveted girder it must be computed.

Chord Stress Method. The second method of designing riveted girders assumes that the tensile stresses are resisted by the tension flange and the compressive stresses by the compression flange. It is assumed that the stress is uniformly distributed over the entire area of the flange. Then the moment of resistance is the same as if the whole stress were acting at the center of gravity of the flange area.

The resisting moment determined from the moment of inertia is

$$M = S\frac{I}{c}$$

The resisting moment by the chord method is as follows: In Fig. 112, t and c represent, respectively, the total tension and total compression values of the flanges, applied at the centers of gravity of the flange sections. The distance d between them is called the "effective depth of the girder". In order to have equilibrium, t must equal c. Each must equal the area A of the flange multiplied by the unit stress S. Then $t=c=A\times S$, and the resisting moment is

$$M=A\times S\times d$$

Having determined the bending moment in inch-pounds from the loads on a girder, the procedure by the chord method is as follows:

Assume the total depth of girder and from this approximate the effective depth d in inches. This can be taken at 2 to 4 inches less than the total depth, depending on the size of flange angles. By dividing the bending moment M by the effective depth d, the flange stress t or c is obtained; and dividing the flange stress by the average unit stress, say 14,500 pounds per square inch, the result is the net area in square inches required for the flange. The sections required to make up this net area can then be determined.

The foregoing computations are expressed by the formula

$$A=\frac{M}{Sd}$$

The average value of the unit stress to be used is proportioned from the extreme fiber stress, 16,000 pounds per square inch. Thus if the effective depth is $\frac{9}{10}$ of the extreme depth, the average unit stress to be used is $\frac{9}{10}$ of 16,000, or 14,400 pounds per square inch.

The result of the first trial is only approximate. From the section thus determined the value of d can be computed and the above operations repeated. This result, which is also approximate if any change is made in the section, is usually accurate enough to be accepted as final. Most specifications permit $\frac{1}{8}$ of the web to be counted in each flange section.

Illustrative Example. Assume M equals 420,000 foot-pounds; total depth of girder 36 inches; approximate value of d equals 33 inches. To find the required section

$$M=420{,}000\times 12=5{,}040{,}000 \text{ in.-lb.}$$

$$A=\frac{5{,}040{,}000}{14{,}500\times 33}=10.53 \text{ sq. in.}$$

$$A=\begin{cases} \frac{1}{8}\text{ web } 36''\times\frac{5}{16}'' & = 1.41 \text{ sq. in.} \\ 2\text{Ls } 6''\times 3\frac{1}{2}''\times\frac{5}{8}''=11.10 & \\ \text{less 1 rivet hole}=1.10 & =\underline{10.00} \end{cases}$$

$$\therefore \qquad A=11.41 \text{ sq. in.}$$

As the area of the chosen section is greater than the calculated value, it is satisfactory.

PROBLEM

Fig. 113 illustrates the plate girder described in the above example. Compute the correct value of *d*. (Note: No account is taken of the part of web plate which is counted as flange section, in computing the position of the c. g. of the flange. Also no account is taken of the rivet holes in the web.) Compute the net flange area required and, if necessary, correct the size of angles.

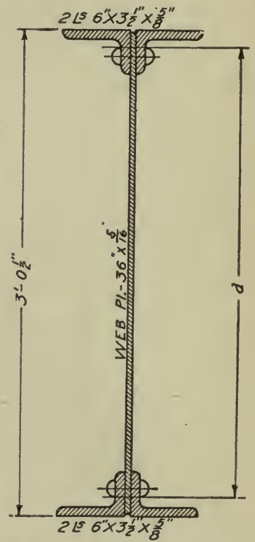


Fig. 113. Section and Details of Plate Girder

The two methods of designing lead to about the same results. No further consideration will be given to the chord method, as the moment of inertia method is preferred.

Calculation of Load Effects. The bending moments and shears are computed in just the same manner for girders as for beams. However, in making a complete design of a riveted girder the bending moment is required for all points along the girder for computing rivet spacing and for determining the length of cover plates, if they are used. Consequently the moment diagram is needed in most cases. (It can be constructed by the methods given in the sections on Bending Moments and Moment Diagrams in "Strength of Materials".)

DESIGN OF PLATE GIRDER

Having computed the bending moments and shears and constructed the diagrams for them, the steps in the design are:

Determine allowable depth
Compute thickness of web
Compute required moment of inertia
Compute flange section which will give required moment of inertia
Determine length of flange plates
Design stiffeners
Design end connection
Compute spacing of rivets for flanges

For illustrating the operations, assume a plate girder as shown in Fig. 114. The span is 45′-0″; load 4000 pounds per lineal foot equals total load of 180,000 pounds; end shear 90,000 pounds;

maximum bending moment 12,150,000 inch-pounds. The shear and moment diagrams are given.

Depth. *Economy.* For any set of conditions governing the design of a plate girder there is a depth which gives the greatest economy of metal. But there are so many conditions entering into the problem that no simple formula can be given for computing it.

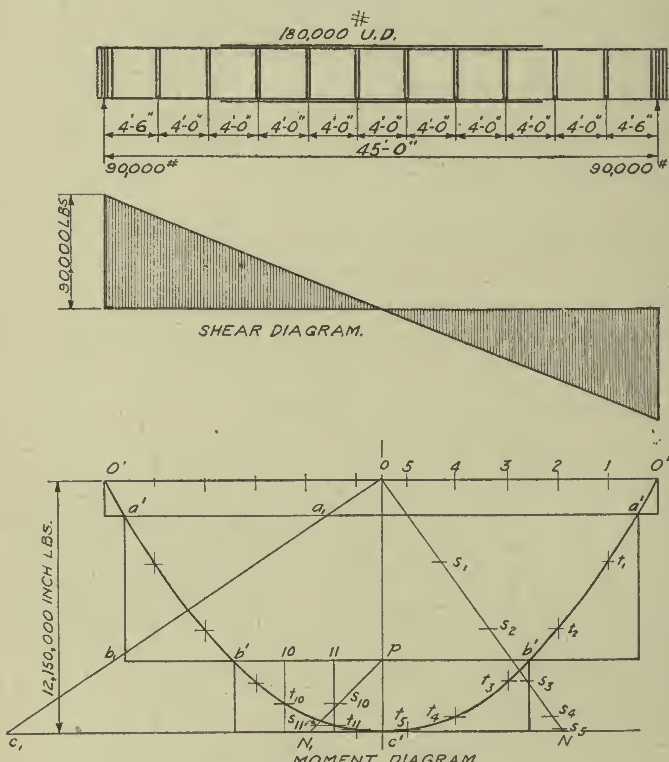


Fig. 114. Plate Girder Design

The effects of some of these conditions can be stated in general terms as follows:

The greater the shear the greater the depth required

The greater the bending moment the greater the depth required

The longer the span the greater the depth required

The thicker the web plate the less the depth

For lateral stiffness shallow depth is better

The smaller the deflection allowed the greater the depth needed

If it is desired to determine the most economical depth for a given case, several depths must be assumed, the designs made, and the cross sections or weights computed. A few trials will lead to the desired result.

The depth of the girder may be as small as $\frac{1}{20}$ of the span and may be as great as $\frac{1}{4}$ the span, but the usual range is $\frac{1}{10}$ to $\frac{1}{6}$. In the absence of any governing feature $\frac{1}{8}$ of the span may be assumed as a suitable depth.

Other Considerations. Usually other considerations than economy will determine the depth. In building construction it is generally desirable to make the girders as shallow as practicable, then the depth may be governed by deflection, by practicable thickness of web or section of flanges, or by details of connections. The final result must be determined by trial designs.

In the example, Fig. 114, assume the depth of web plate to be 48 inches. On account of the fact that the edges of the plate will not be exactly straight (unless they have been planed), it is customary to set the flange angles $\frac{1}{4}$ inch beyond the edge of the plate, making the depth in this case $48\frac{1}{2}$ inches back to back of angles.

Thickness of Web. In building work, $\frac{5}{16}$ inch is a suitable thickness to adopt as the minimum. For exceptional cases when the loads are light $\frac{1}{4}$ inch may be used. Under Unit Stresses, p. 51, the allowable shear on girder webs is given, i. e., 10,000 pounds per square inch. This is the average shear on the net cross section of the web. In the example, Fig. 114, the maximum shear is 90,000 pounds; then the net area of the web must be $\frac{90,000}{10,000}$ or 9.0 square inches. The depth of the web is 48 inches, from which must be deducted 2 rivet holes $\frac{7}{8}$ inch in diameter, making the net depth $46\frac{1}{4}$ inches. The thickness required to give the net cross section is $\frac{9.0}{46.25}$ or 0.19 inches. Hence a plate 0.19 inch thick fulfills the requirements for shear on the web. This is less than the minimum adopted, so the thickness is made $\frac{5}{16}$ inch.

PROBLEM

What thickness of web is required for a shear of 220,000 pounds, depth 44 inches?

Before the thickness of web can be accepted as being satisfactory, it must be known to provide ample bearing for the rivets which connect the flanges to the web. The design of this riveting is explained later. For the present purpose the method used is this: Assume that an amount of stress equal to the maximum vertical shear must be transmitted from the web to each flange within a distance equal to the depth of the web. Applying this to the example, the maximum vertical shear is 90,000 pounds and this amount must be transmitted from web to flange in a distance of 48 inches, which equals the depth of the web. The bearing value of a $\frac{3}{4}$-inch rivet in a $\frac{5}{16}$-inch web is 5860 pounds. The number required is $\frac{90,000}{5860}$ or 16. This number of rivets in a distance of 48 inches gives a spacing of 3 inches, which is satisfactory and requires only one row of rivets. (Two rows could be used, giving space for twice as many rivets as are needed.) Therefore, the web thickness is satisfactory.

Shearing Value of Web Plates. A study of the shearing value of web plates compared with the bearing value of rivets in the web will show that sufficient bearing value can be developed to equal the shearing value. Consequently, the bearing test need not be applied. For a unit shear of 10,000 pounds per square inch and a unit bearing of 25,000 pounds per square inch, it can be shown that two rows of $\frac{3}{4}$-inch rivets, spaced $3\frac{3}{4}$ inches center to center in each row, will have the same bearing value as the shearing value of the plate (no reduction being made in shearing value on account of rivet holes).

PROBLEM

Assume a plate 64 inches deep and $\frac{3}{8}$ inch thick. Prove the foregoing statement.

Moment of Inertia Required. Having the bending moment and the depth of the girder, the value of the required moment of inertia can be computed from the formula, (see p. 78).

$$I = \frac{Mc}{S}$$

In the example, Fig. 114, $M = 12,150,000$ in.-lb.; $S = 16,000\#$. If no flange plates are used, the distance c is measured to the back of the angle, i. e., $24\frac{1}{4}$ inches. Then

$$I = \frac{12,150,000 \times 24\frac{1}{4}}{16,000} = 18,415$$

If it develops that flange plates must be used, the value of the moment of inertia must be increased to correspond to the increased depth.

Flange Section. Having determined the moment of inertia required, it is next necessary to find by trial the section which has this moment of inertia. To avoid tedious figuring, a rough approximation is first made. The web plate being determined, its moment of inertia may be computed or be taken from the handbook.

$$I \text{ for Pl. } 48'' \times \tfrac{5}{16}'' = 2880$$

This amount deducted from 18,415 leaves 15,535 as the net value of I to be supplied by the flanges. The general formula for moment of inertia, p. 38, is

$$I = Ar^2$$

In this case r is about 22.5 inches, then $r^2 = 506$, and $A = \frac{15{,}535}{506}$ or 30.7 square inches. This is the net area of the two flanges. The gross section must be larger to allow for rivet holes; for this add 2.3 square inches, making 33.0 square inches, or 16.5 square inches for each flange. This area may be made up of 2 angles without a plate or of 2 angles with a plate. Both cases are given.

Case A—Without Flange Plates. Without flange plates, use 2Ls $6'' \times 6'' \times \frac{3}{4}''$, having an area of 2×8.44 or 16.88 square inches. For this case the total depth is $48\frac{1}{2}$ inches, as previously determined, and no correction is needed for the required value of I, viz, 18,415. Now compute its value for the approximate section, Fig. 115, making the necessary corrections for rivet holes.

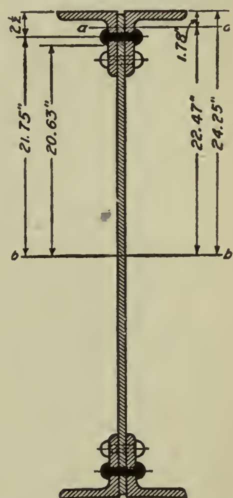


Fig. 115. Section of Plate Girder Without Flange Plates

$$I = \left\{ \begin{array}{lrr}
\text{1 Pl. } 48'' \times \tfrac{5}{16}'' \text{ (from tables)} \ldots\ldots\ldots\ldots\ldots & 2{,}880 & \\
\text{Deduct for holes } 2 \times \tfrac{7}{8} \times \tfrac{5}{16} \times 21.75 \times 21.75 & \underline{260} & 2{,}620 \\
 & & \\
\text{4 Ls } 6 \times 6 \times \tfrac{3}{4} \text{ (from tables) about axis } a\text{-}a & 113 & \\
\quad \text{about axis } b\text{-}b \;\; 4 \times 8.44 \times 22.47 \times 22.47 & \underline{17{,}045} & \\
 & 17{,}158 & \\
\text{Deduct for holes } 4 \times \tfrac{7}{8} \times \tfrac{3}{4} \times 21.75 \times 21.75 & \underline{1{,}241} & \underline{15{,}917} \\
\qquad \text{Total net value of } I & & 18{,}537
\end{array} \right.$$

In deducting for rivet holes, the diameter of hole deducted is $\frac{7}{8}$ inch for a $\frac{3}{4}$-inch rivet. The distance to the holes is taken at the outer of the two rows of holes.

The moment of inertia of the section is somewhat larger than the required amount, therefore the section is satisfactory.

Case B—With Flange Plates. With flange plates it is usually specified that not less than one-half the flange area shall be in the angles, or the largest size of angle shall be used. In this example it has been found that only one row of rivets is necessary for connecting flange to web. For the first trial use 2Ls $6'' \times 4'' \times \frac{5}{8}''$ and 1 Pl. $14'' \times \frac{7}{16}''$. Then the gross area of one flange equals

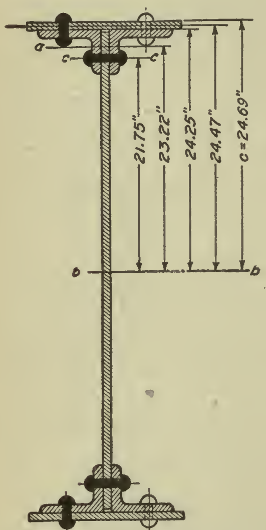


Fig. 116. Section of Plate Girder with Flange Plates

for 2Ls $6'' \times 4'' \times \frac{5}{8}''$	$2 \times 5.86 = 11.72$
for 1 Pl. $14'' \times \frac{7}{16}''$	$= 6.12$
Total area	$= 17.84$

The section is shown in Fig. 116. For this section the value of c is $24.25 + 0.44$ or 24.69. The required value of I must be corrected to correspond:

$$I = \frac{12{,}150{,}000 \times 24.69}{16{,}000} = 18{,}750$$

The value of I computed for the assumed section is

$$I = \left\{ \begin{array}{llll}
\text{1 Pl. } 48'' \times \frac{5}{16}'' & = 2880 & & \\
\text{Deduct for holes } 2 \times \frac{7}{8} \times \frac{5}{16} \times 21.75 \times 21.75 & = \underline{260} & & 2{,}620 \\
\text{4 Ls } 6'' \times 4'' \times \frac{5}{8}'' \text{ about axis } a\text{-}a & = 30 & & \\
\text{about axis } b\text{-}b \; 4 \times 5.86 \times 23.22 \times 23.22 & = \underline{12{,}637} & & \\
 & 12{,}667 & & \\
\qquad \text{Deduct for holes} & & & \\
4 \times \frac{7}{8} \times \frac{5}{8} \times 21.75 \times 21.75 = 1036 & & & \\
4 \times \frac{7}{8} \times \frac{5}{8} \times 23.94 \times 23.94 = \underline{1260} & & \underline{2{,}296} & 10{,}371 \\
\text{2 Pl. } 14'' \times \frac{7}{16}'' \text{ less 2 rivet holes} & & & \\
2 \times 12\frac{1}{4} \times \frac{7}{16} \times 24.47 \times 24.47 & & & = \underline{6{,}418} \\
\qquad \text{Total net value of } I & & & = 19{,}409
\end{array} \right.$$

This value of I is in excess of the required value, the latter being 18,750, hence the section may be reduced. The correction can be made without going through the calculations in detail. The angles need not be changed, but the flange plates may be reduced in thickness. By inspection it can be seen that a reduction of $\frac{1}{16}$ inch in thickness reduces I by $\frac{1}{7}$ of 6418, or 917. The resulting net value of I is $19{,}409-917$ or 18,492. This reduction in the thickness of the flange plate also reduces the required value of I. It now becomes

$$I=\frac{12{,}150{,}000\times 24.63}{16{,}000}=18{,}700.$$

These results are sufficiently close and the reduced section is used although it is somewhat scant.

The revised section is

web plate	$48''\times\frac{5}{16}''$
each flange	$\begin{cases} 2\text{Ls } 6''\times 4''\times\frac{5}{8}'' \\ 1 \text{ Pl. } 14''\times\frac{3}{8}'' \end{cases}$

The sectional areas of the two designs are

Case A.	1 Pl. $48\times\frac{5}{16}$	15.00 sq. in.
	4Ls $6\times 6\times\frac{3}{4}$	33.76 sq. in.
		48.76 sq. in.
Case B.	1 Pl. $48\times\frac{5}{16}$	15.00 sq. in.
	4Ls $6\times 4\times\frac{5}{8}$	23.44 sq. in.
	2 Pl. $14\times\frac{3}{8}$	10.50 sq. in.
		48.94 sq. in.

This showing is slightly in favor of Case A, but it is more favorable to Case B when it is considered that the flange plates do not extend the full length of the girder. Case B also has the advantage of greater lateral stiffness due to its greater width. On the other hand the cost of the additional riveting may amount to more than the saving in weight. Also the use of the flange plates, taking into account the rivet heads, increases the over-all depth about two inches, which may be objectionable in some cases. In general, the design without flange plates is preferred.

Width of Flange Plates. The width of a flange plate is limited by the permissible projection beyond the outer row of rivets. The limits are eight times the thickness of the plate, or a maximum of six inches. In the above example this limit is $8 \times \frac{3}{8}''$ or $3''$. This permits a distance of 8 inches between the gage lines, which is satisfactory.

The customary widths of flange plates vary by 2 inches, thus, 10-inch, 12-inch, 14-inch, etc. For 6-inch flange angles the maxi-

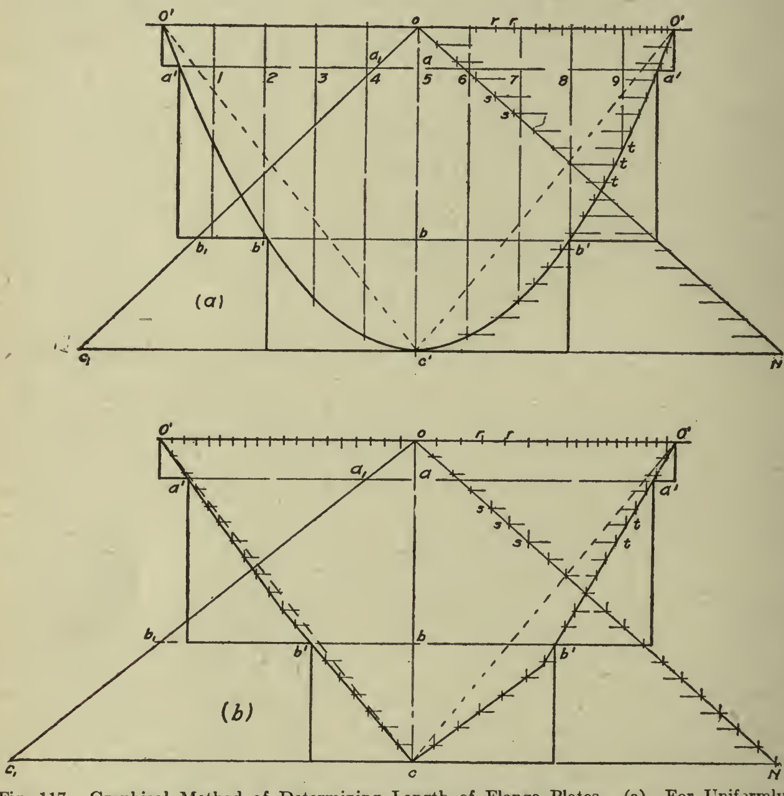


Fig. 117. Graphical Method of Determining Length of Flange Plates. (a) For Uniformly Distributed Loads; (b) For Concentrated Loads

mum width is 20 inches, and for 8-inch angles, 24 inches, but 18 and 20 inches, respectively, are preferable, and 14 inches and 18 inches are most used. When more than one plate is used on a flange, usually the outer one is made less in thickness than the inner one.

Length of Flange Plates. The flange section which has just been computed is the section required at the place of maximum bending moment. The bending moment decreases toward the ends, as shown in the moment diagram Fig. 114, and, if it were practicable to do so, the flanges might be decreased correspondingly. It is necessary for practical reasons to extend the flange angles the full length of the girder but the flange plates can be stopped at the points where they are no longer needed. The plate ceases to be needed at the point where the bending moment equals the resisting moment of the web plate and flange angles. This can be computed by the methods and from the data already given, but the process is tedious and the results can be obtained more easily by graphical methods with sufficient accuracy.

Graphical Solution for Uniformly Distributed Loads. Let Fig. 117-a represent the moment diagram for any uniformly distributed load. The lines at *1, 2, 3,* etc., represent the amount of the bending moment at the several points along the girder. The maximum bending moment is at *5.* The resisting moment is represented by the line $o\,c'$. This line is divided into three parts, $o\,a$ representing the resisting moment of the web plate, $a\,b$ the resisting moment of the flange angles, and $b\,c'$ the resisting moment of the flange plates. Then the distance $a'a'$ equals the theoretical length of the flange angles, but practically they are made the full length of the girder, and $b'b'$ equals the theoretical length of the flange plates. If more than one plate is used on each flange, additional divisions may be made of the line oc', and the lengths determined in the same manner.

If the resisting moments of the several parts of the flanges have not been computed, their moments of inertia may be used for this purpose in the following manner. On the edge of a sheet of paper or on a scale lay off at any convenient scale $o\,a_1$, a_1b_1, and b_1c_1 equal, respectively, to the values of I for the web plate, flange angles, and flange plates. Hold the zero point at o and swing the paper or scale to the position where c_1 falls on the horizontal line through the apex of the moment diagram c'. Then the horizontal lines through a_1 and b_1 will cut the diagram at $a'\,a'$ and $b'\,b'$ and give the lengths of flange plates required.

Graphical Solution for Concentrated Loads. Fig. 117-b represents a moment diagram for concentrated loads. The same explanations and procedure apply as for uniformly distributed loads.

Taking the girder section determined for Case B, p. 142, the length of its flange plates can be determined by the method just described, using the moment diagram in Fig. 114. The values of I as computed on p. 143, are

for web plate........................	2,620
for flange angles.....................	10,371
for flange plates.....................	5,501
	18,492

Using a convenient scale lay off $o\,c_1$ equals 18,492, so that c_1 falls on the horizontal line through c'. Then divide $o\,c_1$ at a_1 and b_1 so that $o\,a_1 = 2620$, $a_1\,b_1 = 10{,}371$, and $b_1\,c_1 = 5501$. Draw horizontal lines through a_1 and b_1, cutting the moment diagram at $a'\,a'$ and $b'\,b'$. Then $a'\,a'$ and $b'\,b'$ represent the theoretical lengths of the flange angles and the flange plates, respectively. As previously stated, the flange angles always extend the full length of the girder. The flange plates are usually made two or three feet longer than theoretically required. In this case the length $b'\,b'$ is 23′-6″ (approx.); the plates are made 26′-0″ long. This extra length is used so that some stress can be developed in the plate at the points $b'\,b'$.

Web Stiffeners. Schneider's Specifications* provide "The web shall have stiffeners at the ends and inner edges of bearing plates, and at all points of concentrated loads, and also at intermediate points, when the thickness of the web is less than one-sixtieth of the unsupported distance between flange angles, generally not farther apart than the depth of the full web plate, with a minimum limit of 5 feet."

The theory of stresses concerned in the design of stiffeners is too complicated for consideration in this text, but some simple rules can be established which will lead to safe construction. Web stiffeners may be divided into two distinct classes: (1) stiffeners at loaded points and (2) intermediate stiffeners.

Stiffeners at Loaded Points. The chief purpose of stiffeners at loaded points is to transmit the loads to the girder web. According to the theory of stresses in girders, the load must be applied to the web and produce shear therein from which tension and compression are produced in the flanges. It is, therefore, necessary to carry the applied loads into the web plate as directly as possible. If the load is uniformly distributed on either the top or bottom flange, it is

*"The Structural Design of Buildings" by C. C. Schneider, M. Am. Soc. C. E., *Transactions American Society of Civil Engineers*, Vol. LIV, p. 495.

transmitted to the web by the rivets connecting the flange angles to the web. The effect of this load on the number of rivets required is considered later in the text.

When concentrated loads are applied, enough rivets cannot be placed in the flanges to transmit the load to the web, and also it is desirable that the load be applied throughout the depth of the web plate. To meet these conditions stiffener angles are used. These

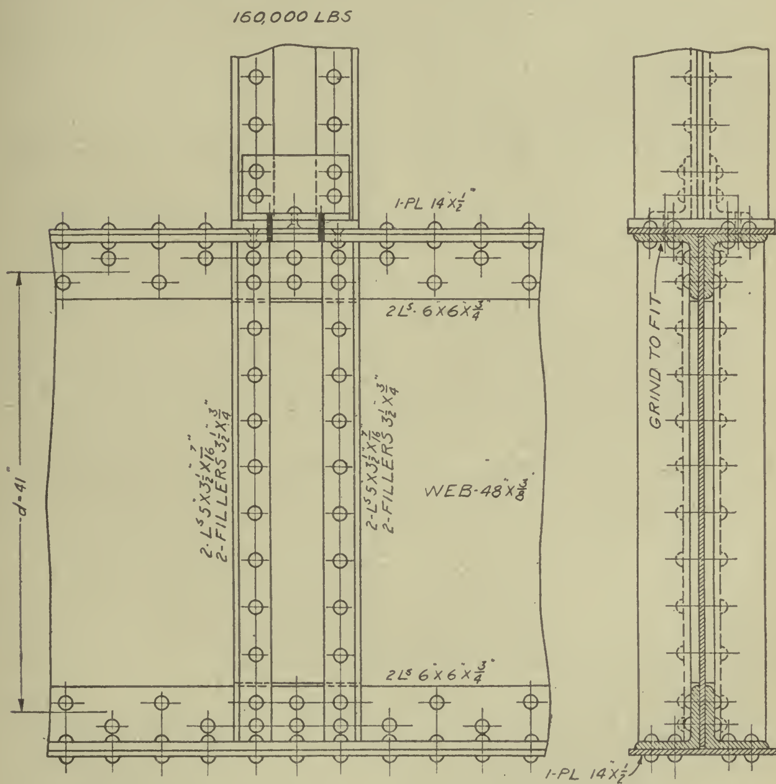


Fig. 118. Details of Girder Showing Use of Stiffeners Under Concentrated Load

stiffeners may be designed as short compression members using a unit stress of 12,000 pounds per square inch. They must be attached to the web plate with enough rivets to transmit the load. Generally the bearing value of rivets in the web plate will govern.

As an example, assume that a girder supports a concentrated load of 160,000 pounds, Fig. 118. On account of the width of bearing of the load, it is desirable to use two pairs of stiffeners. The area required is $\frac{160{,}000}{12{,}000}$ or 13.33 square inches. 4 Ls $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$ —area 4×3.53 or 14.12 square inches—provide the necessary sectional area. The thickness of the girder web being $\frac{3}{8}$ inch, the bearing value of a $\frac{3}{4}$-inch rivet is 7030 pounds. Then the number of rivets required is $\frac{160{,}000}{7030}$ or 23. There is ample space for this number of rivets.

The condition at the end bearing of a plate girder is analogous to that described for a concentrated load and is treated in the same manner. If the end of the girder connects to a column or another girder by means of web angles, the design is made in the same manner as for the web connection of I-beams.

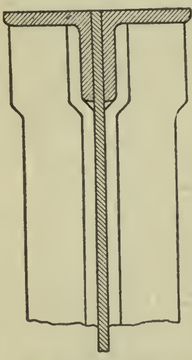


Fig. 119. Crimped Stiffeners

Intermediate Stiffeners. Intermediate stiffeners are used to prevent buckling of the web plate. According to the specifications quoted above, stiffeners must be used if the unsupported depth of plate is more than 60 times its thickness. Such stiffeners are to be spaced not farther than the depth of the girder, or for deep girders not more than 5 feet. Applying this to the girder illustrated in Fig. 118, it is found that stiffeners are required, for the unsupported depth is 36 inches, while 60 times the thickness $\frac{3}{8}$ inch is $22\frac{1}{2}$ inches. The depth of the girder is 4 feet, so the stiffeners are spaced 4 feet.

Stiffeners at loaded points serve incidentally to stiffen the web and are taken into account in spacing the intermediate stiffeners. Intermediate stiffeners are usually angles in pairs. The leg of the angle parallel to the web plate need be only wide enough for riveting, say 3 inches, as it adds but little to the lateral stiffness. The outstanding leg must be determined arbitrarily. For a 30-inch girder, 3 inches may be used; and for a 90-inch girder, 6 inches; and others in proportion. The thickness should be consistent with the size of the angle and not less than the thickness of the web plate; and the width of the outstanding leg should be somewhat less than the outstanding leg of the flange angles.

Stiffeners at loaded points must be ground to fit accurately against the loaded flange; intermediate stiffeners need not be so carefully fitted. The use of fillers under stiffener angles is not necessary, but a better fit can be obtained when they are used. This makes it desirable to use them at loaded points and end bearings. Where fillers are not used, the stiffener angles must be crimped to fit the flange angles, Fig. 119. There is little difference in cost, as the expense of crimping offsets the cost of the filler plates.

Refer to the girder in Fig. 114. There being no concentrated loads, *stiffeners at loaded points* are required only at the end bearings. The reaction at each end is 90,000 pounds. The area of stiffener angles required is $\frac{90{,}000}{12{,}000}$ or 7.5 square inch. 4 Ls $3\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$ have sufficient area, but it is desirable to have them approximately as wide as the flange angles, so 4 Ls $5'' \times 3'' \times \frac{5}{16}''$ are used. Sixteen rivets are required. There is ample space for them.

The web plate is $\frac{5}{16}$ inch thick and has an unsupported depth of 36 inches, hence it requires *intermediate stiffeners*. These are spaced about 4 feet apart (equal to the depth of the girder). Angles $4'' \times 3'' \times \frac{5}{16}''$ may be used for these stiffeners.

Rivets Connecting Flange Angles to Web. In order to make the several pieces of the plate girder act as a unit, they must be rigidly connected. It is evident that if the angles and plates were simply placed in their relative positions without being riveted, they would not co-operate but would tend to act independently. This is explained under Horizontal Shear in "Strength of Materials," Part II.

Number of Rivets. The loads on the girder are applied either directly or indirectly to the web, producing vertical shear. By flexure, the vertical shear produces horizontal shear, which becomes tension and compression in fibers below and above the neutral axis, respectively. Most of these stresses occur in the flange plates and angles and must be transmitted to them from the web by the rivets which connect the angles to the web plate. There must be enough rivets to transmit the whole amount of the stress and they must be located at the points where the stress should pass from the web to the flanges. Then in each flange there must be such a number of rivets between the point of maximum flange stress (maximum moment) and each end to transmit the total flange stress; or, stated in other terms, the resisting moment of the rivets between the point of maximum bending moment and each end must equal the maximum bending moment, and this equals the resisting moment of the girder section.

In Fig. 118, let d be the average distance between the rivets in the top and bottom flanges; k the bearing value of one rivet (usually bearing in the web plate); M the bending moment in inch-pounds; and N the number of rivets in one end of one flange. Then $k \times d$ equals the resisting moment of one pair of rivets in inch-pounds and $N = \frac{M}{k \times d}$ equals the number of pairs or the number of rivets in each flange from the center or point of maximum bending moment to either end. For example, assume the following data:

$M = 450{,}000$ ft.-lb. $= 5{,}400{,}000$ in.-lb.
$k = 7030\#$, bearing value of a $\frac{3}{4}$-inch rivet in a $\frac{3}{8}$-inch web
$d = 41''$

then
$$N = \frac{5{,}400{,}000}{7030 \times 41} = 19 \text{ rivets}$$

Rivet Spacing in Flanges. If the rivets, Fig. 117-a and -b, were spaced uniformly, their resisting moment would be represented by the moment diagram $o'\,c'\,o'$, whereas, the bending moment diagram is $o'\,a'\,b'\,c'\,b'\,a'\,o'$. From this it is clear that the resisting moment of the rivets is less than the bending moment at all points except at the maximum. But these rivets can be so spaced that the two moment diagrams will coincide. To determine this spacing proceed as follows: Lay off $o\,N$ equal to the total value of the number of rivets, say 19, and divide it into 19 spaces at the points s. Through the points s, draw horizontal lines intersecting the moment diagram at points t. Through the points t, draw vertical lines intersecting the base line at the points r. Then the points r are the locations of the rivets.

It is important to note that the rivets are closer together near the ends, i. e., where the bending moment is changing rapidly. On the left side of Fig. 117-b, the spaces are nearly equal because this side of the moment diagram is nearly a straight line. There is a change of spacing wherever there is a change in direction of the moment diagram. For the uniform load, Fig. 117-a, there is a change in each space. Of course it is not practicable to space the rivets strictly in accordance with the theory. The practical method is to divide the girder into sections, usually taking the divisions formed by the stiffeners, and space the rivets equally in each division.

In the problem, Fig. 114, use the following data:

$M = 12{,}150{,}000$ in.-lb.
$k = 5860\#$, bearing value of a $\frac{3}{4}$-inch rivet in a $\frac{5}{16}$-inch web
$d = 41.26''$ (Case A, Fig. 115)

then
$$N = \frac{12{,}150{,}000}{5860 \times 41.26} = 50 \text{ rivets}$$

Lay off $o\,N$ equals 50. Along $o\,o'$ lay off the points *1*, *2*, *3*, etc., marking the positions of the stiffeners. Through these points draw verticals intersecting the moment diagram at t_1, t_2, etc.; thence draw horizontals intersecting $o\ N$ at s_1, s_2, s_3, etc. Then $o\,s_1$ represents the number of rivets between o and *1*; $s_1\,s_2$, the number between *1* and *2*; $s_2\,s_3$ the number between *2* and *3*; etc.

$o\,s_1$ represents 17 rivets; the distance o'-1 is 54 inches; space the rivets 3 inches center to center.

$s_1\,s_2$ represents 14 rivets; the distance *1-2* is 48 inches; space the rivets $3\frac{1}{2}$ inches center to center.

$s_2 s_3$ represents 10 rivets; the distance *2-3* is 48 inches; space the rivets $4\frac{1}{2}$ inches center to center.

$s_3 s_4$ represents 7 rivets; the distance *3-4* is 48 inches; space the rivets 6 inches center to center, and this being the maximum spacing allowed, continue it to the center of the span.

If Case B be used, the procedure is just the same. The value of d would be larger (Fig. 116) and, consequently, the number of rivets smaller.

Riveting for Cover Plates. In Case B there must also be determined the necessary riveting for attaching the cover plates to the flange angles. The procedure is similar to that just given. In Fig. 114, $p\,c'$ represents the resisting moment of the cover plates and, therefore, the required resisting moment of the rivets. The rivets are in single shear, and the moment arm is the distance back to back of flange angles. Use the following data:

$M = 3{,}600{,}000$ in.-lb. (approx.).
$k = 4420\#$, single shearing value of a $\frac{3}{4}$-inch rivet.
$d = 48\frac{1}{2}''$ (Fig. 116)

Then
$$N = \frac{3{,}600{,}000}{48\frac{1}{2} \times 4420} = 17 \text{ rivets}$$

Lay off $p\ N_1$ equals 17. Along $b'\ b'$ lay off the points *10, 11*, etc., at intervals of say 4 feet. Draw verticals to t_{10}, t_{11}, etc., and horizontals to s_{10}, s_{11}, etc.

$p\ s_{10}$ represents 9 rivets; the distance *b′-10* is 48 inches. There are two rows of rivets in the flange plate, so there are $4\frac{1}{2}$ rivets required in one row in 48 inches, i. e., spaced about 10 inches, center to center. But the maximum allowable spacing is 6 inches, center to center, and this is used throughout the length of the cover plates except at the ends where a spacing of 4 inches for a distance of two feet is adopted arbitrarily.

Rivet Spacing Computed from Web Bearing. The method, p. 140, for checking the thickness of the web plate for rivet bearing may be used for determining the rivet spacing; for example, assume that an amount of stress equal to the vertical shear must be transmitted from the web to each flange within a distance equal to the depth of the web. Then the number of rivets required in this distance is determined by dividing the vertical shear by the bearing value of one rivet.

Referring to Fig. 114 and applying this method:

Shear at $0' = 90{,}000\#$
No. of rivets in 48″ $\dfrac{90{,}000}{5860} = 16$, spacing about 3″

Shear at $1 = 72{,}000\#$
No. of rivets in 48″ $\dfrac{72{,}000}{5860} = 13$, spacing about $3\frac{1}{2}$″

Shear at $2 = 56{,}000\#$
No. of rivets in 48″ $\dfrac{56{,}000}{5860} = 10$, spacing about $4\frac{1}{2}$″

Shear at $3 = 40{,}000\#$
No. of rivets in 48″ $\dfrac{40{,}000}{5860} = 7$, spacing about 6″

Spacing when Load Transmitted through Flange Rivets into Web. If the load on the girder is applied in such a way that it must be transmitted through the flange rivets into the web, then the rivet spacing must take this into account. The exact method of doing so is difficult to apply, but safe results can be obtained by simply adding enough rivets to transmit the load to the web. Thus in Fig. 114 it has been determined that 17 rivets are required between *o′-1*. The load on this space is 18,000 pounds, which requires 4 rivets to transmit it into the web plate. Then the total number of rivets is 21 and the spacing $2\frac{1}{2}$ inches.

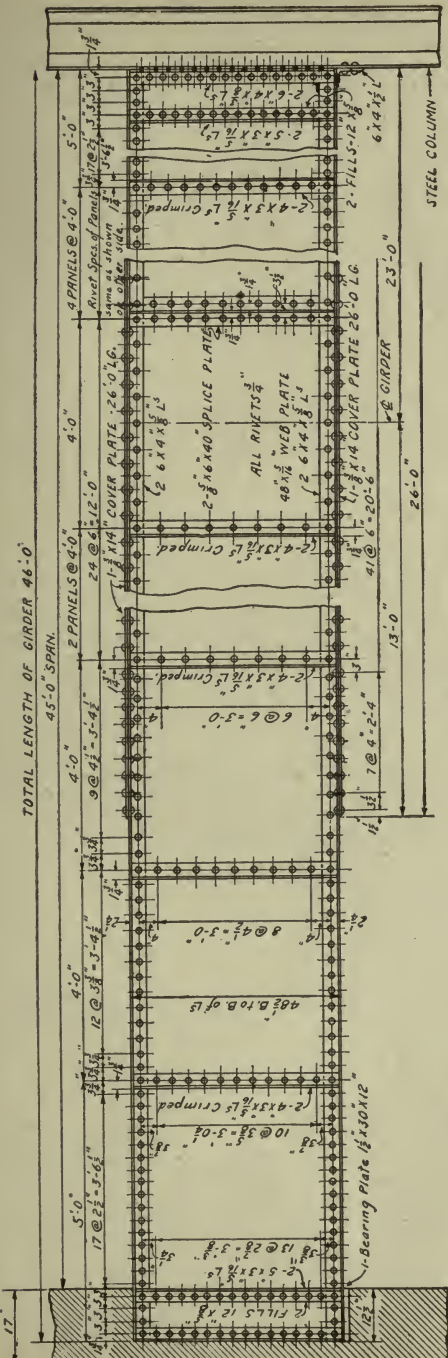


Fig. 120. Design Drawing of Plate Girder

Assuming that the load is applied on the top flange, the extra rivets are required only in that flange. But in practice the riveting is usually made the same in both flanges. Where stiffeners are used at loaded points, the extra rivets are not required.

The actual location and spacing of the rivets must be worked out in making the shop details in order to afford necessary clearances from stiffeners and to suit any other conditions that may apply to the case. It is sufficient for the designer to indicate the spacing as it has been computed above.

Fig. 120 shows the design drawing for the girder developed in the preceding pages, using Case B, that is, a girder with flange plates.

PROBLEMS

1. Design a plate girder from the data given in Fig. 121. Make the design drawing at ¾-inch scale.

2. Design a plate girder having the same span as the one in Fig. 121, but supporting only one-half the load there specified.

Tables and Diagrams. A number of tables have been published giving strength and properties of plate girders. These tables

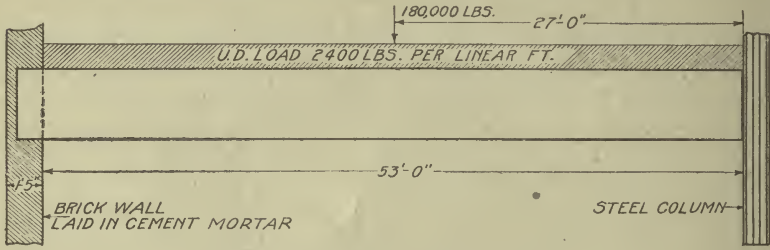


Fig. 121. Data for a Plate Girder Design

are of much assistance in arriving at the approximate section of the required girder, but usually the final design must be computed in detail, as in the foregoing example.

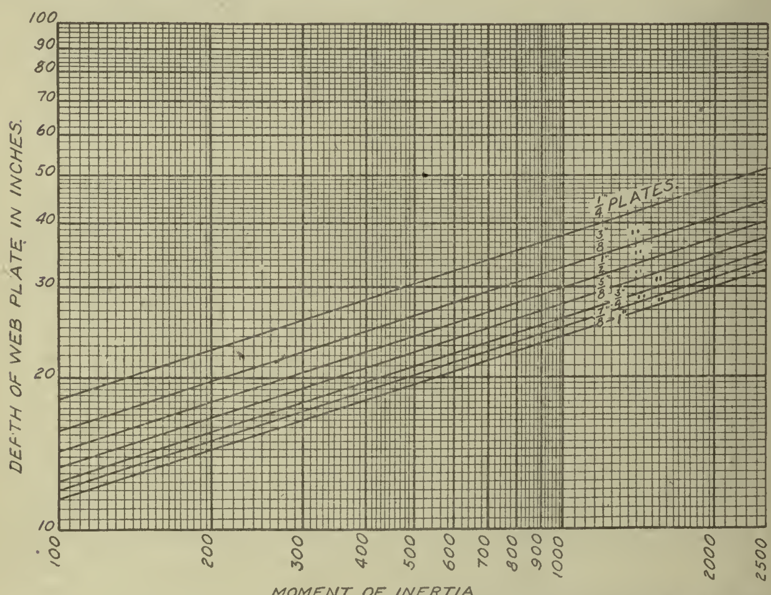
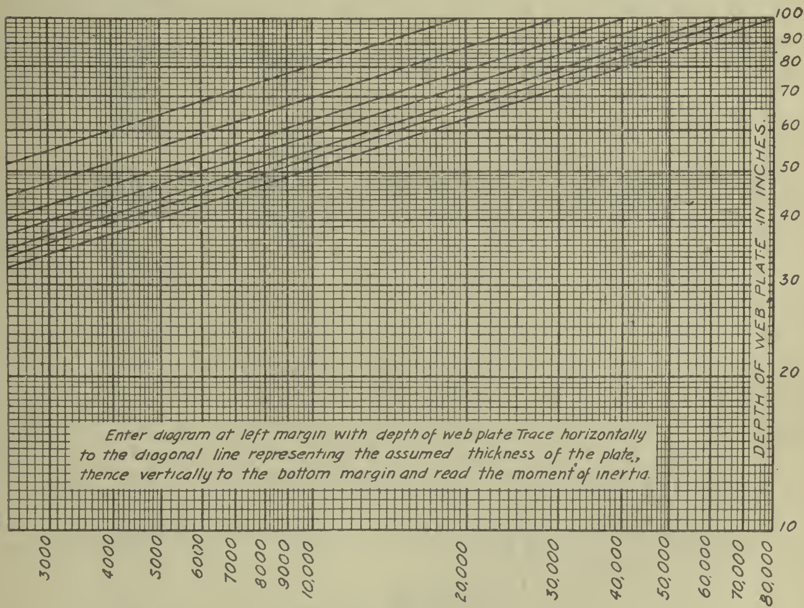


Fig. 122. Diagram for Determining Moments After Deducting Rivet Holes. 2 Holes, ⅞" (¾" Rivets) for ¼"

The large number of plate girder sections that it is possible to make up from the available sizes of web plates, flange angles, and flange plates makes it impracticable to have complete tables of them. The Carnegie Pocket Companion, 1913 edition, contains a valuable table giving the section modulus for a large number of riveted girders.

The handbooks give tables of the moment of inertia of rectangles from which can be taken the value of I for the web plate (from this value must be deducted the value of I for rivet holes). Other reference books give the values of I for web plates with rivet holes deducted and for many sizes of flange angles placed at various depths; similar tables are given for flange plates. By the use of these tables, the value of I for the complete girder section can be found by adding together the values for the web plate, flange angles, and flange plates.*

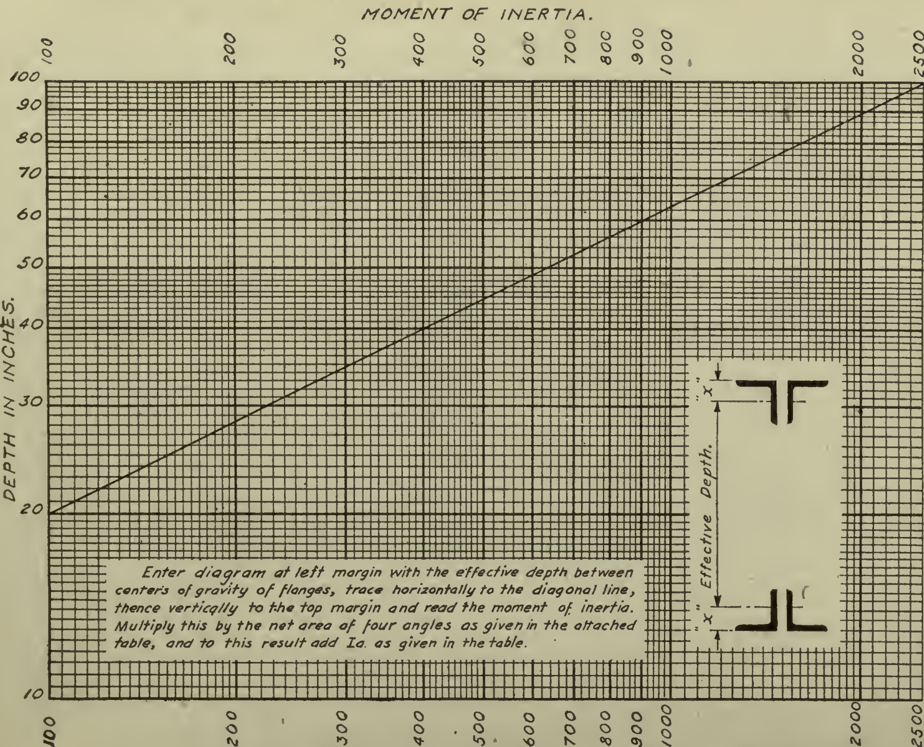
The diagrams, Figs. 122, 123, and 124, give respectively, the values of I for web plates, flange angles, and flange plates. They give the moments of inertia for the sizes of plates and angles most



of Inertia of Web Plates of Plate Girders
to $\frac{1}{2}''$ Plates; 2 Holes, 1'' ($\frac{7}{8}''$ Rivets) for $\frac{5}{8}''$ to 1'' Plates

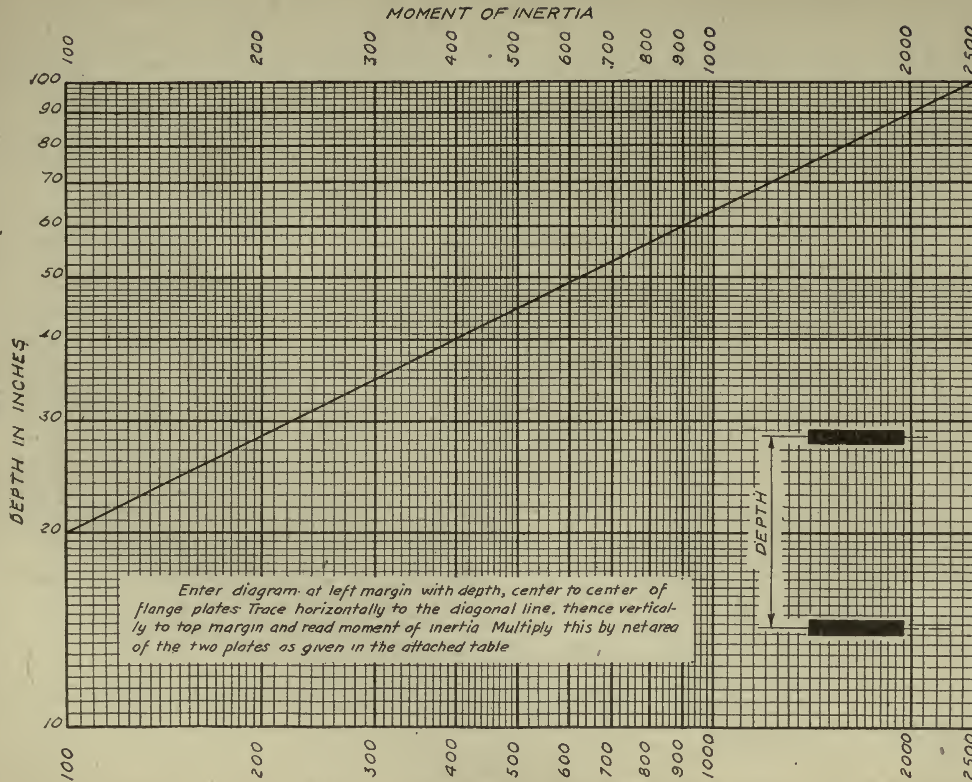
commonly used for plate girders. Values for intermediate sizes of plates and thicknesses of angles can be interpolated. Although not

*"Godfrey's Tables" by Edward Godfrey, M. Am. Soc. C. E.
"Civil Engineer's Pocketbook" by Albert I. Frye, S. B., M. Am. Soc. C. E.



Size of Angles.	Net area of 4 Ls deducting 1-3/4 rivet in each L	Net area of 4 Ls deducting 1-7/8 rivet in each L	Net area of 4 Ls deducting 2-3/4 rivets in each L	Net area of 4 Ls deducting 2-7/8 rivets in each L	Long leg vertical.		Short leg vertical.	
" "					2X	Ia for 4Ls	2X	Ia for 4Ls
4Ls 8X8X1"		56.00		52.00	4.38	356		
" " 8X8X7/8				45.96	4.46	318		
" " 8X8X3/4	43.14.	42.76	40.56	39.76	4.56	279		
" " 8X8X5/8	36.25		34.06		4.64	238		
" " 8X8X1/2	29.25		27.50		4.74	195		
4Ls 8X6X1"		48.00		44.00	5.30	323	3.30	155.
" " 8X6X7/8		42.42		38.92	5.22	289	3.22	139.
" " 8X6X3/4	37.14	36.76	34.51	33.76	5.12	254	3.12	123.
" " 8X6X5/8	31.25		29.06		5.04	216	3.04	105.
" " 8X6X1/2	25.25		23.50		4.94	177	2.94	87.
4Ls 6X6X1"		40.00		36.00	3.72	142		
" " 6X6X7/8		35.58		32.08	3.64	128		
" " 6X6X3/4	31.14	30.76	28.51	27.76	3.56	113		
" " 6X6X5/8	26.25		24.26		3.46	97		
" " 6X6X1/2	21.25		19.50		3.36	80		
" " 6X6X3/8	16.13		14.82		3.28	62		
4Ls 6X4X1"		32.00		28.00	4.34	123	2.34	43.
" " 6X4X7/8		28.46		24.96	4.24	111	2.24	39.
" " 6X4X3/4	25.14	24.76	22.51	21.76	4.16	98	2.16	35.
" " 6X4X5/8	21.25		19.06		4.06	84	2.06	30.
" " 6X4X1/2	17.25		15.50		3.98	70	1.98	25.
" " 6X4X3/8	13.13		11.82		3.88	54	1.88	20.
4Ls 5X3½X7/8		23.18		19.68	3.58	63	2.08	25.
" " 5X3½X3/4	20.62	20.24	17.99	17.24	3.50	55	2.00	22.
" " 5X3½X5/8	17.49		15.30		3.40	48	1.90	19.
" " 5X3½X1/2	14.25		12.50		3.32	40	1.82	16.
" " 5X3½X3/8	10.89		9.58		3.22	31	1.72	13.
4Ls 4X4X3/4	19.14	18.76			2.54	31		
" " 4X4X5/8	16.25				2.46	27		
" " 4X4X1/2	13.25				2.36	22		
" " 4X4X3/8	10.13				2.28	17		
4Ls 4X3X3/4	11.25	15.76			284	28	1.84	13
" " 4X3X5/8	9.25				2.74	24	1.76	11
" " 4X3X1/2	11.25				2.66	20	1.66	10
" " 4X3X3/8	8.61				2.56	16	1.56	8
4Ls 3½X2½X5/8	11.25				2.50	15	1.50	64
" " 3½X2½X1/2	9.25				2.40	13	1.40	5.4
" " 3½X2½X3/8	7.13				2.32	10	1.32	4.4

Fig. 123. Diagram for Determining Moments of Inertia of Flange Angles of Plate Girders



Size of Plates.	Net Area of 2 Plates deducting for 2-¾" Rivets.	Net Area of 2 Plates deducting for 2-⅞" Rivets.
2 Plts. - 10"X1"		16.00
" " 10"X⅞"		14.00
" " 10"X¾"	12.38	12.00
" " 10"X⅝"	10.32	10.00
" " 10"X½"	8.26	8.00
" " 10"X⅜"	6.18	6.00
2 Plts. 12"X1"		20.00
" " 12"X⅞"		17.50
" " 12"X¾"	15.38	15.00
" " 12"X⅝"	12.82	12.50
" " 12"X½"	10.26	10.00
" " 12"X⅜"	7.68	7.50
2 Plts. 14"X1"		24.00
" " 14"X⅞"		21.00
" " 14"X¾"	18.38	18.00
" " 14"X⅝"	15.32	15.00
" " 14"X½"	12.26	12.00
" " 14"X⅜"	9.18	9.00
2 Plts. 16"X1"		28.00
" " 16"X⅞"		24.50
" 16"X¾"	21.39	21.00
" " 16"X⅝"	17.82	17.50
" " 16"X½"	14.25	14.00
" " 16"X⅜"	10.68	10.50
2 Plts 18"X1"		32.00
" " 18"X⅞"		28.00
" " 18"X¾"	24.40	24.00
" " 18"X⅝"	20.30	20.00
" " 18"X½"	16.25	16.00
" " 18"X⅜"	12.20	12.00
2 Plts 20"X1"		36.00
" " 20"X⅞"		31.50
" " 20"X¾"	27.38	27.00
" " 20"X⅝"	22.81	22.50
" " 20"X½"	18.25	18.00
" " 20"X⅜"	13.69	13.50

Fig. 124. Diagram for Determining Moments of Inertia of Flange Plates of Plate Girders

mathematically exact, the results obtained from these diagrams are accurate enough for designing, and will lead to the selection of the same sections as would be determined by computation.

The tables and diagrams give only the sections to be used for the girder. The flange plate length, stiffeners, end connections, and rivet spacing must still be designed by the methods heretofore explained. In many cases, these latter items are left to the detailers; but they are properly a part of the design and should be worked out at the same time the girder section is determined, as the detailer is not likely to have as clear an understanding of the conditions as the designer.

PROBLEM

Check the girder sections in Figs. 115 and 116 by means of the diagrams in Figs. 122, 123, and 124.

OTHER FORMS OF RIVETED GIRDERS

The discussion and examples thus far have dealt with the plate girder. The principles and the methods involved are the same for all forms of riveted girders.

I-Beams with Flange Plates. A form of girder, Fig. 111-a, is used when shallow girders are required and the I-beams are not strong enough. This often occurs in joists and girders of a floor when it is desired to maintain approximately the same depth for members which carry heavy and light loads.

Moment of Inertia. To determine the moment of inertia of the girder, take from the handbook the value of I for the beam and deduct therefrom the value of I for the holes in the flanges; add to this net value for the beam, the value of I for the net section of the flange plates. For example compute the moment of inertia for 15″ I 42# and 2 Pl. 8″×$\frac{3}{4}$″.

I for 15″ I 42#	442	
deduct for 4 rivet holes		
$4\times\frac{7}{8}\times\frac{5}{8}\times7.2\times7.2$	114	
		328
for 2 Pl. 8″×$\frac{3}{4}$″ after deducting rivet holes		
$2\times6\frac{1}{4}\times\frac{3}{4}\times7.9\times7.9$		585
Total value of I		913

Note that two rivet holes are deducted from each flange and from each plate. If the rivet holes are carefully staggered, only one-half

TABLE IV

Moments of Inertia of I-Beams with Holes in Flanges

(Holes for ¾″ rivets computed ⅞″ diam.)

SECTION	MOMENTS OF INERTIA			Grip, or Thickness of Metal at Hole
	Whole	1 Hole Out of Each Flange	2 Holes Out of Each Flange	
27″I 83#	2888.6	2623.0	2357.4	.89
24″I 100#	2380.3	2149.0	1917.7	1.00
24″I 80#	2087.9	1884.9	1681.9	.87
24″I 69½#	1928.0	1734.3	1540.6	.82
21″I 57½#	1227.5	1090.0	952.5	.74
20″I 80#	1466.5	1320.0	1173.5	.92
20″I 65#	1169.6	1042.4	915.2	.79
18″I 75#	1141.3	1026.8	911.3	.90
18″I 55#	795.6	704.9	614.2	.69
18″I 36#	733.2	645.2	557.2	.67
15″I 80#	795.5	706.6	617.7	1.03
15″I 60#	609.0	536.6	464.2	.82
15″I 42#	441.7	385.3	328.9	.62
15″I 36#	405.1	351.8	298.5	.59
12″I 40#	268.9	231.3	193.7	.66
12″I 31½#	215.8	184.5	153.2	.545
12″I 27½#	199.6	169.9	140.2	.51
10″I 25#	122.1	102.7	83.3	.49
9″I 21#	84.9	70.2	55.5	.46
8″I 18#	56.9	46.1	35.3	.425

of this number need be deducted. The shearing value of the web must be investigated and the length of flange plates and rivet spacing computed in the same manner as for plate girders.

PROBLEMS

1. What is the resisting moment of a girder made of one 18″ I 55# and two flange plates 8″×¾″?

2. A beam has a span of 24 feet and supports a u. d. load of 80,000 pounds. Design the beam using a 18″ I 55# with flange plates. Determine length of plates and rivet spacing.

3. What is the resisting moment of a 20″ I 65# with two ⅞-inch holes in each flange? (Note the great loss of strength due to punching holes in the flanges.)

The moments of inertia of I-beams with holes in the flanges are given in Table IV and of flange plates in the diagram, Fig. 123.

Beam Box Girders. Beam box girders, Fig. 111-d, are designed in just the same way as single I-beams with flange plates. They are not economical and should be used only when the available depth prevents the use of a deeper girder. The handbooks give tables of strength of this form of riveted girders.

PROBLEMS

1. Compute the moment of inertia of a girder made of two I-beams 24″×80# and two plates 18″×$\frac{5}{8}$″.

2. Design a beam box girder to support a load of 300,000 pounds at the middle of a 30-foot span. Use 24-inch beams.

3. What is the resisting moment of a girder made of two 15″ [s 33# and two plates 14″×½″?

Plate Box Girders. The plate box girder, Fig. 111-c, needs no explanation as to the method of design, requiring the same procedure as the plate girder. It is used for very heavy loads when the depth allowed is greater than the deepest I-beam but not sufficient to permit the use of a girder with a single web. It is to be noted that the rivets connecting the flange angles to the webs are in single shear, hence the shearing value rather than the bearing value of the rivets will be used in computing rivet spacing.

PROBLEM

Compute the moment of inertia of a girder made of two web plates 36″×½″, four angles 6″×6″×$\frac{3}{4}$″, two flange plates 22″×$\frac{3}{4}$″, and two flange plates 22″×½″.

Unsymmetrical Sections. Thus far in the discussion of riveted girders the sections considered have been symmetrical about the neutral axis and, therefore, the neutral axis has been at mid-depth. It sometimes happens that the two flanges cannot be the same. This makes the computation of the moment of inertia more difficult. Having made the first approximation of the section, it is necessary to find the center of gravity of the assumed section, p. 35, and then the moment of inertia about the neutral axis (through the center of gravity), p. 36.

The common examples of unsymmetrical sections are crane girders, I-beam lintels with one flange plate, girders requiring extra lateral stiffness on account of unsupported top flange, and I-beams with rivet holes in the tension flange at the place of maximum bending moment.

In designing such girders the flanges are made as nearly equal as practicable, so that the neutral axis may be near mid-depth. Of course this cannot be done when a single flange plate is used on an I-beam. With the exception noted above, viz, locating the

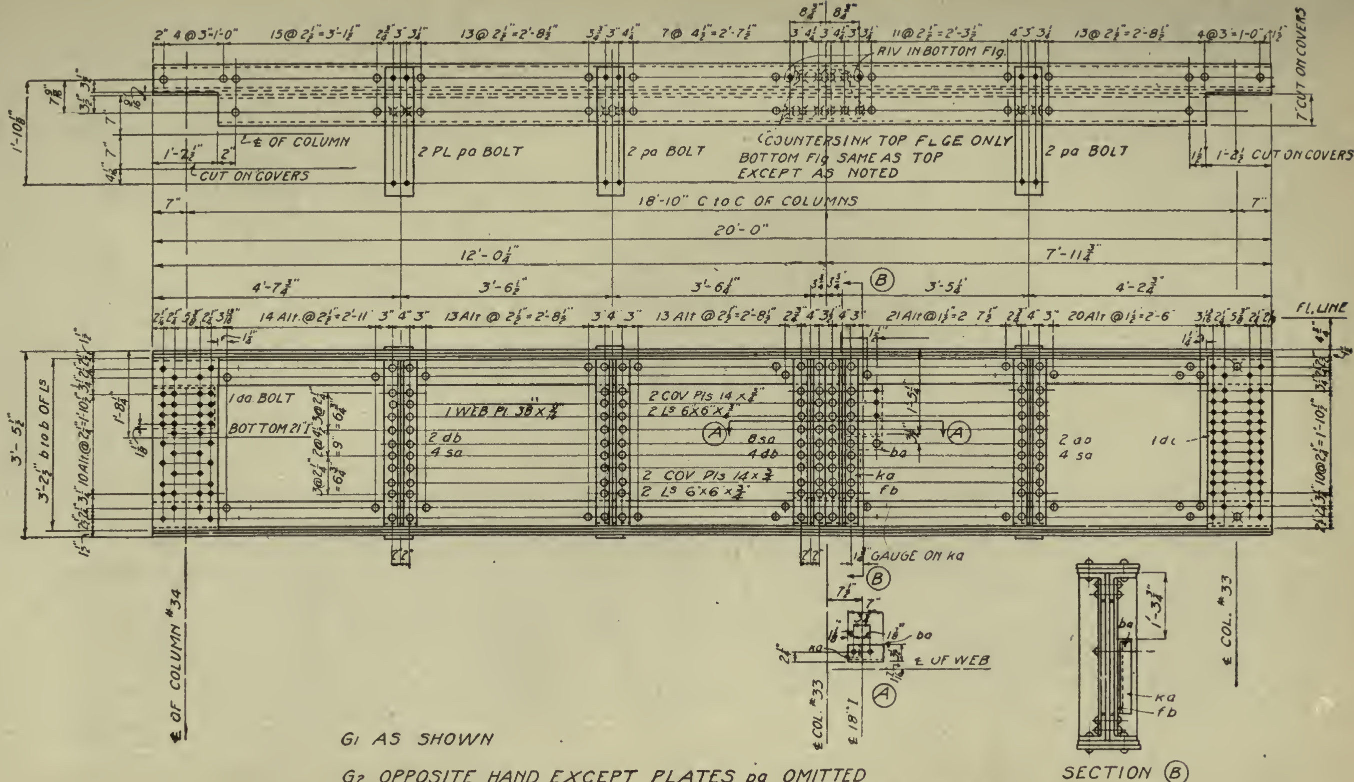


Fig. 125. Shop Detail Drawing of Girder, the Design Drawing of Which Is Shown in Plate O

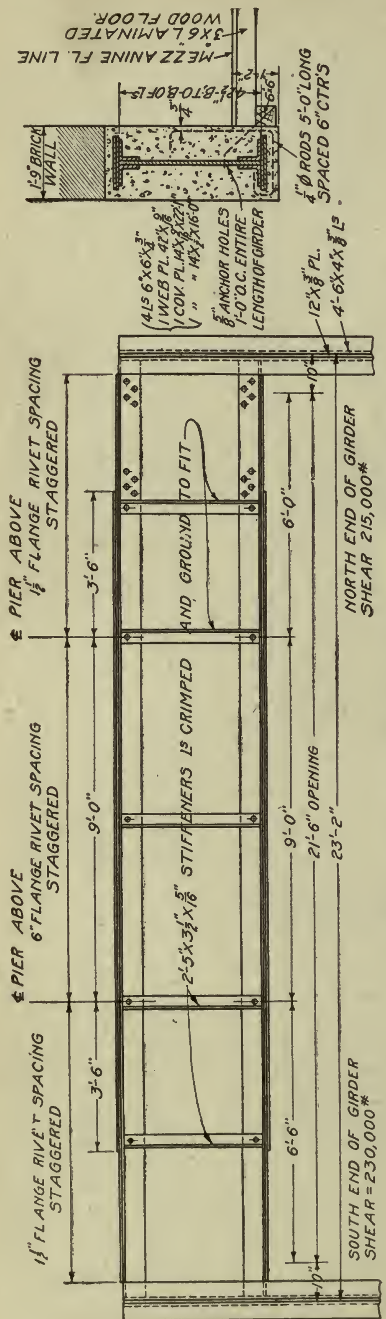


Fig. 126. Drawing of Plate Girder Used as a Lintel Over Driveway

neutral axis, the procedure in designing is the same as for symmetrical girders.

PROBLEMS

1. A lintel is made of a 12″ I 31½# and a plate on the top flange 12″×$\frac{5}{16}$″. What is the moment of inertia of the section?

2. What is the resisting moment of a 15″ I 42# which has two holes for ¾-inch rivets in the bottom flange?

PRACTICAL APPLICATIONS

Girder Supporting a Column. In order to get the rooms in the lower part of a building arranged satisfactorily, it is sometimes desirable to space the columns differently than they are placed above. This makes it necessary to carry the upper columns on girders. Such a case is shown in Plate O, p. 285. As is usual in such cases, the amount of vertical space available is limited and the depth of the girder is fixed by other considerations than economy of design. The top is limited by the floor level above, it being necessary to have room for fireproofing and for the finished flooring. The bottom is limited by the clearance required for the floor below. The actual depth of web is determined after making a preliminary design of the flanges and finding the approximate thickness of flange plates.

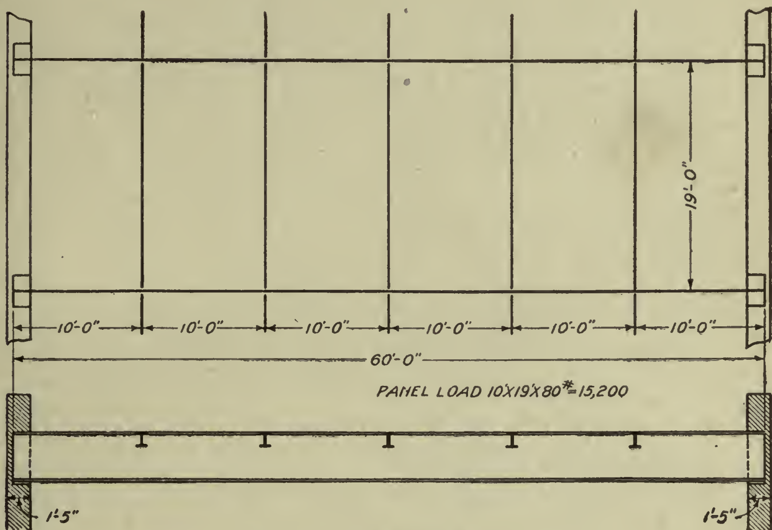


Fig. 127. Girder for Garage Roof

The vertical shear is so large that a single web plate would have greater thickness than is desirable and, furthermore, the shape and position of the supporting columns would make the connection of a single girder somewhat difficult to design. This leads to the adoption of two web plates.

At the supporting columns it is desired to connect one web plate to each flange of the column as shown. If a box girder were used, it would be difficult to erect it, hence two girders best fulfill the conditions. Having settled the above points, the girder is designed by the methods which have been given. Plate O shows the design drawing of the girder and Fig. 125 is the shop detail drawing.

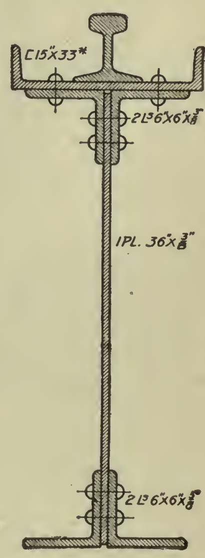


Fig. 128. Section of Typical Crane Girder

PROBLEM

In the first story of a building it is necessary to omit a column and support the upper part of the column on a girder. The span of the girder is 36 feet. The load is 540,000 pounds applied at the center of the span, and in addition to this there is a u. d. load from the second floor, the weight of girder with its fireproofing amounting to 4200 pounds per lineal foot. The depth available is 50 inches. Design the cross section of the girder.

Plate Girder Lintel. Fig. 126 shows a plate girder used as a lintel over a driveway into a building. It supports the wall above and the floor loads which bear on the wall.

Roof Girder. A garage roof is to be built with no supporting columns, so it must be carried from wall to wall on girders. The roof slab rests on I-beams which

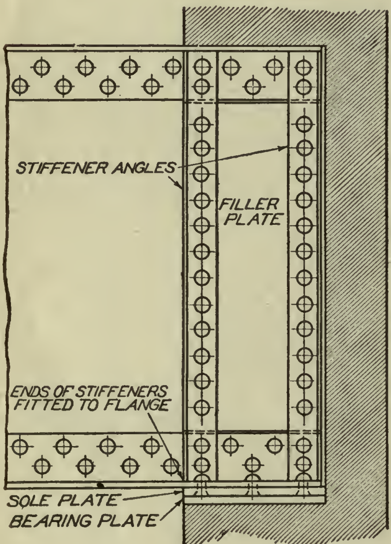


Fig. 129. Plate Girder Bearing on Masonry

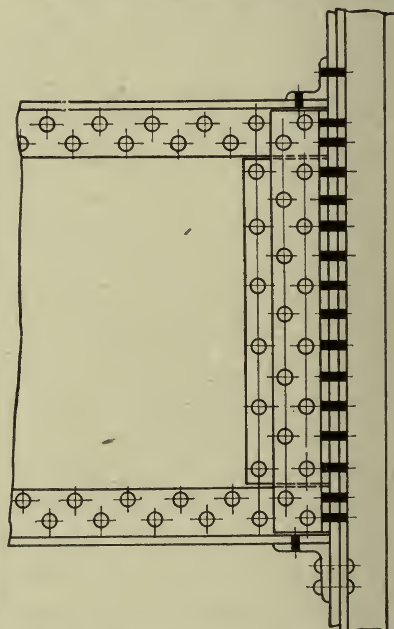


Fig. 130. Diagram Showing Web Connection of Girder to Column

are connected to the girders. The dimensions and loads are given in Fig. 127. There is no limitation of depth, the most economical section being desired.

PROBLEM

Design the girder for the conditions given above and make design drawing at $\frac{3}{4}$-inch scale.

Crane Girders. Crane girders do not belong to the class of buildings now under consideration. Fig. 128 represents a typical crane girder and is given to illustrate the use of an unsymmetrical section of girder. The stresses in a crane girder and the design are explained under Runway Girders in "Roof Trusses". The channel on the top flange is required to give lateral stiffness to the girder in order to resist the lateral thrust of the crane when the carriage is moving crosswise of the building. It also serves incidentally as a guard rail.

PROBLEM

Locate the neutral axis of the girder illustrated in Fig. 128.

DETAILS OF CONSTRUCTION*

End Bearings. When the end of a girder bears on masonry, Fig. 129, the bearing plate is designed in the same manner as for beams. With riveted girders it is much more frequently necessary to replace the plain bearing plate by I-beams to spread the bearing along walls, than when the girder is an I-beam. A sole plate should be riveted to the bottom of the girder. It stiffens the flange angles and furnishes a more even bearing surface than the angles. In high-grade work, the bottom of the girder may be faced before the sole plate is attached.

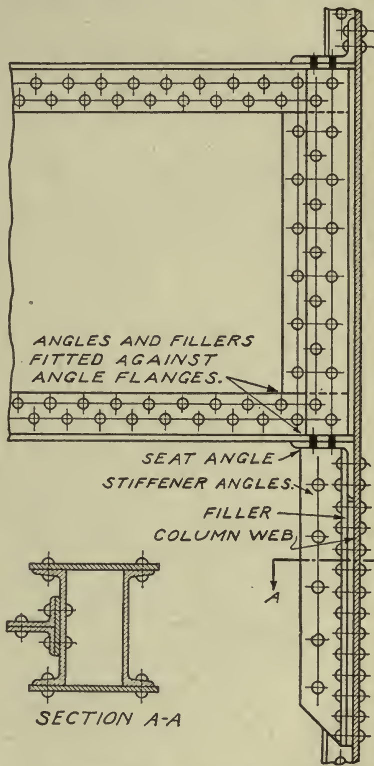


Fig. 131. Diagram Showing Bracket Connection of Girder to Column

A very heavy load may require a bearing plate thicker than it is practicable to obtain. Then, if it is not desired to use I-beam grillage, a cast-iron pedestal may be used similar to those used for columns. The method of designing them is given under columns, p. 220.

PROBLEM

Design the end bearing for the girder specified in Fig. 114.

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*The details of *stiffener angles, filler plates, flange plates,* and *rivet spacing* have been discussed and illustrated in the preceding pages.

Connections to Columns. *Web Angle Connection.* The connection of a girder to a column is usually made with web angles. The connection is designed in the same manner as for I-beams. The angle legs connecting to the girder web should be wide enough to

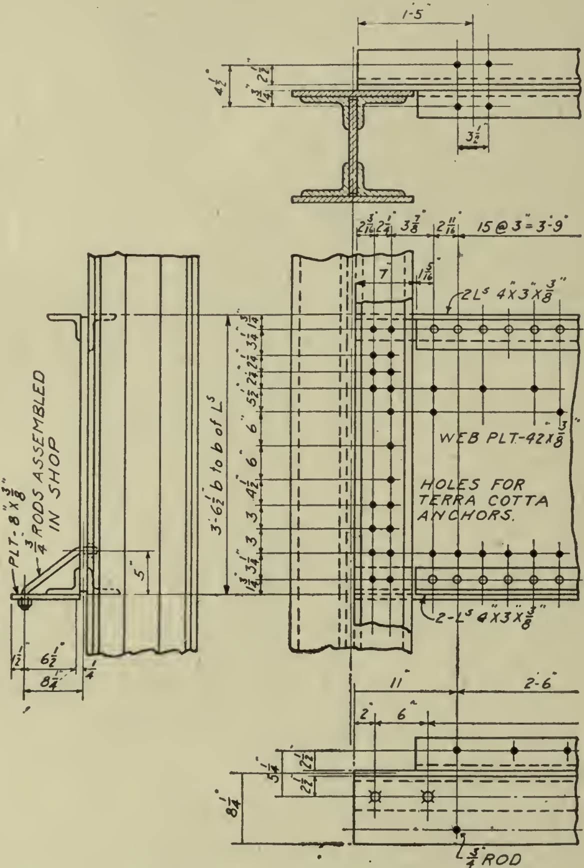


Fig. 132. Diagram Showing Connection of Girder to Face of Column

take two rows of rivets and, if the construction is heavy, the filler plate should be wide enough to take a row of rivets beyond the edge of the angles, Fig. 130. The end angles must be set accurately to the correct length and at right angles to the axis of the girder. In railroad bridge construction the end angles are required to be faced and, to allow for it, the angles used are ⅛ inch thicker than otherwise would be required. This should be done on heavy work in building construction.

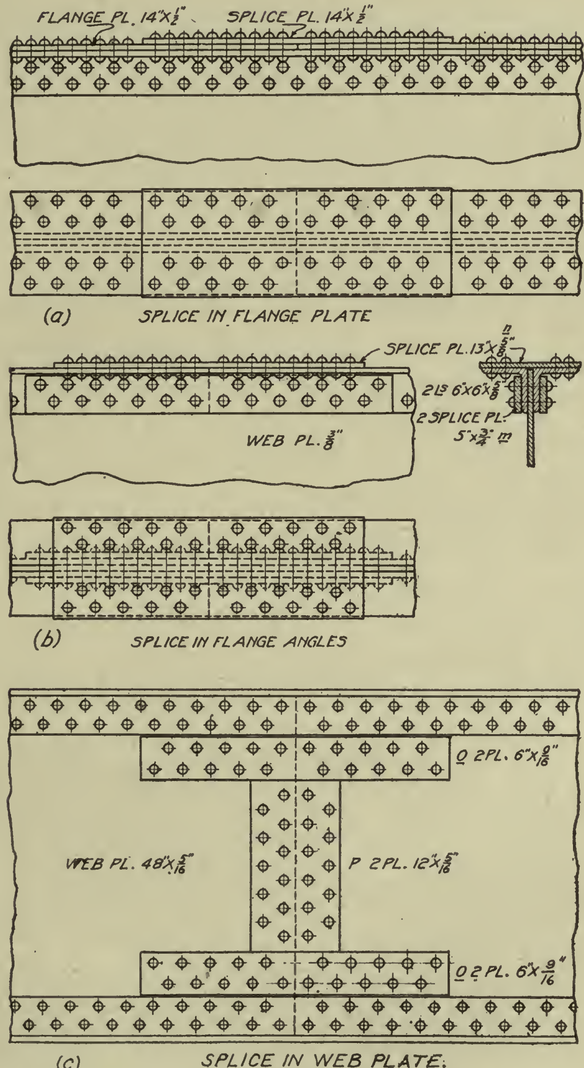


Fig. 133. Diagrams of Splices in the Members of a Plate Girder Section

Bracket Connection. The bracket connection, Fig. 131, may be used. It does not make as stiff a joint as the web connection and should not be used unless there is some special reason for it. This type of connection is specially applicable to box columns on which the brackets must be riveted before the column is assembled. Other forms of connection may be used to meet special conditions. Fig. 132 shows a connection of the web directly to the face of the column.

Splices. It is self-evident that there should be no splice in a girder section or in any of its members unless such a splice is absolutely necessary. If the splicing is of individual members rather than the whole girder section, the extra work is done at the shop instead of in the field and, therefore, is not so serious.

Splicing Due to Transportation Difficulties. The splicing of an entire girder section may be occasioned by transportation conditions but it is expensive on account of extra material and field riveting required, and cannot be considered as good as the unspliced section. A girder of any length likely to occur in building construction can be shipped by rail, so that the matter involves only the comparison of the extra freight cost with the cost of the splice. But transportation by boat involves not only the extra charge for long members but an absolute limit to the length that can be stowed. The designer, if not familiar with freight rates and rules, must investigate them, if girders longer than 36 feet are to be shipped.

Splicing Due to Members Longer than Stock Sizes. The individual members of a girder may need splicing, due to inability to secure material of sufficient length, which often happens when material is ordered from stock. This indicates the desirability of consulting stock lists while designing, so that the available sections may be used. The rolling mills regularly furnish angles 60 feet long and by special arrangement will furnish longer lengths. All usual sizes of cover plates are furnished in lengths up to 85 feet. Web plates are most likely to require splicing. Lists of extreme sizes are given in the handbooks. Greater lengths than there listed can be secured from some mills, but it is safer to be governed by these lists unless definite arrangements can be made for the longer plates.

Full Strength Splices for Flanges. Both tension and compression flanges must be fully spliced, i. e., the entire tension or compression must go through the splice plates and angles and the rivets by which they are attached. In this case no reliance is placed on abutting ends of compression members as is done in columns.

Figs. 133-a, -b, and -c show, respectively, splices in a flange plate, in flange angles, and in web plate.

Splice for Flange Plate. Fig. 133-a. The flange plate is $14'' \times \frac{1}{2}''$. The stress must be carried across the gap by a single plate (assuming that there is no unused capacity in the flange angles), which must not be less than $14'' \times \frac{1}{2}''$. The net area of this plate after deducting rivet holes is $12\frac{1}{4}'' \times \frac{1}{2}''$ or 6.125 square inches. Its tensile value is $6.125 \times 16{,}000$ or 98,000 pounds. The splice rivets are in single shear, hence the number required on each side of the joint is $\frac{98{,}000}{5300}$ or 19. Use 20 rivets.

Splice for Flange Angles. Fig. 133-b. The flange angles are 2 Ls $6'' \times 6'' \times \frac{5}{8}''$. Their area is 2×7.11 or 14.22 square inches, which, after deducting one rivet hole from each angle, becomes a net area of 13.12 square inches. The splice plates must have this net area. It is desired to splice both legs of each angle as directly as possible, so the splice plates are arranged as shown. Their sizes and net areas are

m	2 Pl. $5'' \times \frac{3}{4}''$, net area	6.18
n	1 Pl. $13'' \times \frac{5}{8}''$, net area	7.03
	Total	13.21 sq. in.

From these values the number of rivets can be computed in the usual way, noting that the rivets through m are in double shear and through n in single shear. The plates m must extend beyond n at each end far enough to take two additional rivets. The purpose of this is to relieve the angles of a portion of their stress before the first holes in n are reached. Otherwise, in designing the main girder section one hole additional would be deducted from each angle.

Splice for Web Plate. Fig. 133-c. The web plate must be spliced to transmit shear and bending according to the amount of these stresses where the splice occurs; if at the place of maximum bending moment, only the bending stresses need be considered, shear being zero; if near the end where the flange angles will take care of all the bending stresses, then only the shear need be provided for.

Resistance to Bending. The necessary resistance to bending can be furnished by a flange plate, as in Fig. 133-a; by splice plates on the angles, as plates m in Fig. 133-b; or by splice plates o in Fig.

133-c; or by any combination of them. In either case the moment of inertia of the net section of the splice plate must equal that of the web plate, or such portion of it as is needed at the place where the splice occurs. It must be noted that a web plate which must be spliced loses some of its moment of inertia because of the holes for attaching the splice plates; consequently, it is better, if practicable, to use a form of splice which will add no rivet holes. If a flange plate is used as a part of the girder section, then an additional flange plate may be used for splicing the web. If there is no flange plate in the girder section, then plates such as *m*, Fig. 133-b, may be used to advantage for all or part of the web splice.

Taking for example the girder in Fig. 116, the web plate is 48″

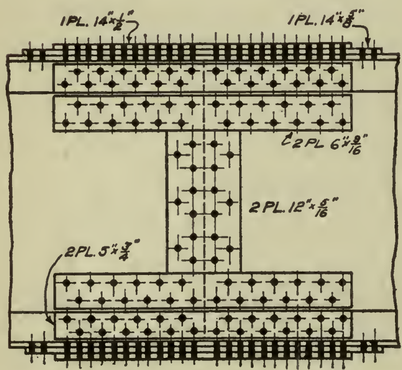


Fig. 134. Splice in Plate Girder

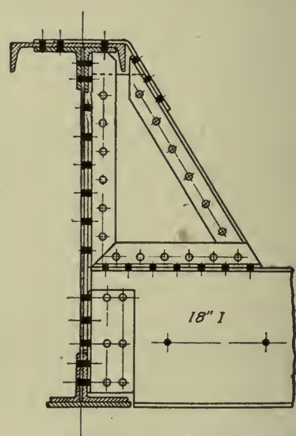


Fig. 135. Bracket for Bracing Top Flange of Girder

$\times \frac{5}{16}$″. Its net moment of inertia is 2620, p. 141. Two flange plates 14″$\times\frac{1}{4}$″, after deducting 2 rivet holes from each, have a net value of I

$$I = 2 \times 12\tfrac{1}{4} \times \tfrac{1}{4} \times 24.9 \times 24.9 = 3800$$

which is more than required.

Taking the girder shown in Fig. 115, if a flange plate should be used for splicing, the angles would be weakened by the rivet holes for attaching the plate. If plates such as *m*, Fig. 133-b, are used, no additional rivets are needed. Try four plates 5″$\times\frac{3}{8}$″. Their net value of I after deducting one rivet hole for each is

$$I = 4 \times 4\tfrac{1}{8} \times \tfrac{3}{8} \times 20\tfrac{1}{2} \times 20\tfrac{1}{2} = 2600$$

which is near enough to be satisfactory.

In a similar manner, plates *o*, Fig. 133-c, are found to be 6″ × $\frac{9}{16}$″. The strength of the splice plate must be developed by rivet bearing in the web plate requiring 10 on each side of the joint. Although this is the most direct method of splicing for bending, it is not as economical as either of the other methods given above.

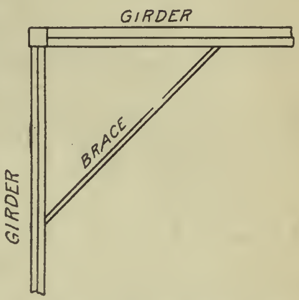


Fig. 136. Brace for Plate Girder

Resistance to Shear. For resisting shear, the splice plates are in the form of the plates *p*, Fig. 133-c. On each side of the joint there must be enough rivets to transmit the total shear. They may be in one or more rows. The thickness of each plate must be at least half that of the web plate and is subject to the same minimum. Hence, in this case the thickness is made $\frac{5}{16}$ inch.

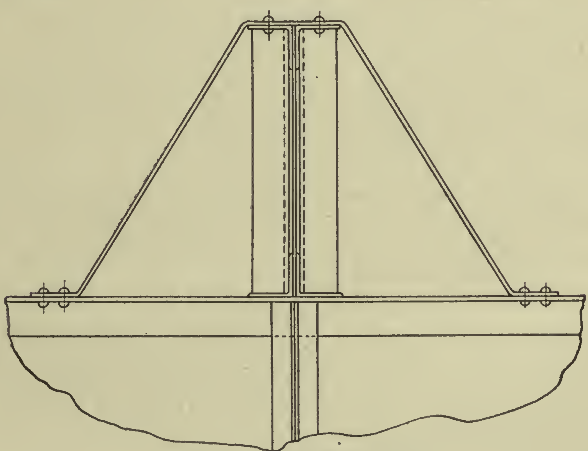


Fig. 137. Brace for End Girder

Position of Splices. Girders completed in the shop will have splices arranged to come at different places; thus the web may be spliced at the center and the angles near one end; still better, one angle may be spliced on one side of the center and the other on the opposite side. Of course, in a field splice all the elements are joined at one place. The method of computing is the same as has been given for the individual parts of the girder (bearing in mind that the rivets are field driven). Fig. 134 illustrates such a splice made up from the several splices shown in Fig. 133.

PROBLEM

Design field splice for plate girder shown in Fig. 115.

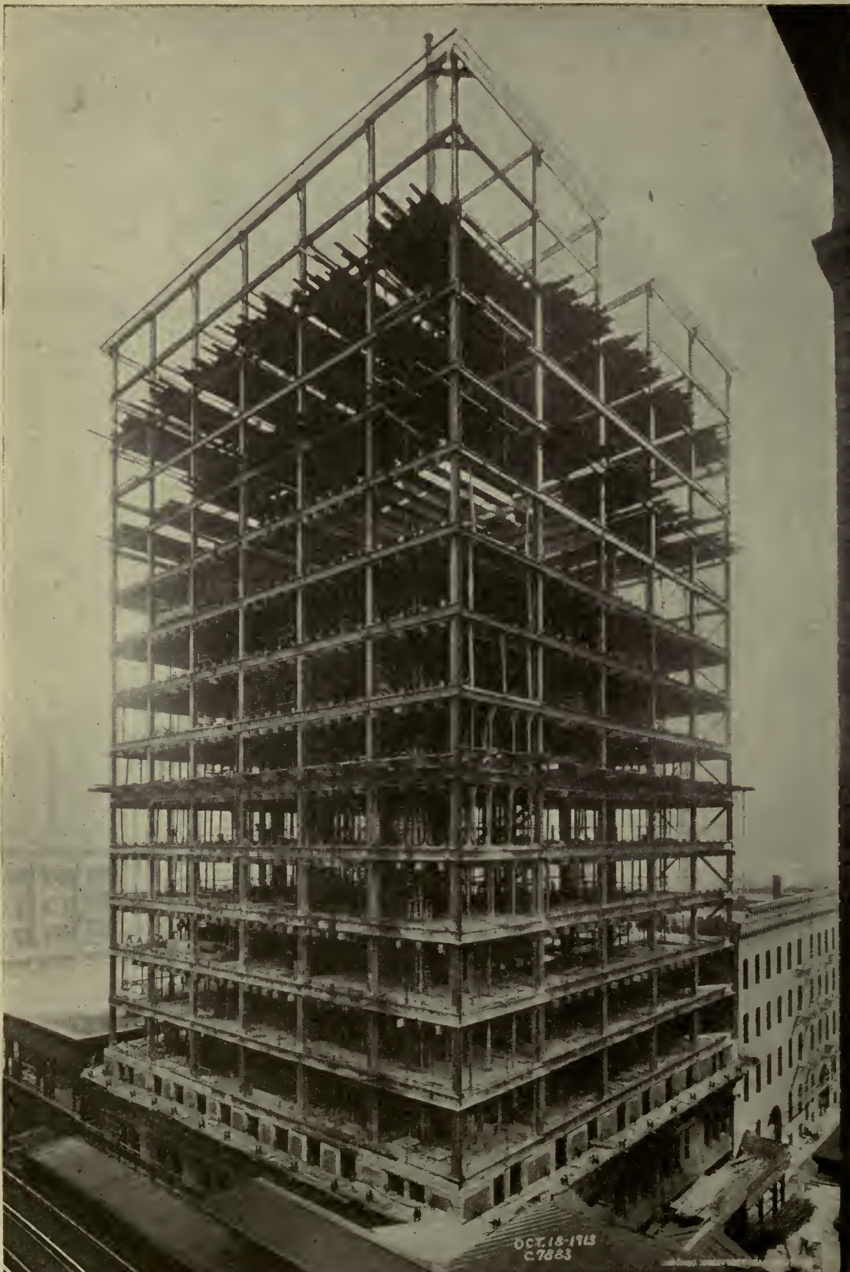
Lateral Support. Girders, like beams, must be supported laterally to prevent the compression flange from buckling. Schneider's Specifications provide that "the unsupported length of flange shall not exceed 16 times its width. In plate girders used as crane runways, if the unsupported length of the compression flange exceeds 12 times its width, the flange shall be figured as a column between the points of support."*

In most cases the lateral support is provided by the joists or floor construction. Where this is not the case, the supports can be provided in a number of different ways. For lengths up to 25 feet, the necessary stiffness can be provided by the use of wide flange plates. For greater lengths, box girders may be used, if the load warrants their use. Fig. 135 shows a plate girder to which a joist connects near the bottom. From this joist a bracket extends up to and supports the top flange. The corner brace indicated in Fig. 136 sometimes may be used to advantage.

As provided in Schneider's Specifications, crane girders whose length exceeds 12 times the width must be designed as columns. The method is the same as given hereinafter for columns.

The ends of the girders must be especially well secured against overturning. When connected to columns or other girders, the desired result is easily attained by the use of web angles or top connection angles. If the end rests upon and is built into masonry, the required support is thus provided. Fig. 137 shows one girder resting on another and braced thereto.

**Transactions American Society of Civil Engineers*, Vol. LIV, p. 495.



STRUCTURAL STEEL SKELETON OF FORT DEARBORN HOTEL, CHICAGO
Courtesy of Holabird and Roche, Architects, Chicago

STEEL CONSTRUCTION

PART III

COMPRESSION MEMBERS—COLUMNS

STEEL COLUMNS

Definitions. A *column* (or *strut*) is a member subjected to compression in the direction of its longitudinal axis, i. e., subjected to axial compression. The term "column" is usually applied to a vertical member subjected directly to a gravity load. The compression members of trusses, and also small isolated members, and members in other than the vertical position, are called "struts".

A series of columns in a vertical line is called a "stack".

The columns in any one story of a building constitute a "tier".

Loads and their Effects. *Computation of Loads.* The loads on a column are applied to it by the column section above and through the connections of other members or other materials. Most commonly this is through beams and girders. The amounts of these loads may be taken from those previously computed for the beams and girders, or may be computed directly from the floor and wall areas tributary to the column. The former method is easier when the loads and areas are irregular, and the latter when the loads are uniform and the arrangement of beams regular. Practical examples of computing the loads are given later in this book.

The ideal condition of loading of a column is had when the load is applied uniformly over the top of a column, and when the bottom of the column bears evenly on its support or foundation. In a stack of columns, the load on any column which comes from the column above is usually applied in this ideal way. But the other loads are generally applied to the sides of the column through beam connections, in many cases with greater loads on one side than on the other.

Loads applied centrally, or which are equally balanced on opposite sides, are called "concentric loads", Fig. 138-a. Loads applied to the sides of the column and not balanced, or those which bear on top but are not centrally placed, are called "eccentric loads", Fig. 138-b. These terms apply to the bearing at the bottom of the column as well as to the loading at the top, but usually the bearing at the bottom is made uniform, i. e., concentric.

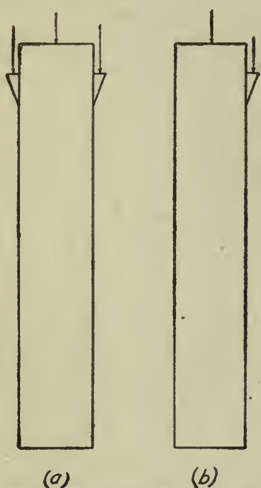


Fig. 138. Diagram Showing Examples of (a) Concentric Loads and (b) Eccentric Loads

Concentric Loads. Concentric loads, Fig. 139, produce direct or axial compression in the column. This compression may be considered as evenly distributed over the entire cross section, even if the loads be balanced loads connected to opposite sides of the column. Then the unit stress P on the column is the load W divided by the area A; which is expressed by the formula

$$P=\frac{W}{A}$$

Conversely the capacity of a column or its total permissible load is the allowable unit stress multiplied by the area:

$$W=P\,A$$

For example, assume the load on a column to be 190,000 pounds and the area of the assumed column 16.4 square inches. Then the unit stress, or average compression, is $\frac{190{,}000}{16.4}$ or 11,585 pounds per square inch.

Eccentric Loads. Eccentric loads, Fig. 140, produce axial compression and in addition cause bending stresses. The axial compression is determined in the same way as for concentric loads, and the bending stresses in the same manner as for beams, p. 81.

Fig. 139. Diagram of Stresses from Concentric Loads

The bending, or eccentric, moment of the load is the amount of the load multiplied by its distance from the neutral axis of the column. The sum of the axial compression per square inch and the maximum compression fiber stress per square inch is the maximum combined stress resulting from the eccentric load. (See Flexure and Compression for Beams, in "Strength of Materials", Part II.) This is illustrated in Fig. 140. W' is an eccentric load. The direct stress in the column is represented by the area $a\,b\,c\,d$ and equals W. (This area may represent the total load on the column if there are other loads than W'.) The bending moment produces the compression $o\,b\,b'$ and the tension $o\,c\,c'$. Then the maximum fiber stress in the column is $a\,b'$, being the sum of $a\,b$ and $b\,b'$. On the side opposite to the eccentric load, the tension due to bending overcomes part or all of the compression due to direct stress. The result in this case is $d\,c'$; but the stress in this side of the column rarely needs consideration. Of course, the eccentricity may be so great that the opposite side of the column is in tension, but even this does not require attention unless the column is spliced.

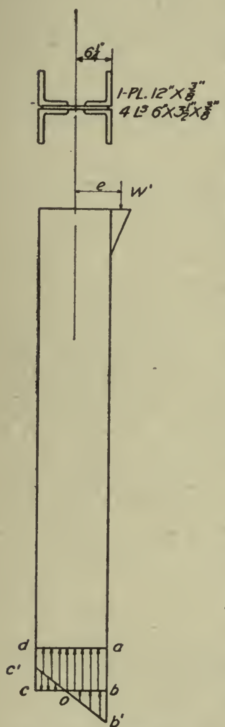


Fig. 140. Diagram of Stresses from Eccentric Loads in Compression

The total stress produced by all the loads equals the sum of the stresses produced by the loads separately*. Some authorities allow *three-fourths of the bending moment* to be used in computing the effect on the column. This practice is satisfactory and is followed in the illustration used later in this book.

Typical Cases. The entire load on the column, including its own weight and the weight of the fireproofing, must be determined (making no distinction between concentric and eccentric loads). Then compute the bending moments due to the eccentric loads, dividing these moments between the respective axes of the column.

(a) As an example, refer to Figs. 139 and 140, letting them represent the same column. Assume W a concentric load of 100,000

*This statement is not exactly correct but represents usual practice.

pounds; W' an eccentric load of 50,000 pounds; and e an eccentricity, or lever arm of W', of 10 inches. Then

$$\text{Total load} = 100{,}000 + 50{,}000 = 150{,}000\#$$

The bending moment due to the eccentric load is

$$M = 50{,}000 \times 10 = 500{,}000 \text{ in.-lb.}$$

As a trial section, take a column made of 1 Pl. $12'' \times \frac{3}{8}''$ and 4Ls $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ from which c, the distance from the neutral axis to the extreme fiber, is 6.125 inches; r, the radius of gyration about the same axis as the bending moment, is 5.00 inches; $\frac{I}{c}$, the section modulus, is 74.7 inches^3; and A, the area, is 18.2 square inches. The average stress resulting from the total load is

$$\frac{150{,}000}{18.2} = 8240\# \text{ per sq. in.}$$

This is represented by $a\,b$ in Fig. 140.

The maximum fiber stress resulting from the bending moment, taking three-fourths of the computed moment, is

$$\frac{\frac{3}{4} \times 500{,}000}{74.7} = 5020\# \text{ per sq. in.}$$

This is represented in Fig. 140 by $b\,b'$ and $c\,c'$ in compression and tension, respectively.

Then the total maximum fiber stress in the column is

$$8240 + 5020 = 13{,}260\# \text{ per sq. in.}$$

This is represented by $a\,b'$.

The method of determining the *allowable* stress has not yet been given so it cannot be decided whether the trial section given above is satisfactory.

(b) Fig. 141 illustrates cases of concentric and eccentric loading. In each of them there may be a concentric load from the column section above. In Fig. 141-a, the loads are concentric, provided those on opposite sides are equal and balance each other. If m be omitted, o becomes eccentric, but as it connects to the web of the column the eccentricity is small and usually is neglected. If n be omitted, p becomes eccentric with a moment arm e and a bending moment $p \times e$. If n is less than p, the difference is the eccentric load and the bending moment is $(p-n) \times e$. In Fig. 141-b, the load u is eccentric about the axis *2-2*, and the bending moment is $u \times e_2$. The resulting fiber stress must be computed from the moment of inertia about the same axis. The load v is eccentric about the axis *1-1*, and the bending moment is $v \times e_1$. The resulting fiber stress must be computed from the moment of inertia about the same axis. Both eccentric loads produce compression at the corner d, hence the effects of both must be added to the axial stress produced by the total load, in order to determine the maximum fiber stress in the column.

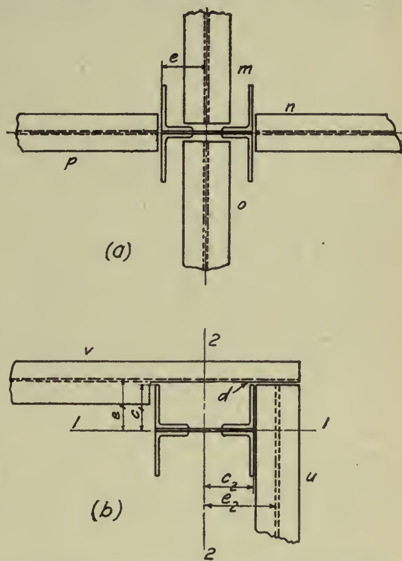


Fig. 141. Diagrams Showing (a) Concentric Load and (b) Eccentric Loads on Columns

As an example, take Fig. 141-b and assume the following data, taking for the trial section, a column made of 1 Pl. $12'' \times \frac{5}{8}''$ and 4 Ls $6'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$ from which I_1 is 213; I_2 is 721; c_1 is $6\frac{3}{8}''$; c_2 is $6\frac{1}{8}''$; e_1 is 7″; e_2 is $9\frac{1}{4}''$; and A is 29.7 sq. in.

Concentric load from column section above	= 150,000
v	= 30,000
u	= 45,000
Total load	= 225,000 #

$$\text{Unit stress from total load} = \frac{225{,}000}{29.7} = 7575\,\#\text{ per sq. in.}$$

$M_1 = 30{,}000 \times 7 = 210{,}000$ in.-lb.

$\frac{3}{4}$ of this $= 157{,}500$ in.-lb.

$$\text{Unit fiber stress due to } v = \frac{157{,}500 \times 6\frac{3}{8}}{213} = 4720\,\#\text{ per sq. in.}$$

$M_2 = 45{,}000 \times 9\frac{1}{4} = 416{,}000$ in.-lb.

$\frac{3}{4}$ of this $= 312{,}000$ in.-lb.

$$\text{Unit fiber stress due to } u = \frac{312{,}000 \times 6\frac{1}{8}}{721} = 2650\,\#\text{ per sq. in.}$$

$$\text{Total fiber stress at } d = 7575 + 4720 + 2650 = 14{,}945\,\#\text{ per sq. in.}$$

PROBLEM

Assume a heavier column section for the example given above and compute the total maximum fiber stress.

Eccentric Load In Terms of Equivalent Concentric Load. The effect of the eccentricity of the load can be expressed in terms of an equivalent concentric load, which can be added to the actual load and the resulting total be applied as a concentric load, giving the same maximum stress as if computed by means of the bending moment. The proportion to be added, if the full eccentricity is used, is given by the expression

$$W_e' = W' \frac{e\,c}{r^2}$$

Or, if the reduction in eccentricity is made in accordance with the rule on p. 175, the expression is

$$W_e' = \tfrac{3}{4} W' \frac{e\,c}{r^2}$$

In these formulas W' is the eccentric load and W_e' is the equivalent concentric load. Before this method can be applied, it is necessary to select the trial column section, and from it compute the values of c and r. As the values of c and r for the trial section will vary but little from those for the final section, it usually will be unnecessary to correct the equivalent concentric load computed by this method.

Referring now to the example illustrated in Figs. 139 and 140 and explained on p. 176, the eccentric effect is

$$W_e' = \tfrac{3}{4} \times 50{,}000 \times \frac{10 \times 6.125}{5 \times 5} = 91{,}875\#$$

Then the total equivalent concentric load on the column is

$$\begin{aligned} W\ &= 100{,}000 \\ W' &= \ \ 50{,}000 \\ W_e' &= \ \ \underline{91{,}875} \\ &\quad\ 241{,}875\# \end{aligned}$$

and the resulting stress in the column is

$$\frac{241{,}875}{18.2} = 13{,}285\# \text{ per sq. in.}$$

This result agrees closely with the results obtained by use of the bending moment. It would agree exactly if all the computations and the values of the properties were given in more exact figures. Note that the equivalent concentric load is not carried down into the next lower section of column but disappears at the bottom of column section under consideration.

PROBLEM

Compute the equivalent concentric load and the resulting unit stress for the eccentric loads u and v in Fig. 141-b from the data given on p. 177.

Strength of Columns. The ideal column is perfectly straight, symmetrical, and homogeneous, but these conditions are never fully attained. The material may not be exactly straight, then inaccurate workmanship, the punching of rivet holes, driving of rivets, abuse in handling, and internal defects of the steel, all co-operate to produce results somewhat short of ideal. These imperfections are of more importance with long than with short columns, and likewise with small columns than with large ones.

The foregoing conditions make it necessary to use lower stresses in columns than are used for beams; also to vary the stresses according to the length and size of the column. The relations cannot be expressed in a rational formula, that is, a formula deduced from theory, as is the case with beams; hence empirical formulas are used, i. e., formulas based on experimental data. A large number of tests have been made to determine the effect of the length and size on the strength of columns. Several formulas have been derived giving results agreeing closely with the tests.

Formula for Unit Stress. The simplest of these formulas and the one now most generally used is

$$P=16{,}000-70\frac{l}{r}$$

in which P is the permissible compression per square inch of cross section; l is the unsupported length of column in inches; and r is the least radius of gyration in inches. The radius of gyration rather than the side or the diameter is used as the measure of the size of the column as it relates more directly to the stiffness.

From the above formula the allowable stress per square inch can be determined for any column having known values of l and r. Thus if $l=180''$ and $r=2.4''$

$$P=16{,}000-70\frac{180}{2.4}=16{,}000-5250=10{,}750\#\text{ per sq. in.}$$

Then the total capacity W equals $P \times A$ (p.174); and assuming $A = 12.0$ sq. in.

$$W = 10{,}750 \times 12.0 = 129{,}000\#$$

The end condition of the column has some effect on the strength. A column which has ends resting on pins or pivots will not support as great a load as one which has flat or fixed bearings. The formula given above applies to columns with flat or fixed ends and as these are used almost universally in building construction, the other formulas need not be considered in this text. Pivoted and pin ends for columns occur in bridge construction and the necessary formulas for them are given in books on that subject.

The values given by the formula do not apply to very long or very short columns. The maximum value of P allowed (see Unit Stresses, p. 51) is 14,000 pounds. This corresponds to a value of $\frac{l}{r} = 30$, so 14,000 must be used when $\frac{l}{r}$ is equal to, or less than, 30. In the other direction the limiting value of $\frac{l}{r}$ is 120, according to most specifications. However, larger values may be used with safety if particular care is taken to avoid eccentricity.

Schneider's Specifications provide that "No compression member shall have a length exceeding 125 times its least radius of gyration, except those for wind and lateral bracing, which may have a length not exceeding 150 times the least radius of gyration."*

The formula takes into account only the average imperfections in columns, and makes no allowance for the different styles of columns. Nevertheless, it is known that columns with solid web plates are more efficient than laced columns, and laced columns in turn are more efficient than columns with batten plates. There is no well-established practice in reference to this but a reasonable allowance is to deduct from the values given by the formula 25 per cent for laced columns and 50 per cent for battened columns.

Having adopted a formula by which the allowable unit stress can be computed, the example given on p. 176 can be completed.

**Transactions American Society Civil Engineers*, Vol. LIV, p. 495.

The trial section there used was a column made of 1 Pl. $12'' \times \frac{3}{8}''$ and 4 Ls $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, from which r (least value) is 2.56″; and A is 18.2 sq. in.

Assume $l = 102''$. The allowable unit stress is

$$P = 16{,}000 - 70\frac{l}{r} = 16{,}000 - 70\frac{102}{2.56} = 13{,}200\# \text{ per sq. in.}$$

The maximum fiber stress computed from the assumed loading is 13,260 pounds per square inch, hence the trial section is satisfactory.

Taking the example on p. 177, the trial section of column is made of 1 Pl. $12'' \times \frac{5}{8}''$ and 4 Ls $6'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$, from which r (least value) is 2.68″; and A is 29.7 sq. in.

Assume $l = 138''$. The allowable unit stress is

$$P = 16{,}000 - 70\frac{138}{2.68} = 12{,}400\# \text{ per sq. in.}$$

The maximum fiber stress computed from the assumed loading is 14,945 pounds per square inch, hence the trial section is not large enough and a heavier section must be tried.

Properties of Column Sections. In the foregoing discussion of the formulas, it appears that certain properties of the column must be known before the formula can be applied. The formula for allowable unit stress requires the radius of gyration r and the unsupported length l of the column section. If the column supports an eccentric load, the moment of inertia I, or the radius of gyration r, and the distance to the extreme fiber c must also be known in order to compute the maximum fiber stress due to bending.

Area. The area A is computed by adding together the areas of the several pieces which make up the column section. The areas of the individual pieces are given in the handbooks. No deduction is made for rivet holes.

Distance from Neutral Axis to Extreme Fiber. The distance to the extreme fiber from the neutral axis is readily computed from the dimensions of the column section. It must be taken from the axis about which the bending moment is computed. Thus, in Fig.

141-b, c_2 must be used in connection with the load u, and c_1 in connection with the load v.

Moment of Inertia. The moment of inertia is computed by the method explained and illustrated on p. 37. It also must be taken in reference to the neutral axis about which the bending moment is computed. Thus in Fig. 141-b, I must be calculated in reference to axis *2-2* for the load u, and to axis *1-1* for the load v.

Radius of Gyration. The radius of gyration is computed about each axis by the method explained and illustrated on p. 38. The lesser value is usually required for computing the unit stress, but either or both may be required for computing eccentric effects. Thus, in Fig. 141-b, both radii of gyration are used.

There are conditions under which the larger radius of gyration is used. One such case is that of a column built into a masonry wall in such a way that it is supported by the masonry in its weaker direction, Fig. 142. Then the larger radius is used, but designers are cautioned against using this unless the wall is so substantial that it gives real support to the column. A casing of brick or concrete or a poorly built brick wall is not sufficient.

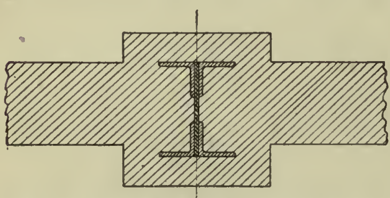


Fig. 142. Section Showing Column Supported by Masonry in Its Weaker Direction

It sometimes happens that a column is supported in one direction at closer intervals than in the other direction. The weaker way of the column should be turned, if practicable, in the direction of the closer supports. Then the design may be governed by the lesser radius combined with the shorter length; or by the greater radius combined with the longer length.

Unsupported Length. The length l is needed for solving the allowable unit stress. It is expressed in inches and is the unsupported length of column. This unsupported length is usually measured from floor to floor, but if there are deep girders with rigid connections, the clear distance between girders may be taken as the length.

PROBLEM

Compute the values of A, I_1, I_2, c_1, c_2, r_1, and r_2 for the column sections, which are shown in Fig. 143.

Column Sections. Practically all rolled sections of steel may be used as columns or struts, but only a few of them are economical when used alone. Most columns are built up of several pieces. Fig. 144 shows a number of sections.

Section a. The *single angle* is not economical but may be used for a light load. When used, its radius of gyration must be taken about the diagonal axis.

Section b. *Two angles* make a satisfactory strut for short lengths and light loads. Usually angles with unequal legs are used, with the long legs parallel. The radii about both axes are nearly the same for most sizes. The value about the axis *2-2* can be varied somewhat by the use of fillers between the angles. Such fillers should be spaced two to three feet apart.

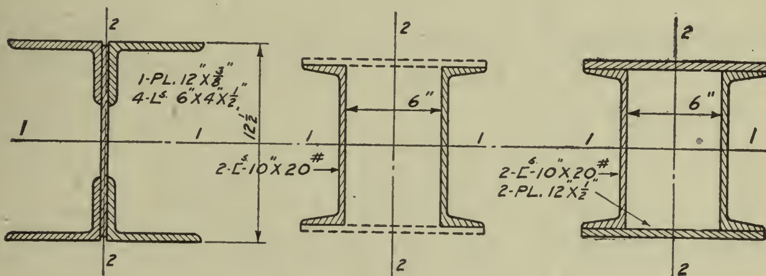


Fig. 143. Diagrams for Estimating Properties of Column Sections

Section c. The *star strut* is made of two angles with batten plates. The batten plates in each direction are spaced from two to four feet apart. They must be wide enough for two rivets in each end. The least radius is about the diagonal axis *3-3*. In accordance with the rule, p. 180, this being a battened section, the unit stress should be only one-half that given by the formula. Consequently, the section is not economical but is suitable to use when the load is light. It is quite useful as a brace between trusses and other similar situations.

Section d. *Four angles* placed at the corners of a square and joined together with lacing bars can be made to have a large radius of gyration with a small area. This makes a column suitable for supporting light loads on a long length. It is not suitable for eccentric loads. The spacing of the angles may be made as great

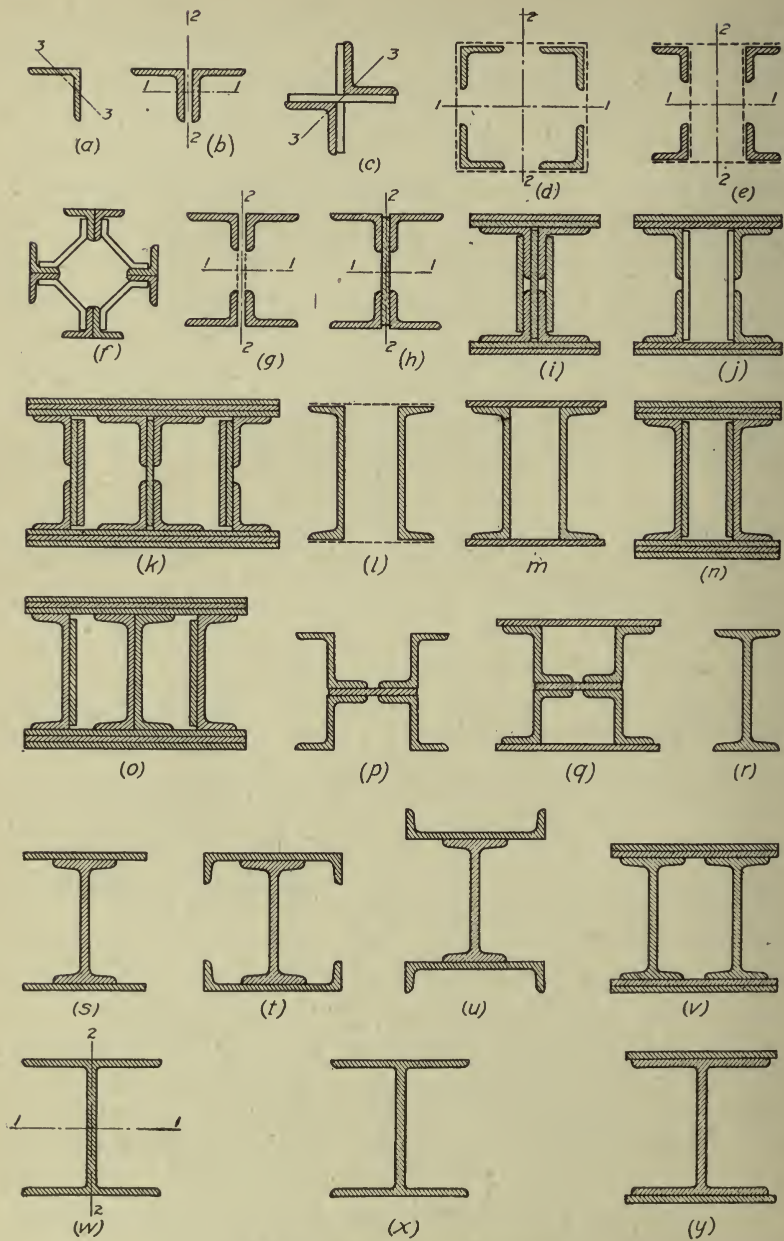


Fig. 144. Typical Column Sections

as required by the conditions. The allowable unit stress on this section must be reduced on account of the lacing in accordance with the rule, p. 180. However, if the column is filled and encased in concrete, the full unit stress may be used. It is well adapted to use in this way. On account of the weight of the lacing and the cost of shop labor, this section is more expensive than most others for a given area, hence it is used only for conditions described above.

Section e. When the angles are placed in this form, the cost of shop work is somewhat reduced, but otherwise the above comments apply.

Section f. The *Gray column** is made of eight angles joined together in pairs and these pairs are assembled into a column by means of batten or tie plates. The batten plates are usually made $8'' \times \frac{3}{8}''$ and spaced 2′-6″, center to center. The advantages of this section are its large radius of gyration and ease of making connections for beams and girders. Its disadvantages are that it is a battened column and, therefore, not capable of carrying the full unit stress given by the column formula; and that the expense of its manufacture is high, due to the bent batten plates. It has been used extensively with the full unit stress; however, it seems more reasonable to make some reduction. Since the battens are quite rigid the column is probably as good as a laced column, hence it can be used with a reduction of 25 per cent from the full unit stress. This column is not adapted to eccentric loads and is best suited to load conditions which would bring in equal loads to each of its four parts. This seldom occurs, the most common arrangement being two girders on opposite sides and two joists on the other sides. Thus two segments of the column are loaded much more heavily than the other two. The batten plates cannot be relied upon to equalize the load, but heavier angles can be used for the heavier loads. If this system of proportioning each segment to suit the loads which connect directly to it is used, the chief objection to this type of column is eliminated.

When filled and encased in concrete, the Gray column is very rigid and can then be loaded to the full unit stress. It is especially suitable as the core of concrete columns, and can be used thus in

*Designed and patented by J. H. Gray, C.E., New York, N. Y.

connection with reinforced concrete floor framing. When so used, the column may be rotated 45 degrees from the axis of the girders if it is desired to pass the reinforcing rods through the column. The bearing of the beams and girders in part can be directly on the concrete core and in part on lug angles riveted to the faces of the column.

The Gray column can be made any desired size using any standard angles. The practicable limits are ten inches square (minimum) and twenty inches square (maximum).

Section g. A column made of *four angles laced* has little merit as compared with the plate and angle column which is next described. Its only claim is that in some cases it may be cheaper to use lacing than to use a web plate. This would be so if there were some special condition requiring a deep column. As with other laced columns, it should not be allowed the full unit stress and should not be subject to any considerable eccentricity.

Section h. The *plate and angle column* is probably the most popular shape for buildings. It does not give the most economical distribution of metal, as the value of r is much greater about the axis *1-1* than about *2-2*. Its advantages are economy of manufacture and ease of making connections. Advantage can be taken of the greater value of r (and therefore of I) about the axis *1-1*, in providing for eccentric loads by so placing the column that the bending moment is about this axis.

Sections i, j, and k. In the use of *heavy forms of plate and angle columns,* a considerable variation in area can be made by varying the thickness of metal, keeping the depth constant, and making only a slight change in the width. If greater area is needed, flange and web plates may be added as in *i,* and still greater area may be secured by using the forms *j* and *k*. Section *k* is difficult to fabricate. The flange plates must be riveted to the center web first, and after this is done it is difficult to insert the outside web.

Section l. *Two channels laced* have a large value of r in proportion to the area. The channels can be so spaced that the values of r for both axes are about equal. This section of column has the same disadvantages as to unit stress and eccentric loads as other laced columns. The connections for beams and columns are more difficult to make than on plate and angle columns.

Sections m, n, and o. The *columns made of channels and plates* have good distribution of metal. Their chief disadvantage is the difficulty of making connections. All rivets in connections, except those which go through the flanges of the channels, must be driven before the plates and channels are assembled. The section *o*, having three webs, has the same difficulty of fabrication as section *k*. Objection is sometimes made to the closed box section. This is discussed later.

Sections p. and q. Section *p* is the standard *Z-bar column,* and section *q* is the *Z-bar column with flange plates.* The distribution of metal is not as good as in channel columns and the connections are even more difficult. These sections were formerly much used but now only rarely.

Section r. The *standard* I-*beam* is not an economical column section but is used to meet special conditions. It is suitable when built into a solid masonry wall with its web perpendicular to the axis of the wall. It is thus supported sidewise continuously and can be designed in reference to its larger radius of gyration. In apartment or residence work it is sometimes so desirable to keep the column within the thickness of the partition that the lack of economy of the I-beam column is justified.

Sections s, t, u, and v. The columns *s, t, u,* and *v* are not much used. There are no serious objections to any of them, and they may have advantages in special situations. Quick service from stock material may require the use of these sections.

Section w. The *Carnegie* H-*sections* are designed especially for use as columns. There are only four sizes, viz, 4, 5, 6, and 8 inches, respectively, and only one weight for each size, consequently their range of usefulness is very limited. The radius of gyration about the axis *1-1* is greater than that about *2-2,* but the distribution of metal is as good as in any H-shaped column. They are economical because so little labor is required for fabricating them. Only the 6-inch and 8-inch sizes can be used where beams must be connected to the flanges.

Sections x and y. The *Bethlehem columns* have a large range of sizes and weights. If the H-section in *x* is not heavy enough for the load, it can be increased by riveting on flange plates as in *y*. The advantage of this type of column is economy of fabrication, the only riveting required being for connections, except when flange plates are used. A part of this advantage is lost in the heavier sections because all holes must be drilled, due to thickness of metal. The thick metal is not as strong nor as reliable as the thinner metal used in built-up sections.

Tables. No comprehensive set of tables giving the properties and strength of columns has been published. But there are many partial tables which are of great assistance in designing. These tables can be divided into three classes as follows: (1) tables giving the properties of the sections; (2) tables giving the values of the allowable unit stresses for different values of $\frac{l}{r}$; and (3) tables giving strength of columns of various sections and lengths.

Properties of Sections. The properties of sections needed are area A; radius of gyration r; moment of inertia I; and distance to extreme fiber c. (See p. 181). If the column is a single rolled section, its properties can be taken from the tables in the handbooks. The values for standard angles and I-beams are given in all the handbooks; for the Carnegie H-columns, in the Carnegie Pocket Companion, 1913 edition; and for the Bethlehem columns, in the Bethlehem handbook.

Built-up columns may be made up in such vast numbers of combinations that no complete or very extensive tables have been published. However, the more common sizes are given in some of the handbooks. The area A and the distance to the extreme fiber c are readily computed from the sizes of material used in the column. The Cambria and Carnegie (1913) handbooks give the radii of gyration r and the moments of inertia I for laced channel columns, plate and channel columns, and plate and angle columns. The Carnegie handbook (1903) gives these properties for Z-bar columns. Similar data for about the same range of sizes are given in a number of other books on steel construction.

Allowable Unit Stress. The allowable unit stress adopted for this work has been given and illustrated heretofore. Its formula is

$$P = 16{,}000 - 70\frac{l}{r}$$

This is sometimes known as the American Railway Engineers' formula and is hereinafter referred to as the A. R. E. formula

In the Carnegie Pocket Companion, (1913 Edition) pp. 254-5, are shown a table and a diagram which give the values of P as determined from several other formulas. The formula recommended by the American Bridge Company does not differ greatly from the A. R. E. formula and may be used (unless local building ordinances require otherwise).

The formula used by the Bethlehem Steel Company is

$$P = 16{,}000 - 55\frac{l}{r}$$

with a maximum value 13,000. The resulting unit stresses for values of $\frac{l}{r}$ greater than 45 are higher than given by the A. R. E. formula, and for values of $\frac{l}{r}$ greater than 65 are higher than given by the American Bridge Company formula.

It saves much time in designing to have the values of P worked out for the usual values of l and r. Table V gives the values of P for values of r ranging from 0.1 inch to 6.0 inches and for lengths ranging from 3 feet to 40 feet. Table VI gives the values of P for values of $\frac{l}{r}$ ranging from 30 to 150.

Strength of Columns. As indicated above, there has not been general agreement on the formula for the allowable unit stress, consequently the tables of strength of columns which have been published have been based on several different formulas.

The Bethlehem handbook gives the strength of Bethlehem H-columns computed from their formula given above. Table VII gives the strengths of these columns based on the A. R. E. formula. (Computed by the Bethlehem Steel Company, for use in Chicago.)

The Carnegie Pocket Companion (1913) gives tables for Carnegie H-columns, I-beam columns, channel columns, and plate and angle columns, based on the American Bridge Company formula.

Table VIII gives the strengths of channel columns based on the A. R. E. formula (computed by the American Bridge Company). The strengths of plate and angle columns based on the A. R. E. formula are given in a pamphlet "Specifications for Steel Structures" (Chicago Edition), published by the American Bridge Company and distributed by its Chicago office.

TABLE V

Unit Stress in Compression in Columns

For values of r from 0.1 to 6.0 and lengths from 3 feet to 40 feet. Unit Stresses are given in Thousands of Pounds per Square Inch.

Radius of Gyration	LENGTH OF COLUMN													
	3′	4′	5′	6′	7′	8′	9′	10′	11′	12′	13′	14′	15′	16′
0.1														
0.2														
0.3	7.6													
0.4	9.7	7.6	5.5											
0.5	11.0	9.3	7.6	5.9										
0.6	11.8	10.4	9.0	7.6	6.2									
0.7	12.4	11.2	10.0	8.8	7.6	6.4								
0.8	12.8	11.8	10.7	9.7	8.6	7.6	6.5	5.5						
0.9	13.2	12.3	11.3	10.4	9.5	8.5	7.6	6.7	5.7					
1.0	13.5	12.6	11.8	11.0	10.1	9.3	8.4	7.6	6.8	5.9				
1.1	13.7	12.9	12.2	11.4	10.6	9.9	9.1	8.4	7.6	6.8	6.1			
1.2	13.9	13.2	12.5	11.8	11.1	10.4	9.7	9.0	8.3	7.6	6.9	6.2	5.5	
1.3	14.1	13.4	12.8	12.1	11.5	10.8	10.2	9.5	8.9	8.2	7.6	6.9	6.3	5.7
1.4	14.2	13.6	13.0	12.4	11.8	11.2	10.6	10.0	9.4	8.8	8.2	7.6	7.0	6.4
1.5	14.3	13.8	13.2	12.6	12.1	11.5	11.0	10.4	9.8	9.3	8.7	8.2	7.6	7.0
1.6	14.4	13.9	13.4	12.8	12.3	11.8	11.3	10.7	10.2	9.7	9.2	8.6	8.1	7.6
1.7	14.5	14.0	13.5	13.0	12.5	12.0	11.5	11.1	10.6	10.1	9.6	9.1	8.6	8.1
1.8	14.6	14.1	13.7	13.2	12.7	12.3	11.8	11.3	10.9	10.4	9.9	9.5	9.0	8.5
1.9	14.7	14.2	13.8	13.3	12.9	12.5	12.0	11.6	11.1	10.7	10.2	9.8	9.4	8.9
2.0	14.7	14.3	13.9	13.5	13.1	12.6	12.2	11.8	11.4	11.0	10.5	10.1	9.7	9.3
2.1	14.8	14.4	14.0	13.6	13.2	12.8	12.4	12.0	11.6	11.2	10.8	10.4	10.0	9.6
2.2	14.8	14.5	14.1	13.7	13.3	12.9	12.6	12.2	11.8	11.4	11.0	10.6	10.3	9.9
2.3	14.9	14.5	14.2	13.8	13.4	13.1	12.7	12.3	12.0	11.6	11.2	10.9	10.5	10.2
2.4	14.9	14.6	14.2	13.9	13.5	13.2	12.8	12.5	12.1	11.8	11.4	11.1	10.7	10.4
2.5	15.0	14.7	14.3	14.0	13.6	13.3	13.0	12.6	12.3	12.0	11.6	11.3	11.0	10.6
2.6	15.0	14.7	14.4	14.1	13.7	13.4	13.1	12.8	12.4	12.1	11.8	11.5	11.1	10.8
2.7	15.1	14.8	14.4	14.1	13.8	13.5	13.2	12.9	12.6	12.3	12.0	11.6	11.3	11.0
2.8	15.1	14.8	14.5	14.2	13.9	13.6	13.3	13.0	12.7	12.4	12.1	11.8	11.5	11.2
2.9	15.1	14.8	14.5	14.3	14.0	13.7	13.4	13.1	12.8	12.5	12.2	11.9	11.7	11.4
3.0	15.2	14.9	14.6	14.3	14.0	13.8	13.5	13.2	12.9	12.6	12.4	12.1	11.8	11.5
Radius of Gyration	3′	4′	5′	6′	7′	8′	9′	10′	11′	12′	13′	14′	15′	16′

TABLE V (Continued)

Formula $P = 16{,}000 - 70\frac{l}{r}$

in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches

Unit stresses above the heavy zigzag line are values of $\frac{l}{r}$ from 125 to 150

LENGTH OF COLUMN														
17′	18′	19′	20′	21′	22′	23′	24′	25′	26′	27′	28′	29′	30′	Radius of Gyration
														0.1
														0.2
														0.3
														0.4
														0.5
														0.6
														0.7
														0.8
														0.9
														1.0
														1.1
														1.2
														1.3
5.8														1.4
6.5	5.9													1.5
7.1	6.5	6.0	5.5											1.6
7.6	7.1	6.6	6.1	5.6										1.7
8.1	7.6	7.1	6.7	6.2	5.7									1.8
8.5	8.0	7.6	7.2	6.7	6.3	5.8								1.9
8.9	8.4	8.0	7.6	7.2	6.8	6.3	5.9	5.5						2.0
9.2	8.8	8.4	8.0	7.6	7.2	6.8	6.4	6.0	5.6					2.1
9.5	9.1	8.7	8.4	8.0	7.6	7.2	6.8	6.4	6.1	5.7				2.2
9.8	9.4	9.1	8.7	8.3	8.0	7.6	7.2	6.9	6.5	6.1	5.8			2.3
10.0	9.7	9.3	9.0	8.6	8.3	7.9	7.6	7.2	6.9	6.5	6.2	5.8	5.5	2.4
10.3	9.9	9.6	9.3	8.9	8.6	8.3	7.9	7.6	7.3	6.9	6.6	6.3	5.9	2.5
10.5	10.2	9.9	9.5	9.2	8.9	8.6	8.2	7.9	7.6	7.3	6.9	6.6	6.3	2.6
10.7	10.4	10.1	9.8	9.5	9.2	8.8	8.5	8.2	7.9	7.6	7.3	7.0	6.7	2.7
10.9	10.6	10.3	10.0	9.7	9.4	9.1	8.8	8.5	8.2	7.9	7.6	7.3	7.0	2.8
11.1	10.8	10.5	10.2	9.9	9.6	9.3	9.0	8.8	8.5	8.2	7.9	7.6	7.3	2.9
11.2	11.0	10.7	10.4	10.1	9.8	9.6	9.3	9.0	8.7	8.4	8.2	7.9	7.6	3.0
17′	18′	19′	20′	21′	22′	23′	24′	25′	26′	27′	28′	29′	30′	Radius of Gyration

TABLE V (Continued)

Unit Stress in Compression in Columns

For values of r from 0.1 to 6.0 and lengths from 3 feet to 40 feet. Unit Stresses are given in Thousands of Pounds per Square Inch

Radius of Gyration	LENGTH OF COLUMN													
	8′	9′	10′	11′	12′	13′	14′	15′	16′	17′	18′	19′	20′	21′
3.1	13.8	13.6	13.3	13.0	12.7	12.5	12.2	11.9	11.7	11.4	11.1	10.8	10.6	10.3
3.2	13.9	13.6	13.4	13.1	12.8	12.6	12.3	12.1	11.8	11.5	11.3	11.0	10.7	10.5
3.3	14.0	13.7	13.4	13.2	12.9	12.7	12.4	12.2	11.9	11.7	11.4	11.2	10.9	10.6
3.4	14.0	13.8	13.5	13.3	13.0	12.8	12.5	12.3	12.0	11.8	11.5	11.3	11.1	10.8
3.5	14.1	13.8	13.6	13.4	13.1	12.9	12.6	12.4	12.2	11.9	11.7	11.4	11.2	11.0
3.6	14.1	13.9	13.7	13.4	13.2	13.0	12.7	12.5	12.3	12.0	11.8	11.6	11.3	11.1
3.7	14.2	14.0	13.7	13.5	13.3	13.0	12.8	12.6	12.4	12.1	11.9	11.7	11.5	11.2
3.8	14.2	14.0	13.8	13.6	13.3	13.1	12.9	12.7	12.5	12.2	12.0	11.8	11.6	11.4
3.9	14.3	14.1	13.8	13.6	13.4	13.2	13.0	12.8	12.5	12.3	12.1	11.9	11.7	11.5
4.0	14.3	14.1	13.9	13.7	13.5	13.3	13.1	12.8	12.6	12.4	12.2	12.0	11.8	11.6
4.1	14.4	14.2	13.9	13.7	13.5	13.3	13.1	12.9	12.7	12.5	12.3	12.1	11.9	11.7
4.2	14.4	14.2	14.0	13.8	13.6	13.4	13.2	13.0	12.8	12.6	12.4	12.2	12.0	11.8
4.3	14.4	14.2	14.0	13.8	13.6	13.5	13.3	13.1	12.9	12.7	12.5	12.3	12.1	11.9
4.4	14.5	14.3	14.1	13.9	13.7	13.5	13.3	13.1	12.9	12.8	12.6	12.4	12.2	12.0
4.5	14.5	14.3	14.1	13.9	13.8	13.6	13.4	13.2	13.0	12.8	12.6	12.4	12.3	12.1
4.6	14.5	14.4	14.2	14.0	13.8	13.6	13.4	13.3	13.1	12.9	12.7	12.5	12.3	12.2
4.7	14.6	14.4	14.2	14.0	13.8	13.7	13.5	13.3	13.1	13.0	12.8	12.6	12.4	12.2
4.8	14.6	14.4	14.2	14.1	13.9	13.7	13.5	13.4	13.2	13.0	12.8	12.7	12.5	12.3
4.9	14.6	14.5	14.3	14.1	13.9	13.8	13.6	13.4	13.3	13.1	12.9	12.7	12.6	12.4
5.0	14.7	14.5	14.3	14.1	14.0	13.8	13.6	13.5	13.3	13.1	13.0	12.8	12.6	12.5
5.1	14.7	14.5	14.3	14.2	14.0	13.9	13.7	13.5	13.4	13.2	13.0	12.9	12.7	12.5
5.2	14.7	14.5	14.4	14.2	14.1	13.9	13.7	13.6	13.4	13.2	13.1	12.9	12.8	12.6
5.3	14.7	14.6	14.4	14.3	14.1	13.9	13.8	13.6	13.5	13.3	13.1	13.0	12.8	12.7
5.4	14.7	14.6	14.4	14.3	14.1	14.0	13.8	13.7	13.5	13.3	13.2	13.0	12.9	12.7
5.5	14.8	14.6	14.5	14.3	14.2	14.0	13.9	13.7	13.6	13.4	13.2	13.1	12.9	12.8
5.6	14.8	14.6	14.5	14.3	14.2	14.0	13.9	13.7	13.6	13.4	13.3	13.1	13.0	12.8
5.7	14.8	14.7	14.5	14.4	14.2	14.1	13.9	13.8	13.6	13.5	13.3	13.2	13.0	12.9
5.8	14.8	14.7	14.5	14.4	14.3	14.1	14.0	13.8	13.7	13.5	13.4	13.2	13.1	13.0
5.9	14.9	14.7	14.6	14.4	14.3	14.1	14.0	13.9	13.7	13.6	13.4	13.3	13.1	13.0
6.0	14.9	14.7	14.6	14.5	14.3	14.2	14.0	13.9	13.8	13.6	13.5	13.3	13.2	13.1
Radius of Gyration	8′	9′	10′	11′	12′	13′	14′	15′	16′	17′	18′	19′	20′	21′

TABLE V (Continued)

Formula $P = 16{,}000 - 70\frac{l}{r}$

in which P = unit stress in pounds per square inch
r = radius of gyration in inches.
l = length in inches.

Unit stresses above the heavy zigzag line are values of $\frac{l}{r}$ from 125 to 150

LENGTH OF COLUMN														Radius of Gyration
22′	23′	24′	25′	26′	27′	28′	29′	30′	32′	34′	36′	38′	40′	
10.0	9.8	9.5	9.2	8.9	8.7	8.4	8.1	7.9	7.3	6.8	6.2	5.7		3.1
10.2	10.0	9.7	9.4	9.2	8.9	8.6	8.4	8.1	7.6	7.1	6.5	6.0	5.5	3.2
10.4	10.1	9.9	9.6	9.4	9.1	8.9	8.6	8.4	7.8	7.3	6.8	6.3	5.8	3.3
10.6	10.3	10.1	9.8	9.6	9.3	9.1	8.8	8.6	8.1	7.6	7.1	6.6	6.1	3.4
10.7	10.5	10.2	10.0	9.8	9.5	9.3	9.0	8.8	8.3	7.8	7.4	6.9	6.4	3.5
10.9	10.6	10.4	10.2	9.9	9.7	9.5	9.2	9.0	8.5	8.1	7.6	7.1	6.7	3.6
11.0	10.8	10.5	10.3	10.1	9.9	9.6	9.4	9.2	8.7	8.3	7.8	7.4	6.9	3.7
11.1	10.9	10.7	10.5	10.2	10.0	9.8	9.6	9.4	8.9	8.5	8.0	7.6	7.2	3.8
11.3	11.0	10.8	10.6	10.4	10.2	10.0	9.7	9.5	9.1	8.7	8.2	7.8	7.4	3.9
11.4	11.2	11.0	10.7	10.5	10.3	10.1	9.9	9.7	9.3	8.9	8.4	8.0	7.6	4.0
11.5	11.3	11.1	10.9	10.7	10.5	10.3	10.1	9.8	9.4	9.0	8.6	8.2	7.8	4.1
11.6	11.4	11.2	11.0	10.8	10.6	10.4	10.2	10.0	9.6	9.2	8.8	8.4	8.0	4.2
11.7	11.5	11.3	11.1	10.9	10.7	10.5	10.3	10.1	9.7	9.4	9.0	8.6	8.2	4.3
11.8	11.6	11.4	11.2	11.0	10.8	10.7	10.5	10.3	9.9	9.5	9.1	8.7	8.4	4.4
11.9	11.7	11.5	11.3	11.1	11.0	10.8	10.6	10.4	10.0	9.6	9.3	8.9	8.5	4.5
12.0	11.8	11.6	11.4	11.2	11.1	10.9	10.7	10.5	10.2	9.8	9.4	9.1	8.7	4.6
12.1	11.9	11.7	11.5	11.3	11.2	11.0	10.8	10.6	10.3	9.9	9.6	9.2	8.8	4.7
12.1	12.0	11.8	11.6	11.4	11.3	11.1	10.9	10.7	10.4	10.0	9.7	9.3	9.0	4.8
12.2	12.1	11.9	11.7	11.5	11.4	11.2	11.0	10.9	10.5	10.2	9.8	9.5	9.1	4.9
12.3	12.1	12.0	11.8	11.6	11.5	11.3	11.1	11.0	10.6	10.3	9.9	9.6	9.3	5.0
12.4	12.2	12.0	11.9	11.7	11.5	11.4	11.2	11.1	10.7	10.4	10.1	9.7	9.4	5.1
12.4	12.3	12.1	12.0	11.8	11.6	11.5	11.3	11.1	10.8	10.5	10.2	9.9	9.5	5.2
12.5	12.3	12.2	12.0	11.9	11.7	11.6	11.4	11.2	10.9	10.6	10.3	10.0	9.7	5.3
12.6	12.4	12.3	12.1	11.9	11.8	11.6	11.5	11.3	11.0	10.7	10.4	10.1	9.8	5.4
12.6	12.5	12.3	12.2	12.0	11.9	11.7	11.6	11.4	11.1	10.8	10.5	10.2	9.9	5.5
12.7	12.5	12.4	12.2	12.1	11.9	11.8	11.6	11.5	11.2	10.9	10.6	10.3	10.0	5.6
12.8	12.6	12.5	12.3	12.2	12.0	11.9	11.7	11.6	11.3	11.0	10.7	10.4	10.1	5.7
12.8	12.7	12.5	12.4	12.2	12.1	11.9	11.8	11.6	11.4	11.1	10.8	10.5	10.2	5.8
12.9	12.7	12.6	12.4	12.3	12.2	12.0	11.9	11.7	11.4	11.2	10.9	10.6	10.3	5.9
12.9	12.8	12.6	12.5	12.4	12.2	12.1	11.9	11.8	11.5	11.2	11.0	10.7	10.4	6.0
22′	23′	24′	25′	26′	27′	28′	29′	30′	32′	34′	36′	38′	40′	Radius of Gyration

TABLE VI

Unit Stress in Compression

Values of P for values of $\frac{l}{r}=30$ to $\frac{l}{r}=150$ from the formula $P=16{,}000-70\frac{l}{r}$

$\frac{l}{r}$	$16{,}000-70\frac{l}{r}$ 14,000 max.	$\frac{l}{r}$	$16{,}000-70\frac{l}{r}$ 14,000 max.
30	13900	105	8650
35	13550	110	8300
40	13200	115	7950
45	12850	120	7600
50	12500	125	7250
55	12150	130	6900
60	11800	135	6550
65	11450	140	6200
70	11100	145	5850
75	10750	150	5500
80	10400		
85	10050		
90	9700		
95	9350		
100	9000		

It must be noted that the tables of strength of columns take no account of eccentricity. If there are eccentric loads, the equivalent concentric loads must be computed by the method given on p. 178, and then these values added to the actual loads. This result gives the total load to be used in selecting the column section from the tables.

Use of the Tables. The following illustrations show the manner of using the tables:

(1) Assume a concentric load of 492,000 pounds on a column 12 feet long. Determine the required column sections made of plates and channels and of plates and angles. Compare the areas of the two columns.

(a) From Table VIII, the channel column section required is

2 [12″×30″
2 Pl. 14″×$\frac{11}{16}$″
Area = 36.9 sq. in.

(b) From the table of plate and angle columns (see handbook), the angle column section required is

1 Pl. $12'' \times \frac{1}{2}''$
4 Ls $6'' \times 4'' \times \frac{1}{2}''$
2 Pl. $14'' \times \frac{1}{2}''$
Area = 39.0 sq. in.

In both cases other sections might be selected.

(2) Assume a load of 640,000 pounds on a column 16 feet long; 80,000 pounds of the load has an eccentricity of 9 inches in the direction of the greatest radius of gyration. Determine the plate and angle column required, using the A. R. E. formula.

A preliminary selection from the table indicates a column whose greatest r is about 6.8 inches and whose c is about $8\frac{1}{2}$ inches. From these approximate values the concentric equivalent load is

$$W_e = \tfrac{3}{4} \times 80{,}000 \times \frac{9 \times 8\frac{1}{2}}{6.8 \times 6.8} = 99{,}000\#$$

This added to the direct load gives a total of 739,000 pounds. The column section required is

1 Pl. $14'' \times \frac{5}{8}''$
4 Ls $6'' \times 4'' \times \frac{5}{8}''$
2 Pl. $14'' \times 1\frac{1}{16}''$

The values of r and c for this section are 6.83 inches and $8\frac{5}{16}$ inches, so the approximate values of r and c used above are accurate enough, hence no corrections need be made.

(3) Assume a column which has an unsupported length of 10 feet 6 inches in its weaker direction and 18 feet in its stronger direction made of

1 Pl. $12'' \times \frac{3}{4}''$
4 Ls $6'' \times 4'' \times \frac{3}{4}''$

Determine the allowable unit stress.

From the table the values of r are 2.69 and 4.91. The corresponding values of l are 126 inches and 216 inches; and of $\frac{l}{r}$ are 43 and 44. The respective unit stresses are taken from Table VI by interpolating between the values for 40 and 45, giving 13,590 and 13,720. The smaller value must be used.

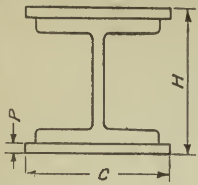


TABLE VII

Safe Loads on Bethlehem Columns

14″ H-Section with Cover Plates

Safe Loads are given in Thousands of Pounds

Weight Section Lb. per Ft.	Dimensions, in. C	P	H	Area of Section Square Inches	Least Radius of Gyr., Inches	UNSUPPORTED LENGTH 10	11	11	13	14	15
284.0	16	1¼	16⅝	83.52	3.98	1160.0	1142.4	1124.8	1107.2	1089.6	1072.0
290.8	16	1 5/16	16¾	85.52	3.99	1188.2	1170.2	1152.2	1134.2	1116.2	1098.2
297.6	16	1⅜	16⅞	87.52	4.01	1217.0	1198.6	1180.4	1162.0	1143.6	1125.4
304.4	16	1 7/16	17	89.52	4.02	1245.2	1226.6	1207.8	1189.2	1170.4	1151.8
311.2	16	1½	17⅛	91.52	4.04	1274.0	1255.0	1236.0	1207.0	1198.0	1178.8
318.0	16	1 9/16	17¼	93.52	4.05	1302.4	1283.0	1263.6	1244.2	1224.8	1205.4
324.8	16	1⅝	17⅜	95.52	4.06	1330.6	1311.0	1291.2	1271.4	1251.6	1231.8
331.6	16	1 11/16	17½	97.52	4.08	1359.6	1339.4	1319.4	1299.4	1279.2	1259.2
338.4	16	1¾	17⅝	99.52	4.09	1388.0	1367.4	1347.0	1326.6	1306.2	1285.8
345.2	16	1 13/16	17¾	101.52	4.10	1416.4	1395.6	1374.8	1354.0	1333.2	1312.4
350.3	17	1¾	17⅝	103.02	4.30	1447.0	1427.0	1406.8	1386.6	1366.6	1346.4
357.5	17	1 13/16	17¾	105.15	4.31	1477.4	1457.0	1436.4	1416.0	1395.4	1375.0
364.7	17	1⅞	17⅞	107.27	4.32	1507.8	1486.0	1466.0	1445.2	1424.4	1403.4
372.0	17	1 15/16	18	109.40	4.33	1538.2	1517.0	1495.8	1474.6	1453.2	1432.0
379.2	17	2	18⅛	111.52	4.35	1569.0	1547.4	1526.0	1504.4	1482.8	1461.2
386.4	17	2 1/16	18¼	113.65	4.36	1599.4	1577.6	1555.6	1533.8	1511.8	1490.0
393.6	17	2⅛	18⅜	115.72	4.37	1629.8	1607.6	1585.2	1563.0	1440.8	1518.6
400.9	17	2 3/16	18½	117.90	4.38	1660.2	1637.6	1615.0	1592.4	1569.8	1547.2
408.1	17	2¼	18⅝	120.02	4.39	1690.6	1667.8	1644.8	1621.8	1598.8	1575.8
415.3	17	2 5/16	18¾	122.15	4.40	1721.2	1697.8	1674.6	1651.2	1628.0	1604.6
423.4	18	2¼	18⅝	124.52	4.62	1766.0	1743.2	1720.6	1698.0	1675.4	1652.8
431.0	18	2 5/16	18¾	126.77	4.63	1798.4	1775.4	1752.4	1729.4	1706.4	1683.4
438.7	18	2⅜	18⅞	129.02	4.64	1830.8	1807.4	1784.0	1760.6	1737.4	1714.0
446.3	18	2 7/16	19	131.27	4.65	1863.2	1839.4	1814.8	1792.0	1768.4	1744.6
454.0	18	2½	19⅛	133.52	4.66	1895.6	1871.6	1847.6	1823.4	1799.4	1775.4
461.6	18	2 9/16	19¼	135.77	4.67	1928.2	1903.6	1879.2	1854.8	1830.4	1806.0
469.3	18	2⅝	19⅜	138.02	4.68	1960.6	1935.8	1911.0	1886.2	1861.6	1836.8
476.9	18	2 11/16	19½	140.27	4.69	1993.0	1968.0	1942.8	1917.8	1892.6	1867.4
484.6	18	2¾	19⅝	142.52	4.70	2025.6	2000.2	1974.6	1949.2	1923.8	1898

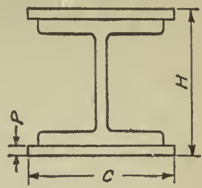
Columns composed of a 14″×148# Special Column Section (H14b) reënforced with cover plates of width and thickness given in table.

TABLE VII (Continued)

Formula $P = 16{,}000 - 70\frac{l}{r}$

in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches

To the left of heavy line values of $\frac{l}{r}$ do not exceed 120



OF COLUMNS IN FEET									Weight Section Lb. per Ft.
16	18	20	22	24	28	32	36	40	
1054.2	1019.0	983.8	948.6	913.2	842.8	772.2	701.8	631.2	284.0
1080.2	1044.2	1008.2	972.2	936.2	864.2	792.2	720.2	648.2	290.8
1107.0	1070.4	1033.6	997.0	960.4	887.0	813.6	740.4	667.0	297.6
1133.0	1095.6	1058.2	1020.8	983.4	908.6	833.8	759.0	684.0	304.4
1159.8	1121.8	1083.8	1045.6	1007.6	931.6	855.4	779.2	703.2	311.2
1186.0	1147.2	1108.4	1069.6	1030.8	953.2	875.6	798.0	720.4	318.0
1212.2	1172.6	1133.0	1093.6	1054.0	975.0	896.0	816.8	737.8	324.8
1239.0	1199.0	1158.8	1118.6	1078.4	998.2	917.8	837.6	757.2	331.6
1265.2	1225.4	1183.6	1142.6	1101.8	1020.0	938.2	856.6	774.8	338.4
1291.6	1250.0	1208.4	1166.8	1125.2	1042.0	958.8	875.6	792.4	345.2
1326.4	1286.0	1245.8	1205.6	1165.4	1084.8	1004.4	923.8	843.4	350.3
1354.6	1313.6	1272.6	1231.6	1190.6	1108.6	1026.6	944.6	862.6	357.5
1382.6	1340.8	1299.2	1257.4	1215.8	1132.2	1048.8	965.4	882.0	364.7
1410.8	1368.4	1326.0	1283.4	1241.0	1156.2	1071.2	986.4	901.4	372.0
1439.8	1396.6	1353.6	1310.6	1267.4	1181.4	1095.2	1009.0	923.0	379.2
1468.0	1424.2	1380.4	1336.6	1292.8	1205.4	1117.8	1030.2	942.6	386.4
1496.2	1451.8	1407.2	1362.8	1318.2	1229.2	1140.2	1051.2	962.2	393.6
1524.6	1479.4	1434.2	1389.0	1343.8	1253.2	1162.8	1072.4	982.0	400.9
1552.8	1507.0	1461.0	1415.0	1369.2	1277.2	1185.4	1093.6	1001.8	408.1
1581.2	1534.6	1488.0	1441.4	1394.8	1301.4	1208.2	1115.0	1021.6	415.3
1630.0	1584.8	1539.6	1494.2	1449.0	1358.4	1267.8	1177.2	1086.8	423.4
1660.4	1614.4	1568.4	1522.4	1476.4	1384.4	1292.4	1200.4	1108.4	431.0
1690.6	1643.8	1597.2	1550.4	1503.8	1410.4	1316.8	1223.4	1130.0	438.7
1721.0	1673.4	1626.0	1578.6	1531.2	1436.4	1341.4	1246.6	1151.8	446.3
1751.2	1703.0	1655.0	1606.8	1558.6	1462.4	1366.2	1269.8	1173.6	454.0
1781.6	1732.8	1683.8	1635.0	1586.2	1488.6	1390.8	1293.2	1195.4	461.6
1812.0	1762.4	1712.8	1663.4	1613.8	1514.6	1415.6	1316.4	1217.4	469.3
1842.4	1792.2	1741.8	1691.6	1641.4	1540.8	1440.4	1339.8	1239.4	476.9
1872.8	1821.8	1770.8	1720.0	1669.0	1567.0	1465.2	1363.4	1261.4	484.6

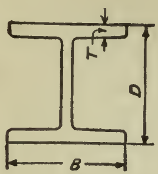


TABLE VII (Continued)

Safe Loads on Bethlehem Columns

14″ H-Section

Safe loads are given in Thousands of Pounds

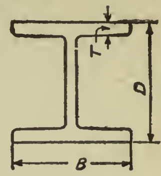
Section Number	Weight Section Lb. per Ft.	Dimensions, Inches				Least Radius of Gyr., Inches	UNSUPPORTED LENGTH				
		D	T	B	Area of Section Sq. In.		10	11	12	13	14
	83.5	13¾	11/16	13.92	24.46	3.47	332.2	326.2	320.4	314.4	308.4
	91.0	13⅞	¾	13.96	26.76	3.49	363.8	357.4	350.8	344.4	338.0
	99.0	14	13/16	14.00	29.06	3.50	395.2	388.2	381.2	374.2	367.4
	106.5	14⅛	⅞	14.04	31.38	3.52	427.2	419.8	412.2	404.8	397.2
	114.5	14¼	15/16	14.08	33.70	3.53	459.0	451.0	443.0	435.0	427.0
	122.5	14⅜	1	14.12	36.04	3.55	491.4	482.8	474.4	465.8	457.2
	130.5	14½	1 1/16	14.16	38.38	3.56	523.6	514.4	505.4	496.4	487.2
	138.0	14⅝	1⅛	14.19	40.59	3.58	554.2	544.6	535.2	525.6	516.2
	146.0	14¾	1 3/16	14.23	42.95	3.59	586.8	576.6	566.6	556.6	546.6
	154.0	14⅞	1¼	14.27	45.33	3.61	619.8	609.2	598.8	588.2	577.6
	162.0	15	1 5/16	14.31	47.71	3.62	652.6	641.6	630.6	619.4	608.4
	170.5	15⅛	1⅜	14.35	50.11	3.64	686.2	674.6	663.0	651.4	639.8
	178.5	15¼	1 7/16	14.39	52.51	3.65	719.4	707.2	695.2	683.0	671.0
H14	186.5	15⅜	1½	14.43	54.92	3.66	752.6	740.0	727.4	714.8	702.2
	195.0	15½	1 9/16	14.47	57.35	3.68	786.6	773.6	760.6	747.4	734.4
	203.5	15⅝	1⅝	14.51	59.78	3.69	820.4	806.8	793.2	779.6	766.0
	211.0	15¾	1 11/16	14.54	62.07	3.70	852.2	838.2	824.0	810.0	795.8
	219.5	15⅞	1¾	14.58	64.52	3.71	886.2	871.6	857.0	842.4	827.8
	227.5	16	1 13/16	14.62	66.98	3.72	920.4	905.4	890.2	875.0	860.0
	236.0	16⅛	1⅞	14.66	69.45	3.74	955.2	939.6	924.0	908.4	892.8
	244.5	16¼	1 15/16	14.70	71.94	3.75	989.8	973.8	957.6	941.6	925.4
	253.0	16⅝	2	14.74	74.43	3.76	1024.6	1008.0	991.4	974.8	958.0
	261.5	16½	2 1/16	14.78	76.93	3.77	1059.4	1042.4	1025.2	1008.0	991.0
	270.0	16⅝	2⅛	14.82	79.44	3.79	1095.0	1077.4	1059.8	1042.2	1024.6
	278.5	16¾	2 3/16	14.86	81.97	3.80	1130.4	1112.2	1094.0	1076.0	1057.8
	287.5	16⅞	2¼	14.90	84.50	3.81	1165.8	1147.0	1128.4	1109.8	1091.2

TABLE VII (Continued)

Formula $P = 16{,}000 - 70\frac{l}{r}$

in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches

To the left of heavy line values of $\frac{l}{r}$ do not exceed 120



OF COLUMNS IN FEET

15	16	18	20	22	24	28	32	36	40	Weight Section Lb. per Ft.
302.6	296.6	284.8	273.0	261.0	249.2	225.6	201.8	178.2	154.6	83.5
331.6	325.2	312.2	299.4	286.4	273.6	247.8	222.0	196.2	170.6	91.0
360.4	353.4	339.4	325.4	311.6	297.6	269.6	241.8	213.8	186.0	99.0
389.8	382.2	367.2	352.4	337.4	322.4	292.4	262.4	232.4	202.6	106.5
419.0	410.8	394.8	378.8	362.8	346.8	314.6	282.6	250.4	218.4	114.5
448.8	440.2	423.2	406.0	389.0	372.0	337.8	303.8	269.6	235.6	122.5
478.2	469.2	451.0	433.0	414.8	396.8	360.6	324.2	288.0	251.8	130.5
506.6	497.0	478.0	459.0	440.0	420.8	382.8	344.6	306.6	268.4	138.0
536.4	526.4	506.2	486.2	466.2	446.0	405.8	365.6	325.4	285.2	146.0
567.0	556.6	535.4	514.4	493.2	472.2	430.0	387.8	345.6	303.4	154.0
597.2	586.2	564.0	542.0	519.8	497.6	453.4	409.0	364.8	320.6	162.0
628.4	616.8	593.6	570.4	547.4	524.2	478.0	431.8	385.4	339.2	170.5
658.8	646.8	622.6	598.4	574.4	550.2	501.8	453.4	405.2	356.8	178.5
689.6	677.0	651.8	626.6	601.4	576.2	525.8	475.4	425.0	374.6	186.5
721.2	708.2	682.0	655.8	629.6	603.4	551.0	498.6	446.4	394.0	195.0
752.4	738.8	711.6	684.4	657.0	629.8	575.4	521.0	466.6	412.2	203.5
781.8	767.6	739.4	711.2	683.2	655.0	598.6	542.2	485.8	429.4	211.0
813.2	798.6	769.4	740.2	711.0	681.8	623.2	564.8	506.4	448.0	219.5
844.8	829.6	799.4	769.2	739.0	708.6	648.2	587.6	527.2	466.8	227.5
877.2	861.6	830.4	799.2	768.0	736.8	674.4	612.0	549.6	487.2	236.0
909.4	893.2	861.0	828.8	796.6	764.2	699.8	635.4	570.8	506.4	244.5
941.4	924.8	891.6	858.4	825.0	791.8	725.2	658.8	592.2	525.8	253.0
973.8	956.6	922.4	888.0	853.8	819.4	751.0	682.4	613.8	545.2	261.5
1007.0	989.4	954.2	919.0	883.6	848.4	778.0	707.6	637.2	566.8	270.0
1039.8	1021.6	985.4	949.2	912.8	876.6	804.2	731.6	659.2	586.8	278.5
1072.6	1054.0	1016.6	979.4	942.2	904.8	830.4	755.8	681.4	606.8	287.5

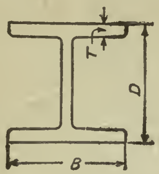


TABLE VII (Continued)

Safe Loads on Bethlehem Columns

12″ H-Section

Safe loads are given in Thousands of Pounds

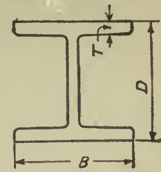
Section Number	Weight Section Lb. per Foot	Dimension, Inches D	T	B	Area of Section Sq. In.	Least Radius of Gyr. Inches	UNSUPPORTED LENGTH 10	11	12	13	14
	64.5	11¾	⅝	11.92	19.00	2.98	250.4	245.0	239.8	234.4	229.0
	71.5	11⅞	11/16	11.96	20.96	3.00	276.6	270.8	265.0	259.0	253.2
	78.0	12	¾	12.00	22.94	3.01	303.0	296.6	290.2	283.8	277.4
	84.5	12⅛	13/16	12.04	24.92	3.03	329.6	322.8	315.8	309.0	302.0
	91.5	12¼	⅞	12.08	26.92	3.04	356.4	348.8	341.4	334.0	326.6
	98.5	12⅜	15/16	12.12	28.92	3.06	383.4	375.4	367.4	359.6	351.6
	105.0	12½	1	12.16	30.94	3.07	410.4	402.0	393.4	385.0	376.6
H12	112.0	12⅝	1 1/16	12.20	32.96	3.08	437.4	428.4	419.4	410.6	401.6
	118.5	12¾	1⅛	12.23	34.87	3.10	463.4	454.0	444.6	435.0	425.6
	125.5	12⅞	1 3/16	12.27	36.91	3.11	490.8	480.8	471.0	461.0	451.0
	132.5	13	1¼	12.31	38.97	3.13	519.0	508.4	498.0	487.6	477.2
	139.5	13⅛	1 5/16	12.35	41.03	3.14	546.8	535.8	524.8	513.8	502.8
	146.5	13¼	1⅜	12.39	43.10	3.15	574.6	563.2	551.6	540.2	528.6
	153.5	13⅜	1 7/16	12.43	45.19	3.16	603.0	591.0	578.8	566.8	554.8
	161.0	13½	1½	12.47	47.28	3.18	631.6	619.2	606.6	594.2	581.6

TABLE VII (Continued)

Formula $P=16{,}000-70\frac{l}{r}$

in which P = unit stress in pounds per square inches
r = radius of gyration in inches
l = length in inches

To the left of heavy line values of $\frac{l}{r}$ do not exceed 120



OF COLUMNS IN FEET									Weight Section Lb. per Foot
15	16	18	20	22	24	28	32	36	
223.6	218.4	207.6	196.8	186.2	165.4	154.0	132.6	111.2	64.5
247.4	241.4	229.8	218.0	206.2	194.6	171.0	147.6	124.0	71.5
271.0	264.6	251.8	239.0	226.2	213.4	187.8	162.2	136.6	78.0
295.0	288.2	274.4	260.6	246.8	233.0	205.2	177.6	150.0	84.5
319.2	311.8	296.8	282.0	267.0	252.2	222.4	192.6	163.0	91.5
343.6	335.6	319.8	304.0	288.0	272.2	240.4	208.6	177.0	98.5
368.0	359.6	342.6	325.8	308.8	291.8	258.0	224.2	190.2	105.0
392.6	383.6	365.6	347.6	329.6	311.6	375.6	239.8	203.8	112.0
416.2	406.8	387.8	369.0	350.0	331.2	293.4	255.6	217.8	118.5
441.0	431.0	411.2	391.2	371.2	351.2	311.4	271.6	231.6	125.5
466.6	456.2	435.2	414.4	393.4	372.6	330.6	288.8	247.0	132.5
491.8	480.8	459.0	437.0	415.0	393.0	349.2	305.2	261.4	139.5
517.2	505.8	482.8	459.8	436.8	413.8	367.8	321.8	275.8	146.5
542.8	530.8	506.8	482.8	458.8	434.8	386.6	338.6	290.6	153.5
569.2	556.6	531.6	506.6	481.8	456.8	406.8	356.8	306.8	161.0

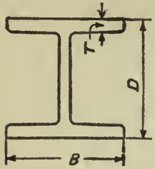


TABLE VII (Continued)

Safe Loads on Bethlehem Columns

10″ H-Section

Safe loads are given in Thousands of Pounds

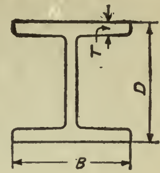
Section Number	Weight Section Lb. per Ft.	Dimensions, Inches			Area of Section Square Inches	Least Radius of Gyr. Inches	UNSUPPORTED LENGTH				
		D	T	B			10	11	12	13	14
	49.0	9⅞	9/16	9.97	14.37	2.49	181.4	176.6	171.8	167.0	162.0
	54.0	10	⅝	10.00	15.91	2.51	201.4	196.0	190.6	185.4	180.0
	59.5	10⅛	11/16	10.04	17.57	2.53	222.8	217.0	211.2	205.2	199.4
	65.5	10¼	¾	10.08	19.23	2.54	244.0	237.8	231.4	225.0	218.6
	71.0	10⅜	13/16	10.12	20.91	2.56	266.0	259.0	252.2	245.4	238.6
	77.0	10½	⅞	10.16	22.59	2.57	287.6	280.2	272.8	265.4	258.0
	82.5	10⅝	15/16	10.20	24.29	2.58	309.6	301.6	293.8	285.8	278.0
	88.5	10¾	1	10.24	25.99	2.60	331.8	323.4	315.0	306.6	298.2
H10	94.0	10⅞	1 1/16	10.28	27.71	2.61	354.2	345.2	336.4	327.4	318.6
	99.5	11	1⅛	10.31	29.32	2.62	375.2	365.8	356.4	347.0	337.6
	105.5	11⅛	1 3/16	10.35	31.06	2.64	398.2	388.2	378.4	368.4	358.6
	111.5	11¼	1¼	10.39	32.80	2.65	420.8	410.4	400.0	389.6	379.2
	117.5	11⅜	1 5/16	10.43	34.55	2.66	443.6	432.8	421.8	411.0	400.0
	123.5	11½	1⅜	10.47	36.32	2.67	466.8	455.4	444.0	432.6	421 2

TABLE VII (Continued)

Formula $P=16{,}000-70\frac{l}{r}$

in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches

To the left of heavy line values of $\frac{l}{r}$ do not exceed 120



OF COLUMNS IN FEET									
15	16	18	20	22	24	26	28	30	Weight Section Lb. per Ft.
157.2	152.4	142.6	133.0	123.2	113.6	103.8	94.2	84.4	49.0
174.6	169.4	158.8	148.0	137.4	126.8	116.2	105.4	94.8	54.0
193.6	187.8	176.2	164.4	152.8	141.2	129.4	117.8	106.2	59.5
212.2	206.0	193.2	180.4	167.8	155.0	142.4	129.6	116.8	65.5
231.6	224.8	211.0	197.4	183.6	169.8	156.2	142.4	128.8	71.0
250.6	243.4	228.6	213.8	199.0	184.2	169.4	154.8	140.0	77.0
270.0	262.2	246.2	230.4	214.6	198.8	183.0	167.2	151.4	82.5
289.8	281.4	264.6	248.0	231.2	214.4	197.6	180.8	164.0	88.5
309.6	300.6	282.8	265.0	247.2	229.4	211.4	193.6	175.8	94.0
328.2	318.8	300.0	281.2	262.4	243.6	224.8	206.0	187.2	99.5
348.8	338.8	319.0	299.4	279.6	259.8	240.0	220.2	200.4	105.5
368.8	358.4	337.6	316.8	296.0	275.2	254.4	233.6	212.8	111.5
389.2	378.2	356.4	334.6	312.8	291.0	269.2	247.4	225.4	117.5
409.8	398.2	375.4	352.6	329.8	306.8	284.0	261.2	238.4	123.5

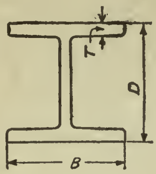


TABLE VII (Continued)

Safe Loads on Bethlehem Columns

8″ H-Section

Safe loads are given in Thousands of Pounds

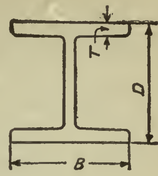
Section Number	Weight Section Lb. per Ft.	Dimensions, Inches			Area of Section Sq. In.	Least Radius of Gyr. Inches	UNSUPPORTED LENGTH				
		D	T	B			8	9	10	11	12
H8	31.5	7⅞	7/16	8.00	9.17	1.98	115.6	111.8	107.8	104.0	100.0
	34.5	8	½	8.00	10.17	2.01	128.8	124.4	120.2	116.0	111.8
	39.0	8⅛	9/16	8.04	11.50	2.03	146.0	141.2	136.4	131.6	126.8
	43.5	8¼	⅝	8.08	12.83	2.04	163.0	157.8	152.4	147.2	141.8
	48.0	8⅜	11/16	8.12	14.18	2.05	180.4	174.6	168.8	163.0	157.2
	53.0	8½	¾	8.16	15.53	2.07	198.0	191.8	185.4	179.2	172.8
	57.5	8⅝	13/16	8.20	16.90	2.08	215.8	209.0	202.2	195.4	188.4
	62.0	8¾	⅞	8.24	18.27	2.09	233.6	226.2	218.8	211.6	204.2
	67.0	8⅞	15/16	8.28	19.66	2.11	252.0	244.2	236.2	228.4	220.6
	71.5	9	1	8.32	21.05	2.12	270.0	261.8	253.4	245.0	236.8
	76.5	9⅛	1 1/16	8.36	22.46	2.13	288.4	279.6	270.8	262.0	253.0
	81.0	9¼	1⅛	8.39	23.78	2.14	305.8	296.4	287.2	277.8	268.4
	85.5	9⅜	1 3/16	8.43	25.20	2.16	324.8	315.0	305.2	295.4	285.6
	90.5	9½	1¼	8.47	26.64	2.17	343.8	333.4	323.2	312.8	302.4

TABLE VII (Continued)

Formula $P=16{,}000-70\frac{l}{r}$

in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches

To the left of heavy line values of $\frac{l}{r}$ do not exceed 120



OF COLUMNS IN FEET									Weight Section Lb. per Foot
13	14	15	16	17	18	20	22	24	
96.2	92.2	88.4	84.4	80.6	76.6	69.0	61.0	53.4	31.5
107.4	103.2	99.0	94.8	90.4	86.2	77.8	69.2	60.8	34.5
122.2	117.4	112.6	107.8	103.2	98.4	88.8	79.4	69.8	39.0
136.6	131.4	126.0	120.8	115.4	110.2	99.6	89.0	78.4	43.5
151.4	145.6	139.8	134.0	128.2	122.2	110.6	99.0	87.4	48.0
166.6	160.2	154.0	147.6	141.4	135.0	122.4	109.8	97.2	53.0
181.6	174.8	168.0	161.2	154.4	147.6	133.8	120.2	106.6	57.5
196.8	189.6	182.2	174.8	167.4	160.2	145.4	130.8	116.0	62.0
212.8	205.0	197.2	189.4	181.6	173.6	158.0	142.4	126.8	67.0
228.4	220.0	211.6	203.4	195.0	186.6	170.0	153.4	136.6	71.5
244.2	235.4	226.4	217.6	208.8	200.0	182.2	164.4	146.8	76.5
259.2	249.8	240.4	231.2	221.8	212.4	193.8	175.2	156.4	81.0
275.8	266.0	256.2	246.4	236.6	226.8	207.2	187.6	168.0	85.5
292.2	281.8	271.6	261.2	251.0	240.6	220.0	199.4	178.8	90.5

TABLE VII (Continued)

Safe Loads on Bethlehem Columns

Girder Beams Used as Columns

Safe loads are given in Thousands of Pounds

Section Number	Depth of Beam Inches	Weight per Foot Pounds	Area of Section Sq. In.	Least Rad. of Gyr. In.	UNSUPPORTED LENGTH 8	9	10	11	12
G30a	30	200	58.71	3.28	819.0	804.0	789.0	774.0	759.0
G30	30	180	53.00	2.86	723.4	708.0	692.4	676.8	661.2
G28a	28	180	52.86	3.18	734.0	720.0	706.2	692.2	678.2
G28	28	165	48.47	2.77	658.0	643.2	628.6	613.8	599.2
G26a	26	160	46.91	3.05	647.2	634.2	621.4	608.4	595.6
G26	26	150	43.94	2.68	592.8	579.0	565.4	551.6	537.8
G24a	24	140	41.16	2.90	563.2	551.2	539.4	527.4	515.4
G24	24	120	35.38	2.66	476.6	465.6	454.4	443.2	432.0
G20a	20	140	41.19	2.91	564.0	552.0	540.2	528.2	516.4
G20	20	112	32.81	2.70	443.2	433.0	422.8	412.6	402.4
G18	18	92	27.12	2.59	363.6	354.8	346.0	337.2	328.4
G15b	15	140	41.27	2.83	562.4	550.0	537.8	525.6	513.4
G15a	15	104	30.50	2.64	410.4	400.6	391.0	381.2	371.6
G15	15	73	21.49	2.39	283.4	275.8	268.4	260.8	253.2
G12a	12	70	20.58	2.36	270.6	263.4	256.0	248.8	241.4
G12	12	55	16.18	2.24	210.4	204.2	198.2	192.2	186.0
G10	10	44	12.95	2.10	165.8	160.6	155.4	150.2	145.0
G9	9	38	11.22	1.98	141.4	136.6	132.0	127.2	122.4
G8	8	32.5	9.54	1.86	118.2	113.8	109.6	105.2	101.0

Beams not secured against yielding sideways and free to fail in direction of least radius of gyration.

TABLE VII (Continued)

Formula $P=16{,}000-70\frac{l}{r}$

in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches.

To the left of heavy line values of $\frac{l}{r}$ do not exceed 120

OF COLUMNS IN FEET											Weight per Ft. Pounds
13	14	15	16	18	20	22	24	28	32	36	
743.8	728.8	713.8	698.8	668.8	638.6	608.6	578.6	518.4	458.2	398.0	200
645.6	630.0	614.6	599.0	567.8	536.6	505.6	474.4	412.2	349.8	287.6	180
664.2	650.2	636.4	622.4	594.4	566.4	538.6	510.6	454.8	399.0	343.0	180
584.4	569.8	555.0	540.4	511.0	481.6	452.2	422.8	364.0	305.2	......	165
582.6	569.6	556.8	543.8	518.0	492.2	466.4	440.4	388.8	337.2	......	160
524.0	510.2	496.4	482.6	455.2	427.6	400.0	372.6	317.4	262.4	......	150
503.6	491.6	479.8	467.8	444.0	420.2	396.2	372.4	324.8	277.0	......	140
420.8	409.6	398.4	387.4	365.0	342.6	320.2	298.0	253.2	208.6	......	120
504.4	492.6	480.6	468.8	445.0	421.2	397.4	373.6	326.2	278.6	......	140
392.2	382.0	371.8	361.6	341.2	320.8	300.4	280.0	239.2	198.4	......	112
319.6	310.8	302.0	293.2	275.6	258.0	240.4	222.8	187.6	152.4	......	92
501.0	488.8	476.6	464.4	439.8	415.4	390.8	366.4	317.4	268.4	......	140
361.8	352.2	342.4	332.8	313.4	294.0	274.4	255.0	216.2	177.4	......	104
245.6	238.0	230.6	223.0	207.8	192.8	177.6	162.6	132.4	......	......	73
234.0	226.8	219.4	212.0	197.4	182.8	168.2	153.4	124.2	......	......	70
180.0	174.0	167.8	161.8	149.6	137.6	125.4	113.2	89.0	......	......	55
139.8	134.6	129.4	124.4	114.0	103.6	93.2	82.8	......	......	......	44
117.6	112.8	108.2	103.4	93.8	84.4	74.8	......	......	......	......	38
96.6	92.4	88.0	83.8	75.0	66.4	57.8	......	......	......	......	32.5

TABLE VII (Continued)

Safe Loads on Bethlehem Columns

I-Beams Used as Columns

Safe Loads are given in Thousands of Pounds

Section Number	Depth of Beam Inches	Weight per Foot Pounds	Area of Section Sq. In.	Least Rad. of Gyr. In.	UNSUPPORTED LENGTH					
					5	6	7	8	9	10
B30	30	120	35.30	2.16	496.2	482.4	468.8	455.0	441.2	427.6
B28	28	105	30.88	2.06	431.2	418.6	406.0	393.4	380.8	368.2
B26	26	90	26.49	1.95	366.8	355.4	344.0	332.6	321.2	309.8
B24a	24	84	24.80	1.92	342.6	331.8	320.8	310.0	299.2	288.4
	24	83	24.59	1.78	335.4	323.8	312.2	300.6	289.0	277.4
B24	24	73	21.47	1.86	295.0	285.4	275.6	266.0	256.2	246.6
B20a	20	82	24.17	1.82	331.0	319.8	308.6	297.4	286.4	275.2
	20	72	21.37	1.88	294.2	284.6	275.0	265.6	256.0	246.4
	20	69	20.26	1.59	270.6	260.0	249.2	238.6	227.8	217.2
B20	20	64	18.86	1.62	252.8	243.0	233.4	223.6	213.8	204.0
	20	59	17.36	1.66	233.8	225.0	216.2	207.4	198.6	190.0
	18	59	17.40	1.50	229.6	220.0	210.2	200.4	190.8	181.0
B18	18	54	15.87	1.54	210.6	202.0	193.4	184.6	176.0	167.4
	18	52	15.24	1.56	202.8	194.6	186.4	178.2	170.0	161.8
	18	48.5	14.25	1.59	190.4	182.8	175.4	167.8	160.2	152.8
B15b	15	71	20.95	1.71	283.8	273.4	263.2	252.8	242.6	232.2
B15a	15	64	18.81	1.49	248.0	237.4	226.8	216.2	205.6	195.0
	15	54	15.88	1.55	211.0	202.4	193.8	185.2	176.6	168.0
	15	46	13.52	1.36	174.6	166.2	157.8	149.6	141.2	132.8
B15	15	41	12.02	1.41	156.6	149.4	142.2	135.0	127.8	120.8
	15	38	11.27	1.44	147.4	140.8	134.4	127.8	121.2	114.6
B12a	12	36	10.61	1.42	138.4	132.2	125.8	119.6	113.2	107.0
	12	32	9.44	1.30	120.6	114.4	108.4	102.2	96.2	90.0
B12	12	28.5	8.42	1.35	108.6	103.2	98.0	92.8	87.6	82.4
B10	10	28.5	8.34	1.21	104.4	98.8	93.0	87.2	81.4	75.6
	10	23.5	6.94	1.27	88.0	83.4	79.0	74.4	69.8	65.2

Beams not secured against yielding sideways and free to fail in direction of least radius of gyration.

TABLE VII (Continued)

Formula $P = 16{,}000 - 70\frac{l}{r}$

in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches

To the left of heavy line values of $\frac{l}{r}$ do not exceed 120

OF COLUMNS IN FEET										Weight per Foot Pounds
11	12	13	14	15	16	18	20	22	24	
413.8	400.0	386.4	372.6	358.8	345.2	317.6	290.2	262.8	235.4	120
355.6	343.0	330.4	317.8	305.2	292.6	267.4	242.2	217.0	191.8	105
298.4	287.0	275.4	264.0	252.6	241.2	218.4	195.6	172.8		90
277.4	266.6	255.8	245.0	234.0	223.2	201.6	179.8	158.2		84
265.8	254.2	242.6	231.0	219.4	207.8	184.6	161.4	138.2		83
236.8	227.2	217.4	207.8	198.0	188.4	169.0	149.6	130.2		73
264.0	252.8	241.6	230.6	219.4	208.2	186.0	163.6	141.4		82
236.8	227.4	217.8	208.2	198.6	189.2	170.0	151.0	131.8		72
206.4	195.8	185.0	174.4	163.6	153.0	131.4	110.0			69
194.2	184.4	174.6	164.8	155.0	145.2	125.8	106.2			64
181.2	172.4	163.6	154.8	146.0	137.2	119.6	102.0			59
171.2	161.4	151.8	142.0	132.2	122.4	103.0				59
158.6	150.0	141.4	132.8	124.0	115.4	98.2				54
153.6	145.4	137.2	129.0	120.8	112.6	96.2				52
145.2	137.6	130.2	122.6	115.0	107.6	92.4				48.5
222.0	211.8	201.4	191.2	180.8	170.6	150.0				71
184.4	173.8	163.2	152.4	141.8	131.2	110.0				64
159.4	150.8	142.2	133.6	125.0	116.4	99.2				54
124.4	116.2	107.8	99.4	91.0	82.8					46
113.6	106.4	99.2	92.0	85.0	77.8					41
108.0	101.4	94.8	88.2	81.8	75.2					38
100.8	94.4	88.2	81.8	75.6	69.4					36
84.0	77.8	71.8	65.6	59.6						32
77.0	71.8	66.6	61.4	56.2						28.5
69.8	64.0	58.2	52.4							28.5
60.6	56.0	51.4	46.8							23.5

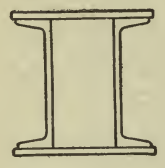


TABLE VIII

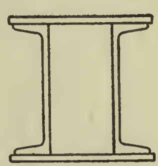
Safe Loads on Channel Columns
6″, 7″, 8″, 9″, and 10″ Channels

Safe Loads are given in Thousands of Pounds

2 [s	2 Pls.	r	Area 2 Pls.	Area Total	UNSUPPORTED LENGTH 8′	9′	10′	11′	12′	13′
6″-8#	Latt.	2.33		4.76	62	61	59	57	55	54
"	$8\times\frac{1}{4}$	2.32	4.00	8.76	115	112	108	105	102	99
"	$\frac{5}{16}$	2.32	5.00	9.76	128	124	121	117	114	110
7″-9¾#	Latt.	2.72		5.70	77	75	74	72	70	68
"	$9\times\frac{1}{4}$	2.67	4.50	10.20	138	134	131	128	125	122
"	$\frac{5}{16}$	2.67	5.63	11.33	153	149	145	142	138	135
8″-11¼#	Latt.	3.11		6.70	93	91	89	87	85	84
"	$10\times\frac{1}{4}$	3.03	5.00	11.70	161	158	155	152	148	145
"	$\frac{5}{16}$	3.02	6.25	12.95	178	175	171	168	164	160
"	$\frac{3}{8}$	3.01	7.50	14.20	196	192	188	184	180	176
8″-13¾#	Latt.	2.98		8.08	111	109	106	104	102	100
"	$10\times\frac{5}{16}$	2.97	6.25	14.33	197	193	189	185	181	177
"	$\frac{3}{8}$	2.96	7.50	15.58	214	210	205	201	196	192
9″-13¼#	Latt.	3.49		7.78	109	107	106	104	102	100
"	$11\times\frac{1}{4}$	3.40	5.50	13.28	186	183	180	176	173	170
"	$\frac{5}{16}$	3.38	6.88	14.66	205	202	198	195	191	187
"	$\frac{3}{8}$	3.36	8.25	16.03	224	220	216	212	208	204
9″-15#	Latt.	3.40		8.82	124	121	119	117	115	113
"	$11\times\frac{1}{4}$	3.36	5.50	14.32	200	197	193	190	186	183
"	$\frac{5}{16}$	3.34	6.88	15.70	220	216	212	208	204	200
"	$\frac{3}{8}$	3.33	8.25	17.07	239	235	230	226	221	217
10″-15#	Latt.	3.87		8.92	127	125	123	121	120	118
"	$12\times\frac{5}{16}$	3.74	7.50	16.42	233	230	226	222	218	215
"	$\frac{3}{8}$	3.72	9.00	17.92	254	250	246	242	238	234
"	$\frac{7}{16}$	3.70	10.50	19.42	275	271	267	262	258	253
"	$\frac{1}{2}$	3.68	12.00	20.92	296	292	287	282	277	272
10″-20#	Latt.	3.66		11.76	167	164	161	159	156	153
"	$12\times\frac{7}{16}$	3.64	10.50	22.26	315	310	305	300	295	289
"	$\frac{1}{2}$	3.63	12.00	23.76	336	331	325	320	314	309
"	$\frac{9}{16}$	3.62	13.50	25.26	357	351	346	340	334	328
"	$\frac{5}{8}$	3.61	15.00	26.76	378	372	366	360	354	348
					8′	9′	10′	11′	12′	13′

TABLE VIII (Continued)

Formula $P = 16{,}000 - 70\ \dfrac{l}{r}$



in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches

To left of heavy line values of $\dfrac{l}{r}$ do not exceed 125

To right of heavy line values of $\dfrac{l}{r}$ do not exceed 150

OF COLUMN IN FEET

14′	15′	16′	17′	18′	19′	20′	21′	22′	23′	24′	26′	28′	30′
52	50	49	47	45	43	41	40	38	36	35	32	28	
96	93	89	86	83	80	77	74	70	67	64	58	51	
107	103	100	96	93	89	86	82	78	75	71	64	57	
67	65	63	61	60	58	56	54	53	51	49	45	42	38
118	115	112	109	106	102	99	96	93	90	86	80	73	67
131	128	124	121	117	113	110	106	103	99	96	89	81	74
82	80	78	76	75	73	71	69	67	66	64	60	57	53
142	139	136	132	129	126	122	119	116	113	109	103	96	90
157	153	150	146	142	139	136	132	128	124	121	113	106	99
172	168	164	160	156	152	148	144	140	136	132	124	116	108
97	95	93	91	88	86	84	81	79	77	75	70	66	61
173	169	164	160	156	152	148	144	140	136	132	124	116	108
187	183	179	174	170	165	161	156	152	148	143	134	125	117
98	96	95	93	91	89	87	85	83	81	80	76	72	68
166	163	160	157	153	150	147	144	140	137	134	127	121	114
184	180	176	173	169	165	162	158	154	151	147	140	133	125
200	196	192	188	184	180	176	172	168	164	160	152	144	136
111	108	106	104	102	100	98	95	93	91	89	85	80	76
179	175	172	168	165	161	158	154	150	147	143	136	129	122
196	192	188	184	180	176	172	168	164	160	156	149	141	133
213	209	204	200	196	192	187	183	178	174	170	161	153	144
116	114	112	110	108	106	104	102	100	98	96	92	88	85
211	207	204	200	196	193	189	185	182	178	174	167	159	152
230	226	222	218	214	210	206	202	198	194	190	182	173	165
249	244	240	236	231	227	223	218	214	209	205	196	187	178
268	263	258	254	249	244	239	235	230	225	220	211	201	191
150	148	145	142	140	137	134	132	129	126	123	118	113	107
284	279	274	269	264	259	253	248	243	238	233	223	212	202
303	298	292	287	281	276	270	265	259	254	248	237	226	215
322	316	310	305	299	293	287	281	275	269	263	252	240	228
341	335	329	322	316	310	304	297	291	285	279	267	254	241
14′	15′	16′	17′	18′	19′	20′	21′	22′	23′	24′	26′	28′	30′

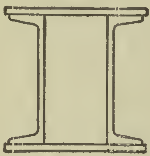


TABLE VIII (Continued)

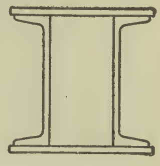
Safe Loads on Channel Columns
12″ Channels

Safe Loads are given in Thousands of Pounds

2 [s	2 Pls.	r	Area 2 Pls.	Area Total	UNSUPPORTED LENGTH 8′	9′	10′	11′	12′	13′
12″-20½#	Latt.	4.61		12.06	175	173	171	169	167	164
"	$14\times\frac{5}{16}$	4.40	8.75	20.81	301	297	293	289	285	281
"	$\frac{3}{8}$	4.38	10.50	22.56	326	322	317	313	309	304
"	$\frac{7}{16}$	4.35	12.25	24.31	352	346	342	337	333	328
"	$\frac{1}{2}$	4.33	14.00	26.06	377	371	366	361	356	351
12″-25#	Latt.	4.43		14.70	213	210	207	204	202	199
"	$14\times\frac{7}{16}$	4.30	12.25	26.95	389	384	378	373	368	363
"	$\frac{1}{2}$	4.29	14.00	28.70	414	409	403	397	392	386
"	$\frac{9}{16}$	4.27	15.75	30.45	439	433	427	421	415	409
"	$\frac{5}{8}$	4.26	17.50	32.20	464	458	452	445	439	433
"	$\frac{11}{16}$	4.25	19.25	33.95	489	483	476	469	463	456
"	$\frac{3}{4}$	4.24	21.00	35.70	514	507	500	493	486	479
12″-30#	Latt.	4.28		17.64	255	251	247	244	241	237
"	$14\times\frac{9}{16}$	4.23	15.75	33.39	481	474	468	461	455	448
"	$\frac{5}{8}$	4.22	17.50	35.14	506	499	492	485	478	471
"	$\frac{11}{16}$	4.21	19.25	36.89	531	524	517	509	502	494
"	$\frac{3}{4}$	4.20	21.00	38.64	556	549	541	533	525	518
"	$\frac{13}{16}$	4.20	22.75	40.39	582	574	566	557	549	541
"	$\frac{7}{8}$	4.19	24.50	42.14	607	598	590	581	573	564
"	$\frac{15}{16}$	4.18	26.25	43.89	632	623	614	605	597	588
"	1	4.18	28.00	45.64	657	648	639	630	620	611
12″-35#	Latt.	4.17		20.58	296	292	288	284	280	276
"	$14\times\frac{11}{16}$	4.17	19.25	39.83	573	565	557	549	541	533
"	$\frac{3}{4}$	4.16	21.00	41.58	598	590	581	573	565	556
"	$\frac{13}{16}$	4.16	22.75	43.33	623	614	606	597	588	579
"	$\frac{7}{8}$	4.15	24.50	45.08	648	639	630	621	612	603
"	$\frac{15}{16}$	4.15	26.25	46.83	674	664	655	645	635	626
"	1	4.14	28.00	48.58	699	689	679	669	659	649
"	$1\frac{1}{8}$	4.14	31.50	52.08	749	738	727	717	706	696
"	$1\frac{1}{4}$	4.13	35.00	55.58	799	787	776	765	753	742
					8′	9′	10′	11′	12′	13′

TABLE VIII (Continued)

Formula $P = 16{,}000 - 70\frac{l}{r}$



in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches

To left of heavy line values of $\frac{l}{r}$ do not exceed 125

To right of heavy line values of $\frac{l}{r}$ do not exceed 150

OF COLUMN IN FEET													
14′	15′	16′	17′	18′	19′	20′	21′	22′	23′	24′	26′	28′	30′
162	160	158	156	153	151	149	147	145	142	140	136	131	127
277	273	269	265	261	257	253	250	246	242	238	230	222	214
300	296	291	287	283	279	274	270	266	261	257	248	240	231
323	318	314	309	304	300	295	290	286	281	276	267	257	248
346	341	336	331	326	321	316	311	306	301	295	285	275	265
196	193	191	188	185	182	179	177	174	171	168	163	157	152
357	352	347	342	336	331	326	321	315	310	305	294	284	273
381	375	369	364	358	352	347	341	335	330	324	313	302	291
403	397	391	385	379	373	367	361	355	349	343	331	319	308
426	420	413	407	401	394	388	382	375	369	363	350	337	325
449	442	436	429	422	416	409	402	395	389	382	369	355	342
472	465	458	451	444	437	430	423	415	408	401	387	373	359
234	230	227	223	220	216	213	210	206	203	199	192	185	178
441	435	428	421	415	408	402	395	388	382	375	362	349	335
464	457	450	443	436	429	422	415	408	401	394	380	366	352
487	480	472	465	458	450	443	436	428	421	414	399	384	370
510	502	495	487	479	471	464	456	448	440	433	417	402	386
533	525	517	509	501	493	485	477	469	460	452	436	420	404
556	548	539	531	522	514	505	497	488	480	472	455	438	421
579	570	561	552	543	535	526	517	508	499	491	473	455	438
602	593	583	574	565	556	547	538	529	519	510	492	473	455
271	267	263	259	255	250	246	242	238	234	230	221	213	205
525	517	509	501	493	485	477	469	461	453	445	429	413	397
548	539	531	523	514	506	497	489	480	472	464	447	430	413
571	562	553	545	536	527	518	510	501	492	483	466	448	431
594	584	575	566	557	548	539	530	520	511	502	484	466	448
617	607	597	588	579	569	559	550	541	531	522	503	484	465
639	629	619	610	600	590	580	570	560	550	541	521	501	481
685	675	664	653	643	632	622	611	601	590	580	558	537	516
731	720	708	697	686	674	663	652	640	629	618	595	572	550
14′	15′	16′	17′	18′	19′	20′	21′	22′	23′	24′	26′	28′	30′

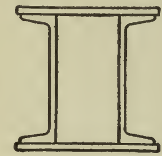


TABLE VIII (Continued)

Safe Loads on Channel Columns
12″ and 15″ Channels

Safe Loads are given in Thousands of Pounds

2 [s	2 Pls.	r	Area 2 Pls.	Area Total	UNSUPPORTED LENGTH 8′	9′	10′	11′	12′	13′
12″-40#	Latt.	4.09		23.52	338	333	328	323	318	314
"	14×13/16	4.12	22.75	46.27	665	655	646	636	627	618
"	7/8	4.11	24.50	48.02	690	680	670	660	651	641
"	15/16	4.11	26.25	49.77	715	705	695	684	674	664
"	1	4.11	28.00	51.52	740	730	719	708	698	687
"	1 1/8	4.10	31.50	55.02	790	779	768	757	745	734
"	1 1/4	4.10	35.00	58.52	840	828	816	804	792	780
"	1 3/8	4.10	38.50	62.02	891	878	865	853	840	827
"	1 1/2	4.09	42.00	65.52	941	927	914	900	887	873
15″-33#	Latt.	4.98		19.80	290	287	283	280	277	273
"	16×3/8	4.84	12.00	31.80	465	459	453	448	443	437
"	7/16	4.83	14.00	33.80	494	488	482	476	470	464
"	1/2	4.82	16.00	35.80	523	517	510	504	498	492
"	5/8	4.80	20.00	39.80	581	574	567	560	553	546
"	3/4	4.79	24.00	43.80	639	632	624	616	609	601
"	1	4.76	32.00	51.80	756	747	738	728	719	710
15″-35#	Latt.	4.94		20.58	301	298	294	291	287	284
"	16×1/2	4.81	16.00	36.58	534	528	521	515	508	502
"	5/8	4.79	20.00	40.58	592	585	578	571	564	557
"	3/4	4.77	24.00	44.58	650	642	635	627	619	611
"	1	4.75	32.00	52.58	767	758	748	739	730	720
"	1 1/2	4.72	48.00	68.58	1000	988	975	963	950	938
15″-40#	Latt.	4.84		23.52	344	340	335	331	327	323
"	16×1/2	4.75	16.00	39.52	576	569	562	555	548	541
"	5/8	4.74	20.00	43.52	635	627	619	611	604	596
"	3/4	4.73	24.00	47.52	693	684	676	668	659	651
"	1	4.71	32.00	55.52	809	799	789	780	770	760
"	1 1/2	4.69	48.00	71.52	1042	1029	1016	1003	991	978
"	2	4.68	64.00	87.52	1274	1259	1243	1227	1211	1196
15″-45#	16×5/8	4.69	20.00	46.48	677	669	660	652	644	635
"	3/4	4.68	24.00	50.48	735	726	717	708	699	690
"	1	4.68	32.00	58.48	851	841	830	820	809	799
"	1 1/2	4.66	48.00	74.48	1084	1071	1058	1044	1031	1017
"	2	4.66	64.00	90.48	1317	1301	1285	1268	1252	1235
15″-50#	16×1	4.64	32.00	61.42	894	883	872	861	849	838
"	1 1/2	4.64	48.00	77.42	1126	1112	1098	1084	1070	1056
"	2	4.63	64.00	93.42	1359	1342	1325	1308	1291	1274
"	2 1/2	4.63	80.00	109.42	1592	1572	1552	1532	1312	1492
					8′	9′	10′	11′	12′	13′

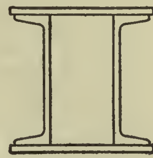
TABLE VIII (Continued)

Formula $P = 16{,}000 - 70\frac{l}{r}$

in which P = unit stress in pounds per square inch
r = radius of gyration in inches
l = length in inches

To left of heavy line values of $\frac{l}{r}$ do not exceed 125

To right of heavy line values of $\frac{l}{r}$ do not exceed 150



OF COLUMNS IN FEET

14′	15′	16′	17′	18′	19′	20′	21′	22′	23′	24′	26′	28′	30′
309	304	299	294	289	285	280	275	270	265	260	251	241	231
608	599	589	580	570	561	552	542	533	523	514	495	476	457
631	621	611	602	592	582	572	562	552	543	533	513	494	474
654	644	634	624	613	603	593	583	572	562	552	532	512	491
677	666	656	646	635	624	614	603	592	582	571	551	530	509
722	711	700	689	677	666	655	644	632	621	610	587	565	542
768	756	744	732	720	708	696	684	672	660	648	624	600	576
814	802	789	776	763	751	738	726	713	700	687	662	636	611
860	846	833	820	806	793	779	766	752	739	725	698	672	645
270	267	263	260	257	253	250	247	243	240	237	230	223	217
432	426	420	415	409	404	398	393	387	382	376	365	354	343
458	453	447	441	435	429	423	417	411	406	400	388	376	364
485	479	473	467	460	454	448	442	435	429	423	411	398	386
539	532	525	518	511	504	497	490	483	476	470	456	442	428
593	586	578	570	562	555	547	540	532	524	516	501	486	470
701	692	683	673	664	655	646	637	628	619	609	591	573	554
280	277	273	270	266	263	259	256	252	249	245	238	231	224
496	489	483	477	470	464	458	451	445	438	432	419	406	394
549	542	535	528	521	514	507	500	493	486	478	464	450	436
603	596	588	580	572	564	556	548	541	533	525	509	494	478
711	702	693	683	674	665	655	646	637	627	618	599	581	562
926	914	902	890	878	866	853	841	828	816	804	780	756	731
319	315	311	307	303	299	295	290	286	282	278	270	262	254
534	527	520	513	506	499	492	485	478	471	464	450	436	422
588	581	573	565	557	550	542	534	527	519	511	496	480	465
642	634	625	617	608	600	592	583	575	566	558	541	524	507
750	740	730	720	710	700	690	680	671	661	651	631	611	591
965	952	939	926	914	901	888	875	863	850	837	811	785	760
1181	1165	1149	1133	1118	1102	1086	1070	1055	1039	1023	992	960	929
627	619	610	602	594	586	577	569	561	552	544	527	510	494
681	672	663	654	645	636	626	617	608	599	590	572	554	536
789	778	768	757	747	736	726	715	705	694	684	663	642	620
1003	990	976	963	950	936	923	909	896	883	869	842	816	789
1219	1202	1186	1170	1154	1137	1121	1105	1088	1072	1056	1023	991	958
827	816	805	794	783	771	760	749	738	727	716	693	671	649
1042	1028	1014	1000	986	972	958	944	930	916	902	874	846	818
1257	1241	1224	1206	1189	1172	1156	1139	1122	1105	1087	1054	1020	987
1473	1453	1433	1413	1393	1373	1354	1334	1314	1294	1274	1234	1195	1156
14′	15′	16′	17′	18′	19′	20′	21′	22′	23′	24′	26′	28′	30′

Details of Construction. *Splices.* The several columns in a stack may be made in one-, two-, or three-story lengths. Two-story lengths are most commonly used. The one-story length permits each story of the column to be designed for the load in that story, whereas a two-story column is designed for the load in the lower of the two stories, the same section being used throughout the two-story length; this gives a greater area in the upper story than is required for the stress in that story. Similarly, for the three-story column the middle and upper stories are excessive; also the three-story column is more difficult to erect. The saving in favor of the one-story column is offset by the expense of the splices for material, shop labor, and erection; hence the common use of the two-story lengths.

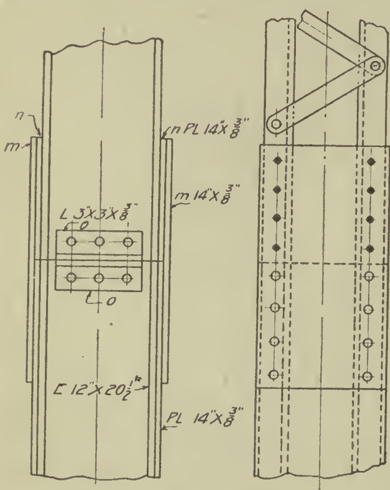


Fig. 145. Splice in a Channel Column

The splice is placed above the floor line a sufficient distance so that the splice plates will not interfere with the beam connections; usually 18 inches is enough space. The strength of the splice plates may vary from a nominal amount to the full strength of the column, generally the former, it being considered that the splice plates serve only to hold the columns rigidly in line. Even when there is bending stress due to eccentric loads, it seldom happens that there is actual tension on one side of the column, hence the splice plates do not transmit any stress.

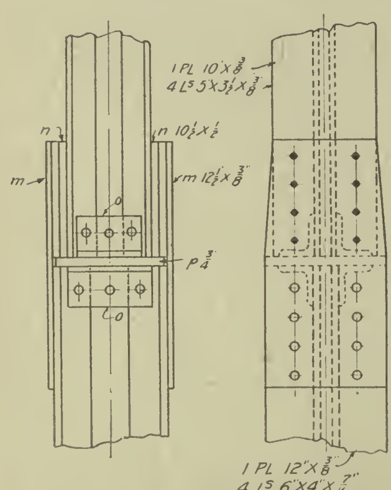


Fig. 146. Splice in a Plate and Angle Column

As the splice plates are not designed to carry stress, the load must be transmitted by direct bearing of the upper column on the lower. This requires that the ends be milled exactly at right angles to the axis of the columns and that the end of the upper column have full bearing on the top of the lower column, or, if this cannot be had on account of change in size or shape of columns, then that a bearing plate be used between the column sections.

Fig. 145 shows a column splice in which the upper section of column rests directly on the lower section. The splice is made by means of the plates *m* and the angles *o*. The plates *n* are fillers which make up the difference in width of the two sections of column. The angles *o* are used on the web of the channels because plates could not be riveted on after the columns are in place.

Fig. 146 shows a column splice in which the upper section of column does not rest directly on the lower section. In addition to

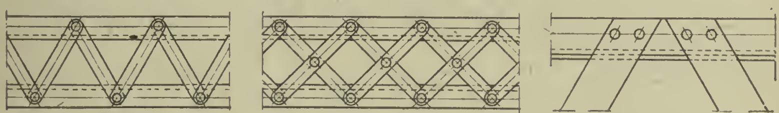


Fig. 147. Types of Lacing for Columns

the splice plates *m*, there are required the filler plates *n*, the bearing plate *p*, and the connection angles *o*.

No rules can be given for the thickness of splice plates, but they should be made consistent with the column section.

Riveting. The specifications for riveting, p. 365, govern the rivet spacing in columns. Refer to these specifications and note the spacing at the ends and the maximum spacing. It is interesting, as an indication of relative cost, to note the number of lines of rivets required for various styles of columns as follows:

Plate and angle columns without cover plates....2 rows
Plate and angle columns with cover plates. . 4 or 6 rows
Channel columns.............................4 rows
Bethlehem columns with cover plates...........4 rows
Zee-bar columns without cover plates...........2 rows
Zee-bar columns with cover plates..............6 rows

Lacing. Schneider's Specifications* contain provisions regard-

*"Specifications for Structural Steel for Buildings", by C. C. Schneider, M. Am. Soc. C. E., *Transactions American Society of Civil Engineers*, Vol. LIV, p. 449.

ing lattice bars, p. 365. Fig. 147 shows the style of lacing referred to. At the ends of laced columns tie plates are required. These plates should have a length not less than the width of the member. Tie plates must also be used where beams connect to the column and when the lacing must be omitted for any cause.

Connections. Connections for beams and girders are described and illustrated under the discussion of beams and girders. The methods there given will enable the designer to work out any special connections required.

Brackets. Brackets projecting to a considerable distance from the face of columns are used for supporting cornices, balconies, etc. If the bracket is constructed with a solid web plate and with parallel flanges, it may be designed as a cantilever girder. The bracket may

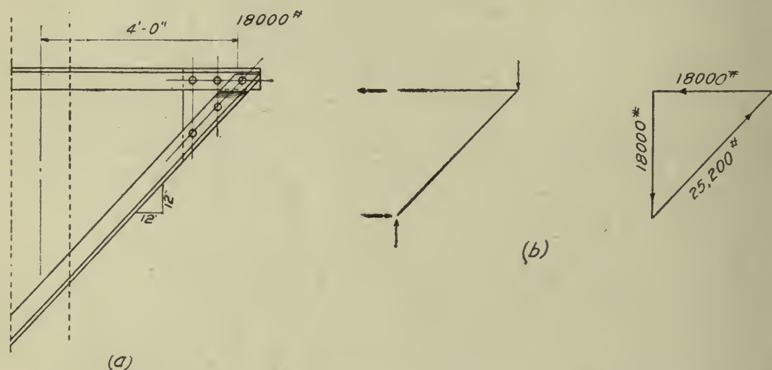


Fig. 148. Bracket on Column

be made up of a tie and a strut, Fig. 148, with no web plate. In this case the stresses are determined by the methods given in "Statics". To illustrate, use the data given in Fig. 148-a and the stress diagram, Fig. 148-b. For a load of 18,000 pounds on the end of the bracket, the stress in the tie is 18,000 pounds and the stress in the strut is 25,200 pounds. The tie and strut can now be designed by the methods given for tension and compression members. It is very important to keep in mind that loads on all projecting brackets are eccentric loads on the columns and the columns must be designed accordingly.

Bases. As the allowable pressure on the masonry foundation of the column is very much less than the stress in the column, it is necessary to provide a base plate to spread the load over the required area of the masonry. Whatever the form of base used, the bottom of the column section must be milled and the top of the base must be also a flat surface. If a steel plate is used, it will be true enough without milling, but all other forms require milling to give a true top surface. Two or more angles are riveted to the bottom of the column to provide a means of bolting the column to the base, Fig. 149. This bolting is done chiefly for assistance in erection.

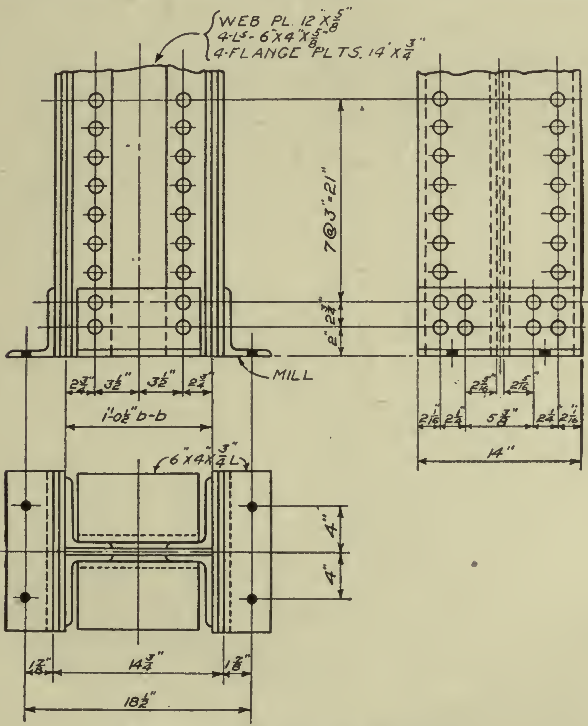


Fig. 149. Details of Bottom of Column

The bases are usually set exactly to elevation and alignment before the columns are set. This makes it easy to get the columns in their correct position. The bases are first supported on wedges and then the space under them grouted.

Flat Plates. The simplest form of column base is a flat plate, which may be either steel or cast iron. Having the load on the column and the allowable unit pressure on the masonry, the area of the plate is computed therefrom. The thickness of the plate is computed in the same manner as bearing plates for beams, p. 130.

The thickest *steel plate* ordinarily available is 1 inch and this limits the size of steel plate that can be used. However, steel slabs up to 12 inches thick can be had from the rolling mills if the quantity is large and considerable time can be allowed for delivery. They are designed in the same manner as plates of ordinary thickness except that the unit stress from bending should be 14,000 pounds per square inch. There is not likely to be any economy in using steel slabs when there is room for using cast-iron pedestals.

Cast-iron plates can be made of any thickness, but when the thickness would be greater than 4 inches, it becomes economical to use cast-iron pedestals. Fig. 150 is a cast-iron plate. The hole in the center is for grouting. In this form of plate the bolts must be in place before the plate is set; the bottom of the plate is recessed for the bolt heads.

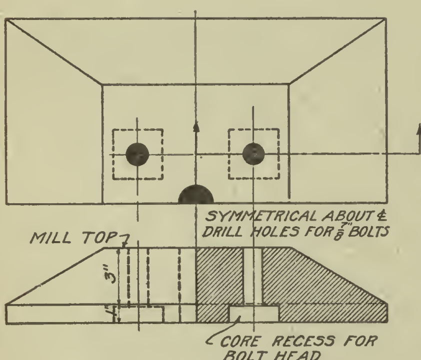


Fig. 150. Cast-Iron Base Plate

Cast-Iron Pedestals. When the size of the required base is so great that a flat plate is not practicable, the cast-iron pedestal is used, Fig. 151. It is impossible to compute the stresses in a cast-iron pedestal with any certainty. However, it is customary to design them by the flexure formula, which seems to give results that are satisfactory. This cannot be done directly, hence the dimensions must be assumed or determined from other considerations, and the resulting section checked by computing its resisting moment.

Illustrative Example. Assume that the load on a column is 600,000 pounds and the unit bearing on masonry is 500 pounds per square inch. Then the area required is

$$\frac{600{,}000}{500} = 1200 \text{ sq. in.}$$

Use a plate $3'\text{-}0'' \times 3'\text{-}0''$, which gives an area slightly in excess of the amount required.

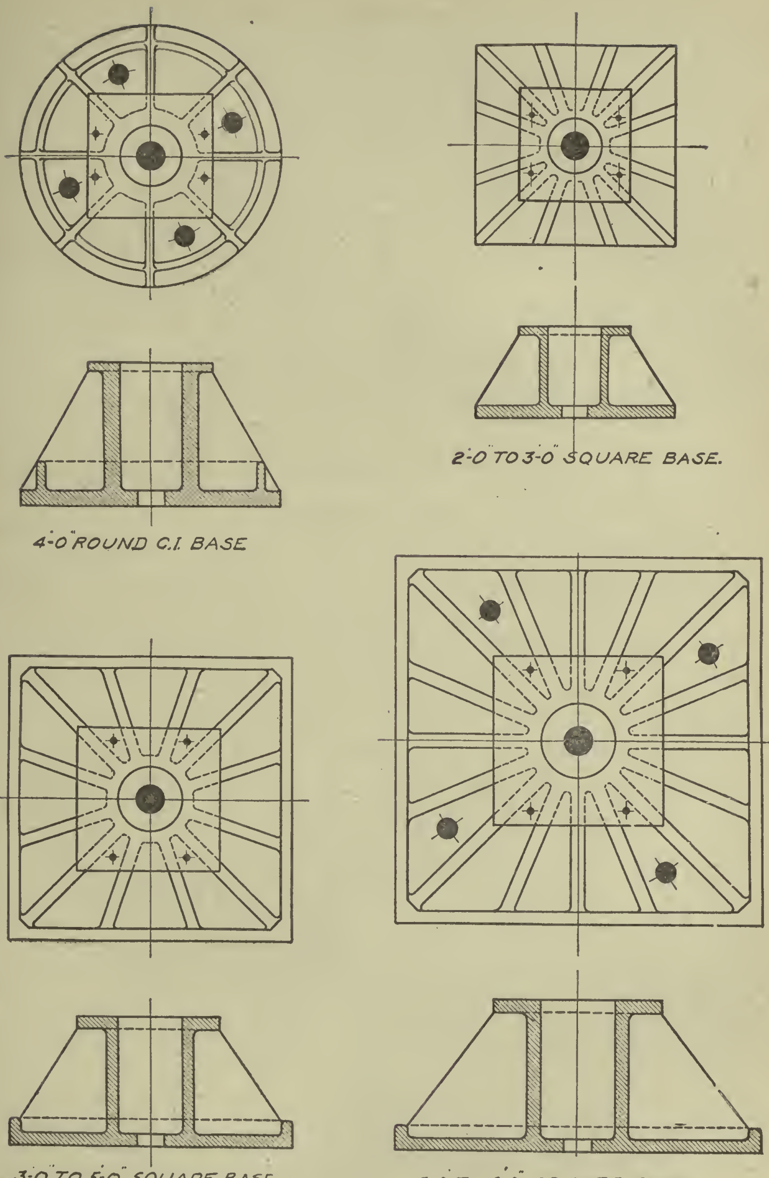


Fig. 151. Types of Cast-Iron Pedestals

The size of the top plate is determined by the size of the column and its connection angles. In this case assume 20 inches.

The height must be assumed. It may vary between one-third and one-half the base. Use 16 inches.

The diameter of the hub is assumed. Use 8 inches inside.

The thickness of metal must be assumed for trial. Use $1\frac{1}{4}$-inch for hub; $1\frac{1}{4}$ inches for top plate; and $1\frac{3}{4}$ inches for base plate.

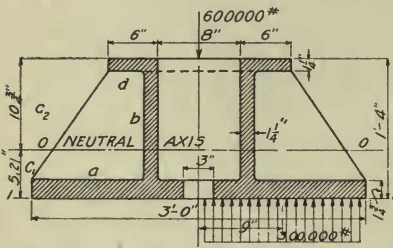


Fig. 152. Section for Computing Strength of Cast-Iron Pedestals

The shaded area in Fig. 152 shows the section available for resisting bending. To determine its resisting moment $R\,M$ it is first necessary to locate its neutral axis and then compute its moment of inertia.

The center of gravity is found by the method given on p. 36. The area of the cross section is

for a	area $= 33'' \times 1\frac{3}{4}''$	$= 57.75$
for b	area $= 2'' \times 13'' \times 1\frac{1}{4}''$	$= 32.50$
for d	area $= 2'' \times 6'' \times 1\frac{1}{4}''$	$= 15.00$
		105.25 sq. in.

Taking moments about the bottom line

for a	moment $= 57.75 \times .875$	$= 50.53$
for b	moment $= 32.50 \times 8.25$	$= 268.12$
for d	moment $= 15.00 \times 15.375$	$= 230.62$
		549.27

The distance from the bottom of the plate to the neutral axis *0-0* is

$$C_1 = \frac{549.27}{105.25} = 5.21''$$

The moment of inertia about the axis *0-0* is

for a	$I=$	from tables	15
		$57.75 \times (4.33)^2$	1083
for b	$I=$	from tables	458
		$32.50 \times (3.04)^2$	301
for d	$I=$	from tables	2
		$15 \times (10.16)^2$	1548
			3407

The allowable stress on cast iron in tension is 3000 pounds per square inch. Then the resisting moment of the section is

$$R\,M = S\,\frac{I}{c} = \frac{3000 \times 3407}{5.21} = 1{,}940{,}000 \text{ in.-lb.}$$

The bending moment M, resulting from the pressure on the bottom of the plate, is determined by treating the plate as an inverted cantilever, Fig. 152.

Then $$M = 300{,}000 \times 9 = 2{,}700{,}000 \text{ in.-lb.}$$

This amount is excessive because it assumes the column load applied at a point at the center of the top of the plate, whereas it occupies considerable area.

As the bending moment computed for the load is 2,700,000 inch-pounds and the resisting moment is 1,940,000 inch-pounds, the trial section is not sufficient. The section can be increased in strength by increasing the height or by increasing the thickness of metal. The most effective places for additional metal are in the top and bottom plates.

Problem

Increase the height of the cast-iron pedestal to 1′-6″, retaining all other dimensions. Compute the resisting moment.

Number of Ribs. The number of ribs to be used can be as indicated by the bases shown in Fig. 151. Therefore, 12 ribs are used for the case illustrated above. The thickness of the rib should be not less than 1 inch and about $\frac{1}{12}$ the clear height be tween the bottom and top plates. Also there must be enough section in the ribs and hub just below the top plate to take the whole load at 10,000 pounds per square inch. In this case the clear height is 13 inches, which indicates 1-inch ribs. This thickness gives ample section for compression.

Shape of Pedestal. Cast-iron pedestals may be made round instead of square if so desired, Fig. 151. The round pedestal has some advantages in manufacture and is especially well suited for round piers. The bending moment on a round base is approximately the total column load multiplied by 0.10 of the diameter of the plate. The resisting moment of the pedestal is computed by the method given above.

When a rim is used around the edge of the bottom plate, it can be computed in the section resisting bending. This rim is desirable in large pedestals.

There must always be a grout hole at or near the center of the pedestal. In large plates additional holes are used.

Steel Grillage. If the masonry bearing is long and narrow, steel I-beams may be used for spreading the load. When so used, they are designed in the same manner as given for bearings for beams, p. 133. These beams rest directly on the masonry, and are filled

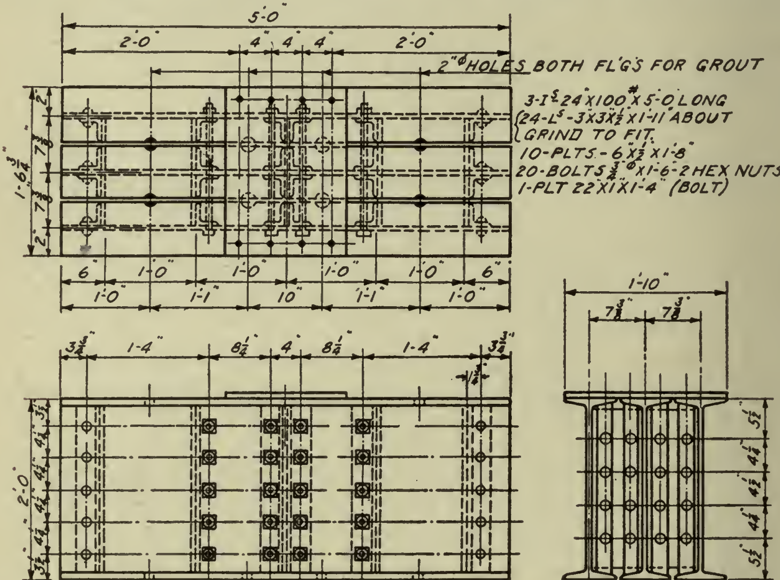


Fig. 153. Steel Grillage Designed for Base of a Column of a 16-Story Building

with cement, concrete, or grout. The webs of the beams must be investigated for shearing strength and, if at all deficient, tight-fitting separators must be used. Separators should be used in any case to hold the beams in position.

The flanges of I-beams are not always exactly at right angles to the webs, hence the beams may not furnish a flat surface for seating the columns. This makes it necessary to plane enough off the top of beams to provide a true surface for the bottom of the column to rest upon. In order to be effective, this must be done after the beams are assembled and rigidly held together.

The load from the column must be properly distributed to the beams forming the grillage, using a steel or cast-iron plate of proper thickness. It may be necessary in some cases to use a cast-iron pedestal on the steel grillage. Fig. 153 shows a steel grillage designed for the base of a column of a 16-story building.

CAST-IRON COLUMNS

Cast-iron columns formerly were used extensively for building work, even for fireproof buildings ten or more stories in height. Now they are used only for small buildings of non-fireproof construction. The change has come about through greater demand for safety and the reduction in cost of steel columns.

Characteristics. *Advantages.* The advantages of cast-iron columns are: They offer greater resistance to fire than unprotected steel columns. They generally can be more quickly obtained. They can be made of any desired shape and ornamented to suit the requirements of architectural design. They occupy a minimum of space in the building.

Disadvantages. Cast-iron columns have the following disadvantages: For supporting a given load a cast-iron column costs more than a steel column. They are subject to defects that are difficult to discover by usual methods of inspection.

Cast-iron columns are made to order. As the brackets and flanges must be cast on the column shaft at the time it is made, it is not possible to have the column shafts in stock.

Cast iron is subject to considerable variation in quality, depending upon the materials used in the melt and the treatment in the furnace and in the molds. It may be soft and tough, or hard and brittle. It is made in small foundries, as compared with the rolling mills which make structural steel. Hence it is not possible to control the quality closely, as can be done with steel.

Blow holes in castings are spaces which the iron does not fill, due to bubbles of air or gas becoming entrapped in the mold. Sand pockets may be formed by the dropping of sand from the molds. In both of these cases the surface of the casting may be perfect, and the defects thus difficult or impossible to find.

The most frequent fault with round columns is eccentricity, due to displacement of the core. The core may sag in the mold, due to its weight, or it may float in the liquid iron. The result is shown in Fig. 154. It may occur at any place in the length of the column.

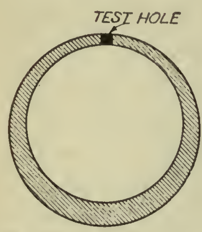


Fig. 154. An Eccentric Cast-Iron Column

At the ends the fault is easily detected, but at intermediate points it is necessary to drill test holes as indicated in the figure. The test holes should be drilled in the top or in the bottom of the casting in reference to its position in the mold. An eccentricity of $\frac{1}{8}$ inch causes appreciable loss of strength. A greater amount than this should cause rejection.

Column Sections. Unless there is some reason for using a special shape, cast-iron columns are made round. The size is designated by the external diameter and the thickness of metal. The sizes commonly made for structural purposes vary from 6 inches to 15 inches in diameter and from $\frac{3}{4}$ inch to $2\frac{1}{2}$ inches in thickness.

Special sections sometimes used are shown in Fig. 155. The angles, U-shapes, and square sections are used chiefly for store front work. They are generally made with the exposed surfaces paneled or otherwise ornamented.

H-shaped columns may be used for general purposes. They are not as economical as round columns, hence are not much used. In some respects they are better than round columns as connections

Fig. 155. Typical Cast-Iron Column Sections

are easily made and all surfaces are open to inspection, making it easier to find defects.

Method of Design. The method of designing cast-iron columns is similar to that used in designing steel columns. The direct load and the concentric equivalent of eccentric loads, if any, are computed in the same manner. The allowable unit stress is computed from a formula similar to that for steel columns. The formula given under Unit Stresses, p. 51, is

$$P = 10{,}000 - 60\frac{l}{r}$$

Eccentric Loading. The concentric equivalent for eccentric loads is computed by the same formula as used for steel columns, viz:

$$W'_e = W' \frac{ec}{r^2}$$

For round cast-iron columns an approximate formula is

$$W'_e = \frac{5M}{d}$$

in which M is the eccentric moment in inch-pounds and d is the diameter of the column in inches.

Fig. 156 shows two cases of eccentric loading of a round column. For the load m, the eccentricity e_m is the distance from the center of the column to the center of the web of the beam. For the load n,

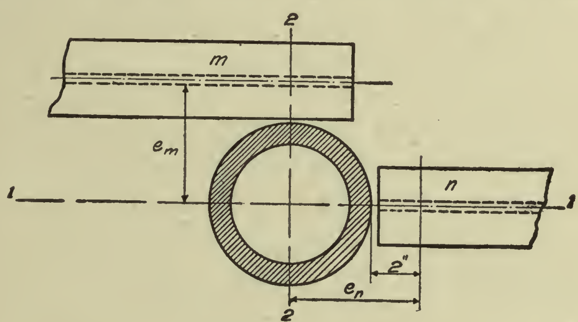


Fig. 156. Diagrams Showing Eccentric Loading on a Round Column

the eccentricity e_n is the distance from the center of the column to the center of bearing on the bracket, this center of bearing being taken at 2 inches from the face of the column, when standard brackets are used.

On page 177, it was pointed out that when two eccentric loads act about the axis *1-1* and *2-2*, respectively, their results must be added together. This is true also of rectangular and round cast-iron columns. But for round columns the maximum effect of two such loads is somewhat less than the sum of their separate effects. The resultant varies with the relative amounts of the eccentric moments, but the difference is not great and the sum of the separate effects can be used without much error.

Factors Required. If the concentric equivalent load is used, the only properties of the section required are: area A; radius of gyration r; and distance to extreme fiber c. The values of these properties can be computed for the rectangular sections by the methods given. For round columns the area is computed from the formula

$$A=\frac{\pi}{4}(d^2-d_1^2)$$

the radius of gyration is computed from the formula

$$r=\tfrac{1}{4}\sqrt{d^2+d_1^2}$$

the distance c is $\frac{1}{2}$ d. In these formulas π is 3.1416; d is outside diameter of column; and d_1 is inside diameter of column. The inside diameter equals the outside diameter less twice the thickness of metal. Thus a column 8 inches in diameter and $1\frac{1}{2}$ inches thickness of metal has an inside diameter of 5 inches.

Illustrative Example. Assume a column with the following dimensions and loads, and determine the thickness of metal required:

Length of column 140″
Concentric load from column above 160,000#
Eccentric load 40,000# – eccentricity 7″
Outside diameter of column (assumed) 10″

Then the eccentric moment is $40{,}000\times 7=280{,}000$ in.-lb., which by the rule on p. 175, is reduced to $\frac{3}{4}\times 280{,}000=210{,}000$ in.-lb. The concentric equivalent is

$$W'_e=\frac{5M}{d}=\frac{5\times 210{,}000}{10}=105{,}000\#$$

The total load for which the column must be designed is

Load from upper column	160,000#
Eccentric load	40,000
Concentric equivalent load	105;000
	305,000#

It is now necessary to assume a trial thickness of metal and compute the strength. Assume 2 inches.

$$A=\frac{\pi}{4}(d^2-d_1^2)=\frac{3.1416}{4}\times(100-36)=50.26 \text{ sq. in.}$$

$$r=\tfrac{1}{4}\sqrt{d^2+d_1^2}=\tfrac{1}{4}\sqrt{100+36}=2.9$$

$$P=10{,}000-60\,\frac{l}{r}=10{,}000-60\times\frac{140}{2.9}=7100\# \text{ per sq. in.}$$

$\therefore$ Total capacity $7100\times 50.26=356{,}800\#$

TABLE IX

SAFE LOADS FOR ROUND CAST-IRON COLUMNS

Thousand Pound Units

$$P = 10{,}000 - 60\,\frac{l}{r}$$

Values to right of heavy line are beyond limit of length, 70 r.

Outside Diam. in.	Thickness in.	LENGTH FEET 6	8	10	12	14	16	18	20	22	Weight lbs. per ft. of length	Area sq. in.	Moment of Inertia I	Radius of Gyration r
6	$\frac{5}{8}$	81.7	73.7	65.7	57.8						33.2	10.6	38.6	1.91
	$\frac{3}{4}$	95.1	85.6	76.1	66.6						38.6	12.4	43.5	1.87
	$\frac{7}{8}$	107.8	96.8	85.8	74.7						44.0	14.1	47.6	1.84
	1	119.4	106.8	94.3	81.7						49.0	15.7	51.0	1.80
	$1\frac{1}{8}$	130.2	116.2	102.2	88.2						53.8	17.2	53.9	1.77
	$1\frac{1}{4}$	140.2	124.8	109.4	93.9						58.2	18.6	56.2	1.74
7	$\frac{3}{4}$	118.7	109.2	99.7	90.2	80.7					46.0	14.7	72.9	2.23
	$\frac{7}{8}$	135.2	124.1	113.0	102.0	90.9					52.6	16.8	80.6	2.19
	1	150.6	138.0	125.4	112.7	100.1					58.9	18.8	87.2	2.15
	$1\frac{1}{8}$	165.1	151.0	136.8	122.6	108.4					64.8	20.8	92.9	2.11
	$1\frac{1}{4}$	178.9	163.3	147.6	132.0	116.4					70.7	22.6	97.7	2.08
	$1\frac{3}{8}$	191.8	174.7	157.6	140.6	123.5					76.1	24.3	101.8	2.05
	$1\frac{1}{2}$	203.8	185.3	166.8	148.3	129.8					81.1	25.9	105.3	2.02
8	$\frac{3}{4}$	142.2	132.7	123.2	113.6	104.0	94.6				53.3	17.1	113.4	2.58
	$\frac{7}{8}$	162.6	151.4	140.3	129.2	118.1	107.0				61.3	19.6	126.2	2.54
	1	181.9	169.2	156.6	143.9	131.2	118.6				68.6	22.0	137.4	2.50
	$1\frac{1}{8}$	200.3	186.1	171.9	157.7	143.4	129.2				76.1	24.3	147.4	2.46
	$1\frac{1}{4}$	218.0	202.3	186.6	170.9	155.2	139.5				82.7	26.5	156.1	2.43
	$1\frac{3}{8}$	234.4	217.2	199.9	182.7	165.3	148.2				89.3	28.6	163.8	2.39
	$1\frac{1}{2}$	250.3	231.6	212.9	194.2	175.5	156.8				94.8	30.6	170.4	2.36
9	$\frac{3}{4}$		156.3	146.6	137.1	127.5	118.0	108.4			60.6	19.4	166.8	2.93
	$\frac{7}{8}$		178.8	167.7	156.6	145.4	134.3	123.2			69.8	22.3	186.4	2.89
	1		200.5	187.8	175.1	162.4	149.7	137.0			78.4	25.1	204.2	2.85
	$1\frac{1}{8}$		221.3	207.0	192.8	178.5	164.2	150.0			87.0	27.8	220.2	2.81
	$1\frac{1}{4}$		241.3	225.5	209.8	194.0	178.2	162.5			94.9	30.4	234.5	2.78
	$1\frac{3}{8}$		260.1	242.8	225.6	208.3	191.0	173.7			103.0	32.9	247.3	2.74
	$1\frac{1}{2}$		278.0	259.2	240.3	221.5	202.6	183.8			110.3	35.3	258.4	2.70
10	$\frac{3}{4}$		179.7	170.1	160.5	151.0	141.4	131.8	122.3		68.2	21.8	234.6	3.28
	1		231.8	219.1	206.4	193.7	181.0	168.2	155.5		88.2	28.3	289.9	3.20
	$1\frac{1}{4}$		280.4	264.6	248.8	233.0	216.3	201.3	185.5		107.2	34.4	335.6	3.13
	$1\frac{1}{2}$		324.9	306.0	287.1	268.2	249.3	230.4	211.5		125.0	40.0	373.1	3.05
	$1\frac{3}{4}$		365.9	344.0	322.1	300.2	278.3	256.4	234.4		141.7	45.4	403.2	2.98
11	1		263.2	250.4	237.7	225.0	212.2	199.5	186.7	174.0	98.0	31.4	396.7	3.55
	$1\frac{1}{4}$		319.4	303.6	287.7	271.8	255.9	239.9	224.0	208.1	119.5	38.3	462.6	3.48
	$1\frac{1}{2}$		371.8	352.9	333.9	315.0	296.0	277.0	258.1	239.1	139.7	44.8	517.8	3.40
	$1\frac{3}{4}$		420.6	398.6	376.6	354.6	332.6	310.6	288.7	266.7	158.7	50.9	563.5	3.33
	2		465.6	440.6	415.6	390.6	365.6	340.7	315.7	290.7	176.4	56.5	601.0	3.26
12	1		294.7	281.9	269.2	256.5	243.8	231.0	218.3	204.7	107.5	34.6	527.1	3.91
	$1\frac{1}{4}$		358.7	342.8	326.9	311.0	295.2	279.3	263.4	247.6	131.4	42.2	618.2	3.83
	$1\frac{1}{2}$		418.8	399.8	380.8	361.8	342.8	323.8	304.8	285.8	154.1	49.5	696.0	3.75
	$1\frac{3}{4}$		475.3	453.3	431.2	409.2	387.1	365.1	343.0	320.9	175.5	56.4	761.8	3.68
	2		528.1	503.0	478.0	452.9	427.8	402.8	377.7	352.6	195.8	62.8	817.0	3.61
13	1		326.0	313.3	300.5	287.8	275.0	262.3	249.6	236.8	117.5	37.7	683.5	4.26
	$1\frac{1}{4}$		397.8	381.9	366.0	350.2	334.2	318.4	302.5	286.6	143.9	46.1	805.3	4.18
	$1\frac{1}{2}$		465.8	446.8	427.7	408.7	389.7	370.6	351.6	332.6	169.0	54.2	911.3	4.10
	$1\frac{3}{4}$		529.8	507.7	485.5	463.4	441.2	419.1	397.0	374.8	192.9	61.9	1002.4	4.02
	2		590.4	565.2	540.0	514.8	489.6	464.4	439.2	414.0	215.6	69.1	1080.2	3.95

Make allowance for eccentricity in accordance with the following formula: $W_e' = 5\,\frac{M}{d}$

W_e'=Equivalent concentric load, lb.; M=Moment of eccentricity, in.-lb.; and d=Diameter, in. See pp. 227, 228.

This amount is greater than required, so the thickness may be reduced. It can be shown that the thickness required is $1\frac{5}{8}$ inches.

Problem

From the data given above, determine the thickness required for a column 12 inches in diameter. Note that eccentricity is 8 inches for this diameter.

Tables. The published tables of strength of cast-iron columns vary greatly, due to the variety of formulas used. Columns other than round are used so little and when used are so likely to be of special dimensions that tables of strength would be of little value. Table IX gives the strength of round columns in accordance with the formula adopted. It also gives the value of r for use in computing the concentric equivalent for eccentric loads. The Chicago Building Ordinance from which this formula is taken limits the

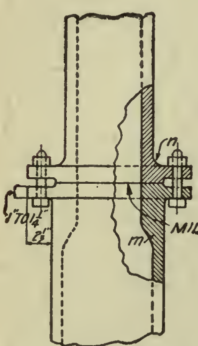


Fig. 157. Splice of Cast-Iron Column by Flanges

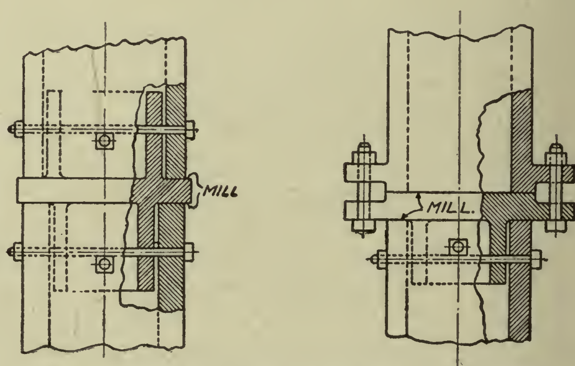


Fig. 158. Splices of Cast-Iron Columns by Dowel Plates

length of cast-iron columns to $70 \times r$. This limit is marked by the heavy zigzag line in the table.

Illustrative Example. Determine the column required to support a load of 191,000 pounds, the length being 11 feet.

From Table IX either of the following sizes may be used:

$8''$ diam. $\times 1\frac{3}{8}''$ metal
$9''$ diam. $\times 1\frac{1}{8}''$ metal
$10''$ diam. $\times 1\ ''$ metal

The 9-inch column is the lightest and will be used if no special consideration indicates the use of one of the other sizes.

Problem

The loads and lengths of a stack of cast-iron columns are given below. Determine the sections.

4th story,	column load,		20,000#	length 13 ft.
3rd "	"	"	70,000#	length 12 ft.
2nd "	"	"	115,000#	length 14 ft.
1st "	"	"	155,000#	length 16 ft.
Basement	"	"	205,000#	length 9 ft.

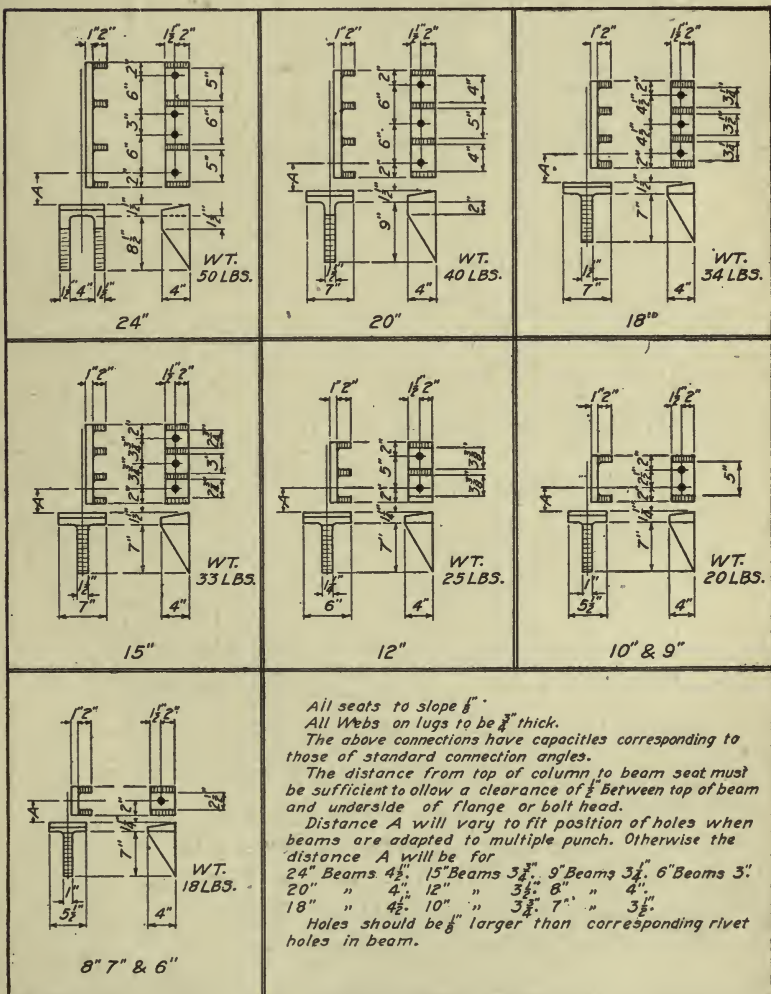


Fig. 159. American Bridge Company Standard Beam Connections

Details of Construction. *Splices.* Splices in cast-iron columns are made by means of flanges as shown in Fig. 157. The load is transmitted from upper to lower column by bearing. The bearing surfaces must be milled exactly at right angles to the axis of the column. If the sections do not match, the metal must be thickened as shown at *m* and *n* to provide the bearing. Some manufacturers set the flanges back from the ends of the column to reduce the area

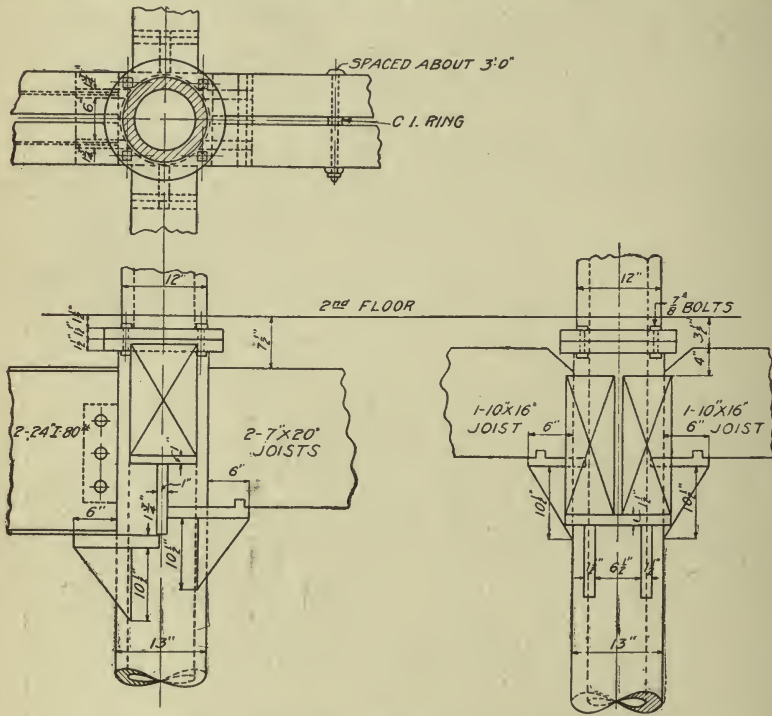


Fig. 160. Double Beam Connections to Cast-Iron Columns

of the milled surface. The flange is made wide enough to take a row of $\frac{3}{4}$-inch bolts. Four or more bolts are used.

The splice can be made by means of a dowel plate. It is not so satisfactory as the flange splice. It is used when there is no space available for the flanges, and also for replacing broken flanges, Fig. 158.

Beam Connections. Beam connections are made by means of brackets and lugs cast on the column. The standard connections designed and used by the American Bridge Company are given in Fig. 159. The entire load is supported by the bracket. The seat of the bracket slopes so that the beam will not bear on the end of the bracket when it deflects. The lug serves to tie the construction together and to hold the beam upright. Bolts must be used for all connections to cast iron, as the casting would be broken by driving rivets.

When double beams are used, the connection is modified as shown in Fig. 160. This figure also shows brackets for supporting wood beams. Fig. 161 shows the detail of the top of a cast-iron column which supports two steel beams.

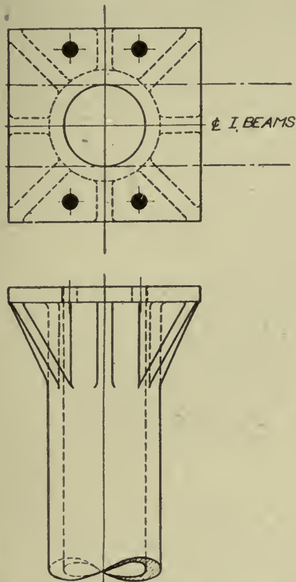


Fig. 161. Top of Cast-Iron Column for Supporting I-Beams

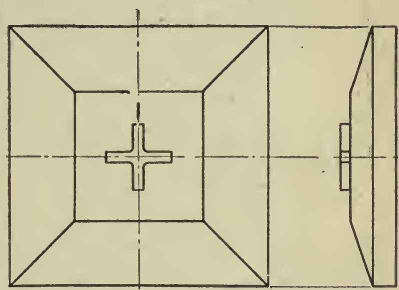


Fig. 162. Base Plate for Cast-Iron Columns

Bases. Cast-iron base plates or cast-iron pedestals are used for cast-iron columns. They are designed in the manner described for the bases of steel columns. If the plate is used, a raised cross is cast on the top to fit inside the column and hold it in place, Fig. 162. If the pedestal is used, the top of it is made to match the flange cast on the column.

TENSION MEMBERS

Definition and Theory. In building construction, it does not often occur that loads must be supported by tension members. Occasional special features, such as balconies or stair landings, require this form of support. The most frequent use of it occurs in trusses (which are not covered in this work).

Axial Tension. A member is subjected to axial tension when the load is applied in line with the axis of the member in a way that tends to stretch or pull the member apart, Fig. 163.

The strength of steel in axial tension varies directly in proportion to the net cross-section area, not being affected by the length (except as to the weight of the member) or by the shape of the section. Under Unit Stresses, the allowable value of P for axial tension is given as 16,000 pounds per square inch; then the strength of a section is

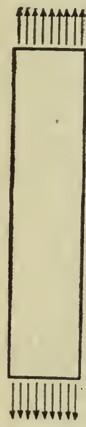


Fig. 163. Diagram Showing a Tension Member

$$W = P\,A = 16{,}000\,A$$

The area used in this formula must be the net area, i. e., the smallest area at any section in the length of the member.

In axial tension the stress is assumed to be distributed over the entire area, as indicated in Fig. 163. This differs from the tension due to bending, which is not uniformly distributed but increases from nothing at the neutral axis to a maximum at the extreme fiber, as explained on p. 78.

Tension Due to Eccentricity. As in the case of compression members, the load on a tension member may be eccentric, and thus produce both axial tension and tension due to bending. The discussion of concentric and eccentric loads in compression applies to tension members. Fig. 164 illustrates the stresses from an eccentric load in tension which corresponds to Fig. 140 in compression; $a\,b\,c\,d$ represents the total axial tension and $a\,b$ the axial tension per square inch due to the load W'; $b\,b'$ represents the tension on the extreme fiber due to the bending moment $W'e$. Then the total extreme fiber stress due to the load W' is $a\,b'$. The concentric equivalent of an eccentric load, as for compression, is expressed by the formula

$$W_e' = W'\frac{e\,c}{r^2}$$

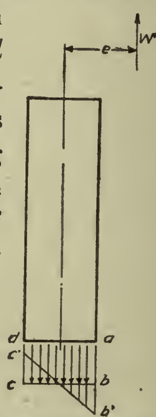


Fig. 164. Diagram of Stresses from Eccentric Loads in Tension

If the member is not symmetrical, the value of c to be used is from the neutral axis to the extreme fiber on the side toward the eccentric load.

Eccentricity in tension members usually results from the form of the connection, and in most cases it can be avoided by careful attention to the details. It generally will be more economical thus to avoid the eccentricity than to provide the additional section necessary to resist it. In altogether too many cases this is neglected. The importance of the effect of eccentricity is illustrated by the following computations.

Assume a load of 100,000 pounds concentric, then the net area required is $\frac{100,000}{16,000}$ or 6.25 square inch. Now assume the same load with an eccentricity of 1 inch, a value of c equal to $2\frac{1}{2}$ inches and r equal to 1.9 inches. The concentric equivalent is

$$W'_e = \frac{100,000 \times 1 \times 2\frac{1}{2}}{1.9 \times 1.9} = 70,000\#$$

The total load is 100,000+70,000 or 170,000 pounds, and the area required is $\frac{170,000}{16,000}$ or 10.6 square inches. In this case it requires

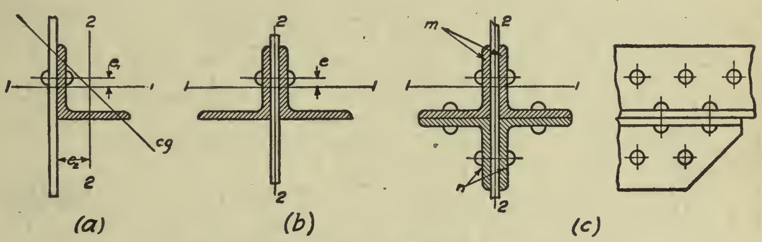


Fig. 165. Types of Connections for Angles

an increase of 70 per cent in the section to provide for the eccentricity

Fig. 165-a shows a single angle connected by one leg. It is eccentric about both axes. Fig. 165-b shows a pair of angles each connected by one leg. This is eccentric about the axis *1-1*. Fig. 165-c shows two views of the same pair of angles *m*, with a pair of connection angles *n* added, which eliminates the eccentricity.

Sections. Almost any form of steel can be used as a tension member. The choice of the section is governed largely by the connections that are to be made to it. Of the structural shapes, angles, plates, and channels are best adapted for tension members in ordinary building work.

Round rods are used for tie-rods, balcony hangers, temporary bracing, and other similar purposes.

Eyebars are seldom used in building work, being more especially adapted to bridge trusses. They may be used where heavy loads occur and rigidity is not important.

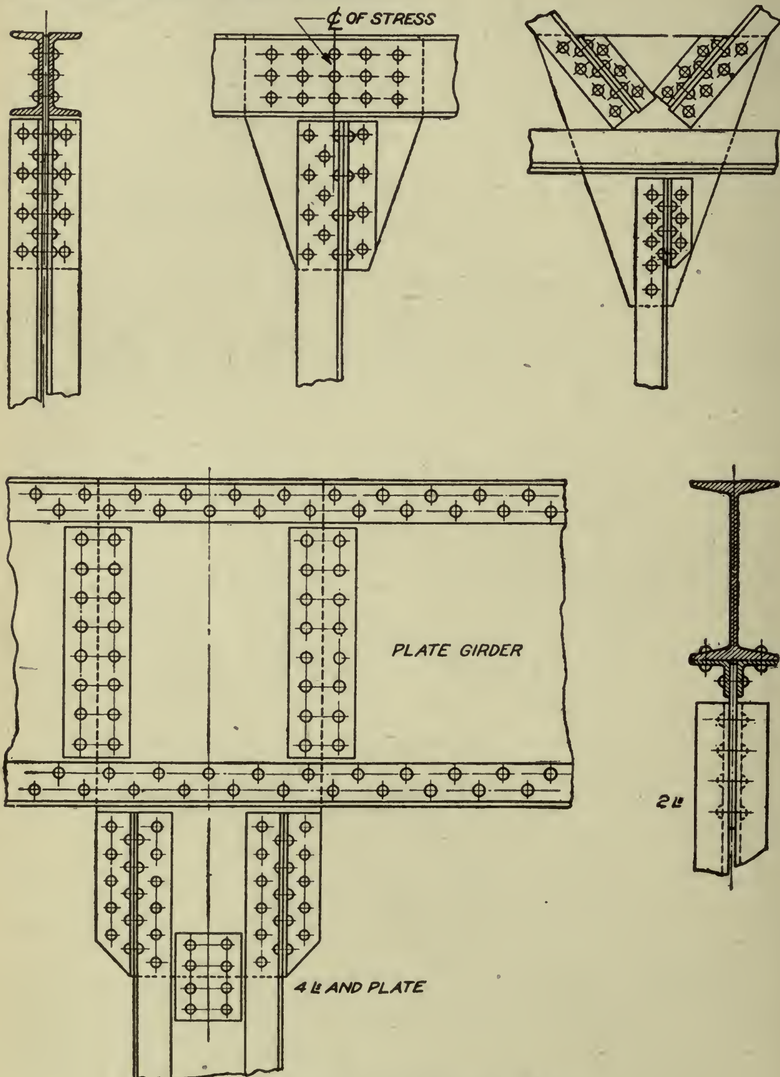


Fig. 166. Types of Connections for Hangers

Net Area. Plates and shapes in tension must be connected by rivets and the rivet holes must be deducted to determine the net area of cross section. The number of rivet holes to be deducted in any case depends upon their arrangement as explained on p. 69. The size of the hole deducted is $\frac{1}{8}$ inch greater than the nominal diameter of the rivet. This allowance is an arbitrary one to cover the actual size of the hole, which is about $\frac{1}{16}$ inch larger than the rivet, and to compensate for injury to the metal around the hole due to punching. Care must be taken to arrange the rivet holes so as to retain the greatest possible area at the critical section.

Round rods can be figured full size if the ends are upset, otherwise the net area must be taken at the root of the thread. When upset ends are used, they are made large enough so that there is an excess of strength in the threads, making the whole section of the rod available. Generally the threads on rods are cut, but they can be made by cold rolling. The latter method makes the diameter at the root of the thread somewhat less than the diameter of the body of the rod, but the treatment seems to make the steel stronger. Tests show that the rolled thread is stronger than the rod on which it is rolled, thus making the whole section of the rod available.

Eyebar heads are always made of sufficient size to develop the strength of the bar, so that the whole section is available.

Details of Connections. *Riveted Connections.* Riveted connections are required when structural shapes or plates are used. Angles, plates, and channels are most commonly used. The top connection usually is made with a gusset plate depending from a beam or girder. Fig. 166 illustrates a number of such connections. The gusset plate may be spliced into the web of a plate girder; set in between two channels; may be an extension of the gusset at the joint of a truss; or may be connected by angles riveted to the flange of an I-beam. (See p. 64). The requirements for the top connection are that the gusset plate shall be of sufficient thickness to give the required bearing for the rivets; and that the rivets connecting the plate to the beam or girder, also those connecting the hanger to the gusset, be sufficient in number and be placed symmetrically about the axis of the tensile stress.

It has been noted that angles in tension must be connected by both legs to avoid eccentricity. This sort of connection is desirable for the further purpose of distributing the stress over the entire section of the hanger as evenly as possible. Angles in pairs are much preferred to single angles. They should be stitched together with rivets and ring fillers spaced about 2 feet apart.

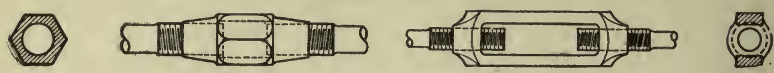


Fig. 167. Turnbuckle and Sleeve Nut

The connections at the bottom of the hanger may be made with gusset plates in the same manner as at the top, or the connecting members may be attached direct to the hanger.

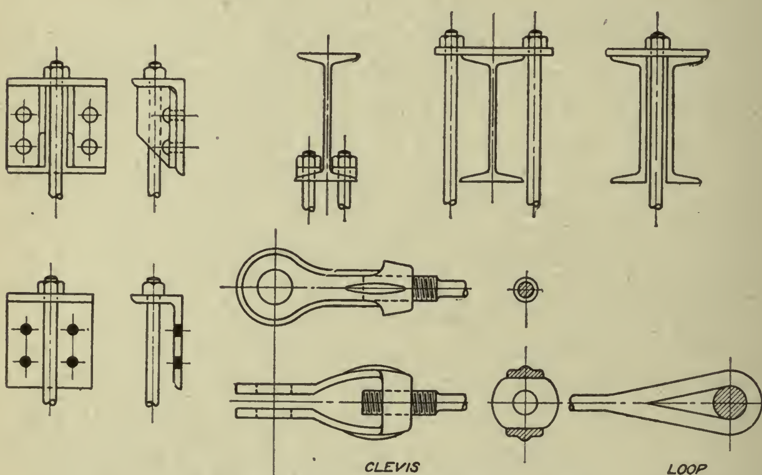


Fig. 168. Types of End Connections for Rods

When it is necessary to splice a tension member, it is evident that the splice must transmit the entire stress in the member. The principles involved and methods to be used are fully explained under Strength of Riveted Joints, p. 67, and have been used in designing the splices in plate girders.

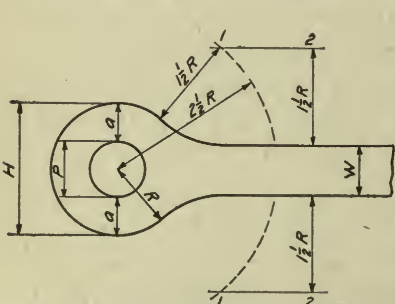


Fig. 169. Details of End Connection of Eyebar

Details of Rods. Rods are specially suited for adjustable members. With certain forms of connections, the adjustment can be made at the ends; with splices, the adjustment can be made at the splice. A rod is spliced by means of a turnbuckle, or sleeve nut, Fig. 167. The ends are threaded right and left to make the member adjustable. The threaded ends are upset to maintain the full strength of the section. The various forms of end connections are shown in Fig. 168. They need no explanation.

Details of Eyebars. Eyebars must be connected at the ends with pins, Fig. 169. Refer to "Structural Drafting" for details of eyebars.

WIND BRACING

GENERAL CONDITIONS

Horizontal Pressures. In the preceding discussion, the loads considered have been gravity loads, i. e., loads acting vertically. In addition to these gravity loads, all structures are subjected to wind loads, or pressures, which are assumed to act horizontally. Probably no locality is entirely free from wind storms, so it is always necessary to provide for wind pressures in designing the framework of buildings.

It is assumed that wind pressure acts horizontally and bears uniformly over the entire windward surface of the building, and that it may occur in any direction. These assumptions are not strictly correct. The wind may be inclined, due to the contour of the ground or to obstructions. It is known that the pressure near the top of a building is greater than near the ground; that the pressure is not uniform over large areas; that the rush of air around the corners produces greater pressure near the corners; and that there is a suction on the leeward side as well as a pressure on the windward side. The wind may strike the building at any angle, but the maximum effect is produced when it strikes squarely against the side (or end) of the building. While the above variations are known to be true, it is impossible to provide for them in detail, hence the assumption stated above is followed and leads to satisfactory results.

Unit Pressure. Many experiments have been made to establish the relation between wind velocity and wind pressure. While a large amount of data has been developed, the mathematical relations are not fully established. Furthermore, it is not certain what maximum velocity should be provided for. Hence it is the general practice to use an assumed pressure in pounds per square foot of the surface. The amount assumed varies. In some cities the building ordinances specify the amount to be used; some specify 20 pounds per square foot; others, 30 pounds. The writer recommends that the framework of all buildings be designed to resist a wind pressure of 20 pounds per square foot on the surface of the building. It can reasonably be assumed that the partitions and walls will add enough to the strength so that the completed structure will resist a pressure of 30 pounds per square foot. Walls should not be counted as resisting any part of the 20 pounds, unless practically solid, i. e., without openings. The above recommendations should be followed with some discretion: increasing the amount carried by the framework in very high buildings, and in buildings which have few partitions or a very large percentage of openings in the walls; decreasing the amount in low buildings, and in buildings which have masonry cross walls. In buildings having outside bearing walls of masonry and a reasonable amount of cross walls, or partitions, these parts may be relied upon to resist the entire wind pressure, provided the height of the building is not more than twice its width.

The maximum wind pressure occurs only at long intervals. It is, therefore, allowable to use higher unit stresses for wind stresses than for gravity stresses. Under Unit Stresses it is provided that for stresses produced by wind forces alone, or combined with those from live and dead loads, the units may be increased fifty per cent over those given for live load and dead load stresses; but the section shall not be less than required, if wind forces be neglected. Generally, the members required to support the gravity loads are utilized for the wind loads. In such cases no additional area is required on account of the wind stress unless this stress exceeds fifty per cent of the gravity load stress.

Paths of Stress. *Transmission of Load to Foundation.* The total wind pressure on the building in the direction under consideration is the assumed unit pressure per square foot multiplied by the projected area exposed to the pressure. This pressure must ultimately be resisted by the foundations of the building. Hence, there must be paths for transmitting the pressure to the foundations from the area to which it is applied. The pressure is applied directly to the masonry walls and windows. These are strong enough as ordinarily built to carry the load to the floors. The floor construction, whether of tile arches, concrete, or even wood construction, acting as a horizontal girder, transmits the load to the points selected for applying it to the steel framework. Thence the steel framework carries the load to the foundation.

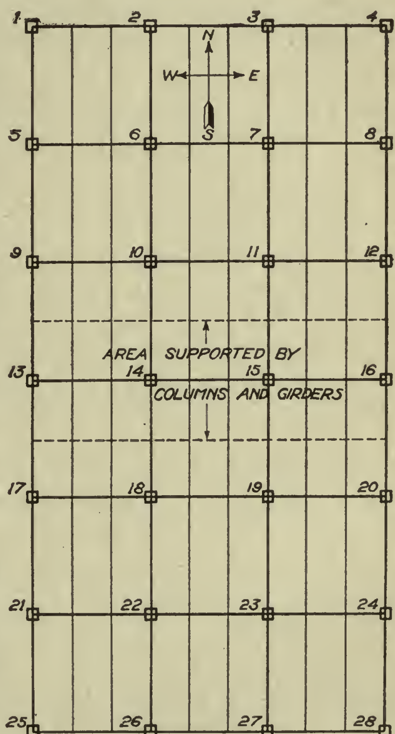


Fig. 170. Framing Plan of Building for Study of Bracing System

Routing the Stress. The designer has some choice as to the steel members which he will utilize for carrying the wind load. So far as the steel is concerned the shortest path is the best, but other considerations may require the use of less direct courses, most commonly through the spandrel beams around the outside of the building. Thus in Fig. 170 is shown a plan of the columns of a building, with the typical floor framing. The heavier lines represent girders and the lighter lines, joists.

Considering first the wind from either the East or the West, the direction of the load is parallel to the narrow way of the building and in the same direction as the floor girders. This situation indicates that the wind load should be carried down along each E.-W. row of columns, viz, 1-4, 5-8, 9-12, etc. Then each line of columns and its girders will have to support the wind pressure on one panel of the face of the building from top to bottom. It is probable that these columns and girders as designed for the gravity stresses will carry the wind stresses. (This of course is governed by the height of the building.) Now if it were decided to carry the entire load to the two ends and carry it through the columns and girders 1-4 and 25-28, the intensity of the stresses would be three times as great and probably would require extra metal in these members. Therefore, so far as economy of steel is concerned, the wind load should be carried down each row of columns. But it may happen that, in order to do this, deep brackets are required in the lower stories for connecting girders to columns, brackets of greater size than is permitted by the architectural requirements; then it becomes necessary to carry the load to the ends, where the spandrel beams and their connections can be made as large as need be. A combination of the two arrangements may be made, the load above a certain floor being carried down on each row of columns, and that below being carried down the end rows.

Next considering the wind from the North or the South, its direction is parallel to the joists. It is probable that these joists are not strong enough to take the wind stresses without adding metal to that required for the gravity stresses. The wind pressure can easily be carried to the two sides of the building along the lines 1-25 and 4-28, where the necessary strength in the spandrel girders can readily be obtained.

The foregoing illustration is comparatively simple; most cases are not so easy to settle. In general terms, the designer should take all possible advantage of interior framing, carrying through the spandrels only that portion of the wind load which cannot be taken by the interior framing.

The bracing strength of the interior framing is limited by the strength of the connections to the columns and not by the strength of the girder and joist sections. The maximum bending moments occur at these connections, and to develop the full strength of the beams would require larger brackets than the architectural treatment would permit. So generally it will be that a large proportion of the wind load must go through the spandrel beams where the limitations as to depth of beams and size of brackets are not so restricted.

It is sometimes possible to use diagonal members for bracing. They make the most direct and efficient form of bracing, and should be used when the conditions permit.

SYSTEMS OF FRAMEWORK

A horizontal load can be transmitted vertically by means of framework by two systems: (1) by triangular framework, Fig. 171,

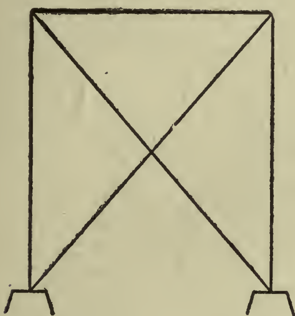


Fig. 171. Diagram of Triangular Framing.

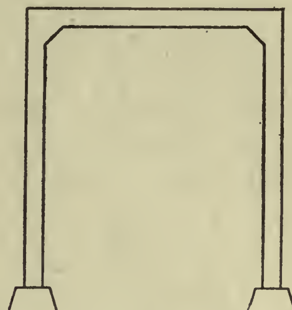


Fig. 172. Diagram of Rectangular Framing

having axial stresses; and (2) by rectangular framework, Fig. 172, having bending stresses.

Triangular Framework. *Single Panels.* Fig. 173 shows a single panel of triangular framing supporting the horizontal force W. The reactions at the foundations are R, V', and V.

$$R = W$$

$$V = V' = \frac{W\,H}{L}$$

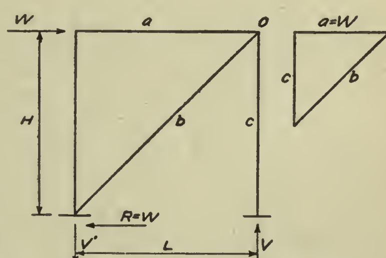


Fig. 173. Diagram of Stresses in Triangular Framing

By inspection it is to be seen that the stress in a equals W; in c equals V. The stresses in b and c can be determined from that in a by resolution of forces (See Concurrent Forces in "Statics"), as indicated in the figure. These stresses are all axial; a and c in compression; b in tension.

When the values of H, L, and W are known, the numerical values for a, b, c, and V can be determined.

Two or More Horizontal Panels. Two or more adjacent panels can be used, as shown in Fig. 174. It is first necessary to divide the load between the two panels. It is simplest to divide the load equally, irrespective of whether the panels are equal in length.

On this basis the stress in a equals W, and in d equals $\frac{1}{2}W$. By resolution, the stresses in b and c, and in e and f can be determined. V_1 equals the stress in c, V_3 equals the stress in f, and V_2 is the difference in stresses c and f. If in this case L_1 equals L_2, then the stress in b equals stress in e; the stress in c equals the stress in f; V_1 equals V_3; and V_2 equals 0.

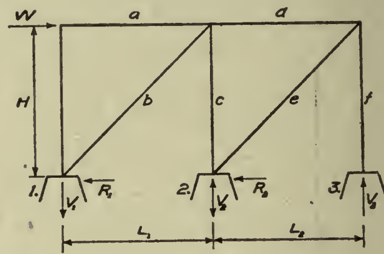


Fig. 174. Diagram of Two Horizontal Panels of Triangular Framing

PROBLEM

Assume four panels similar to those shown in Fig. 174. Let H equal 16 feet; L_1, L_2, L_3, and L_4 equal 20 feet; and W equal 36,000 pounds. Compute the stresses in the diagonals.

Two or More Vertical Panels. Two or more panels may be placed one above the other as in Fig. 175. In this case $R_1 = W_4 + W_3 + W_2$. The value of $V_1 = V_2$ is determined by taking moments about O from which

$$V_2 = \frac{W_2 H_1}{L} + \frac{W_3 (H_1 + H_2)}{L} + \frac{W_4 (H_1 + H_2 + H_3)}{L}$$

The stresses in the members a to k inclusive can be determined by the methods given in "Statics", when the values of W_4, W_3, W_2, H_3, H_2, H_1, and L are known and of R_1 and V_1 are computed.

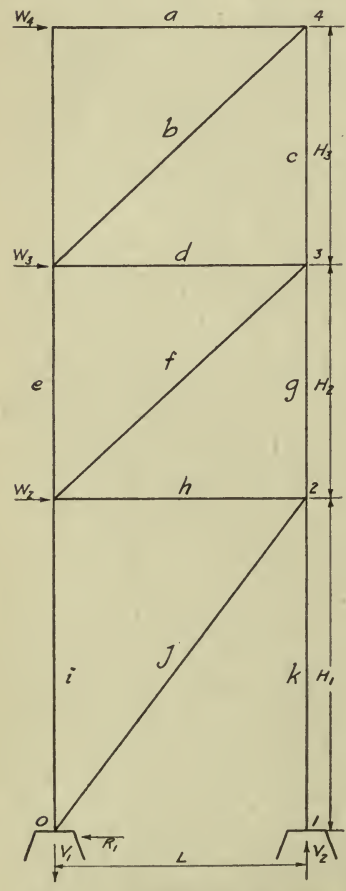


Fig. 175. Diagram of Vertical Panels of Triangular Framing

PROBLEM

In Fig. 175 assume W_4 equals 10,000 pounds; W_3 equals 10,000 pounds; W_2 equals 12,000 pounds; H_1 equals 18 feet; H_2 equals 13 feet; H_3 equals 13 feet; L equals 16 feet. Determine the stresses in a to k inclusive.

Extension of Triangular Framework. Similarly, the triangular framework can be extended indefinitely in both directions, as in Fig. 176. For convenience in solving this case the figure can be separated into horizontal tiers, or stories, and each computed. In doing this, the anti-reactions of one tier must be applied as loads

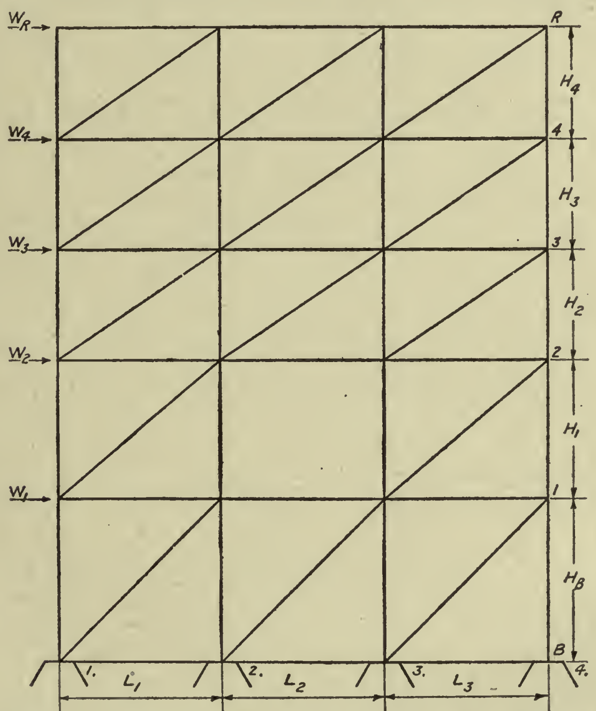


Fig. 176. Diagram of Triangular Framing Extending Over a Building

in the next lower tier. The horizontal load to be resisted at any tier is the sum of all the horizontal loads above that tier; thus the horizontal load or shear at the top of the first story is $W_R + W_4 + W_3 + W_2$.

PROBLEM

Assume loads and dimensions for Fig. 176 and compute the stresses in the diagonal members.

In Figs. 173 to 176 inclusive the diagonals are shown in one direction only. As the wind may come from either direction, both

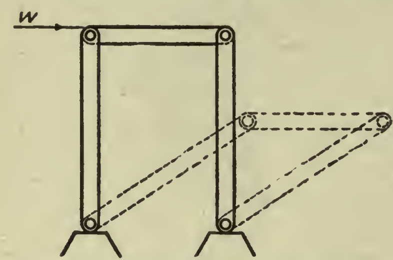


Fig. 177. Diagram of Rectangular Frame with Hinged Joints

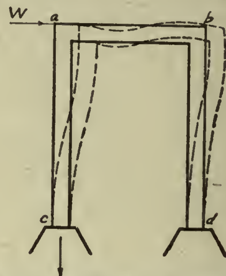


Fig. 178. Diagram of Rectangular Frame with Rigid Joints

diagonals will be used in all cases. In certain panels, circumstances may prevent the use of any diagonal bracing, Fig. 176, in

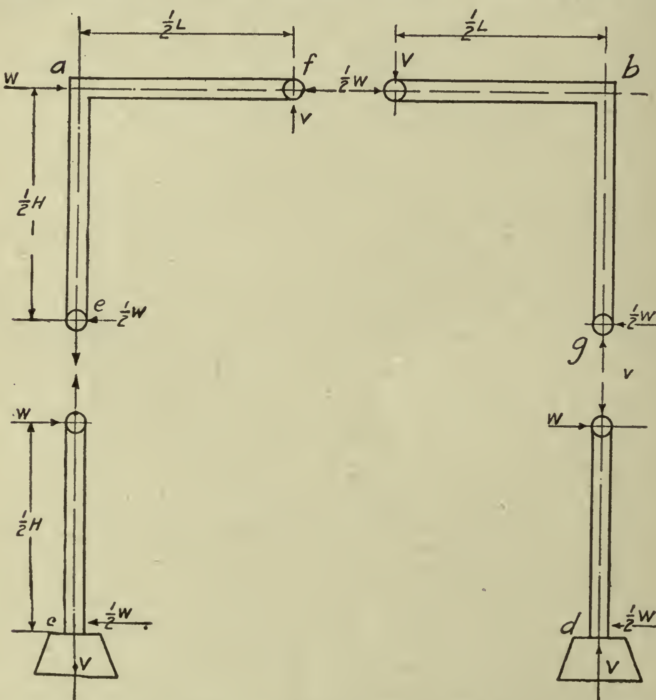


Fig. 179. Diagram of Rectangular Frame Showing Points of Contraflexure

which case the stresses must be distributed among the other panels.

Rectangular Framework. *Single Panel.* A single panel of rectangular framing is illustrated in Fig. 177. The four corners are represented as being hinged, so when the load W is applied the frame will collapse, as indicated by the dotted lines. It has no strength to resist the horizontal force.

Next consider the rectangular frame as shown in Fig. 178. The corners are rigidly connected. When the load W is applied, the frame tends to take the shape indicated by the dotted lines. In doing so, each of the members must bend into reverse curves. Thus the frame offers great resistance to the horizontal force.

When a member is bent into reverse curves, the point of reversal is called the "point of contraflexure". There is no bending stress in the member at this point and hinged joints might be introduced at such points without affecting the stability of the frame so far as the horizontal load is concerned. This is indicated in Fig. 179. The point of contraflexure is taken at the middle of the length of each member. This is not exactly correct, but is accurate enough for designing, in all ordinary cases.

In order to more easily understand the stresses in the frame, consider the points of contraflexure e, f, and g as hinged joints. They divide the frame into four parts which can be considered separately in determining the stresses. Take first $e\,a\,f$, and assume the horizontal reactions at e and f to be equal, hence each is $\frac{1}{2}W$. The vertical reactions at e and f must form a couple which will balance the moment of the horizontal loads, hence, taking moments about e,

$$V \times \tfrac{1}{2}L = \tfrac{1}{2}W \times \tfrac{1}{2}H$$

from which

$$V = \tfrac{1}{2}W\frac{H}{L}$$

The bending moment at a in the vertical member, is $\frac{1}{2}W \times \frac{1}{2}H$ or $\frac{1}{4}WH$; and in the horizontal member is $V \times \frac{1}{2}L$ which equals $\frac{1}{2}W\frac{H}{L} \times \frac{1}{2}L$ or $\frac{1}{4}\,WH$.

Next consider the part $e\,c$, which is subjected to the loads $\frac{1}{2}W$ and V applied at e. The reactions at c are the same in amount but opposite in direction. To maintain equilibrium, there must be a couple to neutralize the moment of the horizontal force at e about the center c. This couple is furnished by the foundation which is assumed to be ample to resist the bending moment in the post at c, which is

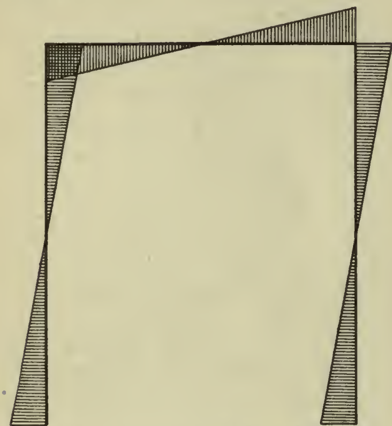


Fig. 180. Moment Diagram of Single Rectangular Panel

$$\tfrac{1}{2}W\times\tfrac{1}{2}H=\tfrac{1}{4}\,WH$$

In like manner the bending moments at b and d can be shown to be $\frac{1}{4}\,WH$. Note that the numerical value of the bending moment is the same at the four corners of the frame. The moment diagram is given in Fig. 180.

In addition to the bending stresses in the members, there are axial stresses, as indicated by the forces and reactions illustrated:

in $a\,b$	$\frac{1}{2}\,W$, compression
in $b\,d$	$V=\frac{1}{2}\,W\,\frac{H}{L}$, compression
in $a\,c$	$V=\frac{1}{2}\,W\,\frac{H}{L}$, tension

Problem

Refer to Fig. 179. Assume W equals 10,000 pounds, H equals 16 feet, L equals 20 feet. Compute the axial stresses in the three members of the frame. Compute the bending moment at a. Construct the moment diagram.

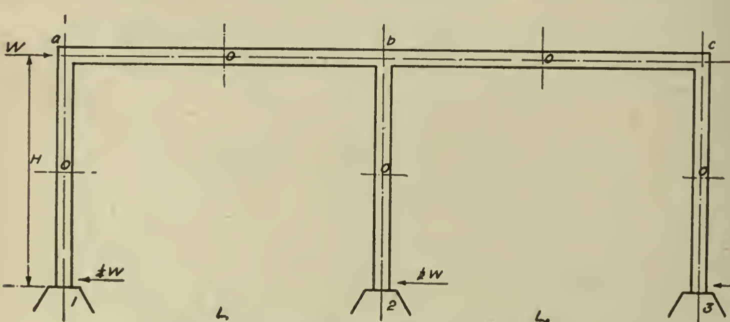


Fig. 181. Diagram for Two Rectangular Panels

Two Horizontal Panels. Next consider a framework of two panels, i. e., made of three columns and two girders, as in Fig. 181, subjected to a load W. It is necessary to assume the division of the horizontal reactions between the foundations *1, 2,* and *3*. Several different methods are used in practice. It is not of much importance which is used, if the stresses resulting from the assumed divisions are adequately provided for. In this text it is assumed that the reactions at the end columns are one-half of those at the intermediate columns. Thus the reactions at *1, 2,* and *3* are $\frac{1}{4}W$, $\frac{1}{2}W$, and $\frac{1}{4}W$, respectively. By reasoning similar to that used for the single panel, the maximum bending moments are found to be:

at the base and top of columns *1* and *3*,	$\frac{1}{4}W\times\frac{1}{2}H=\frac{1}{8}WH$
at the base and top of column *2*,	$\frac{1}{2}W\times\frac{1}{2}H=\frac{1}{4}WH$
and in the girders to the right of a and b and to the left of b and c,	$\frac{\frac{1}{4}WH}{L}\times\frac{1}{2}L=\frac{1}{8}WH$

In analyzing this case, the frame may be considered as made up of two separate panels, each of which carries one-half the load W.

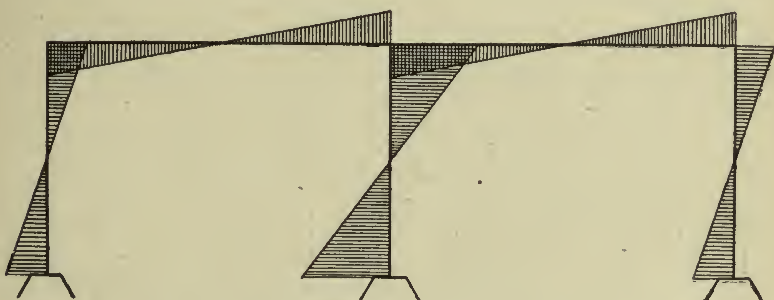


Fig. 182. Moment Diagram for Frame of Two Rectangular Panels

Then the bending moment at all maximum points is $\frac{1}{8}WH$. But column *2* is common to both, hence its total stresses are the algebraic sums of the stresses from the two panels. As the bending stresses are of the same sign, the bending stresses in column *2* are twice those in columns *1* and *3*; on the other hand the axial stresses in column *2* are opposite in sign and tend to neutralize each other. The resultant is zero if L_1 equals L_2. The moment diagram of this case is given in Fig. 182.

Horizontal Row of Panels. The foregoing method now can be applied to a frame of any number of panels. The total horizontal load or shear is divided by the number of panels. Give one portion to each of the intermediate columns and one-half portion to each of the outside columns. Thus in Fig. 183 there are five panels.

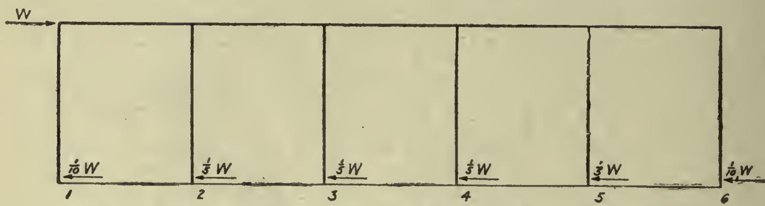


Fig. 183. Diagram Showing Division of Shear in a Frame of Five Panels

The shear is distributed thus: $\frac{1}{10}W$ at columns *1* and *6*, and $\frac{1}{5}W$ at columns *2, 3, 4,* and *5*. The bending moments in columns *1* and *6* are: $\frac{1}{20}WH$; in columns *2, 3, 4,* and *5*, $\frac{1}{10}WH$; and in all girders, $\frac{1}{20}WH$.

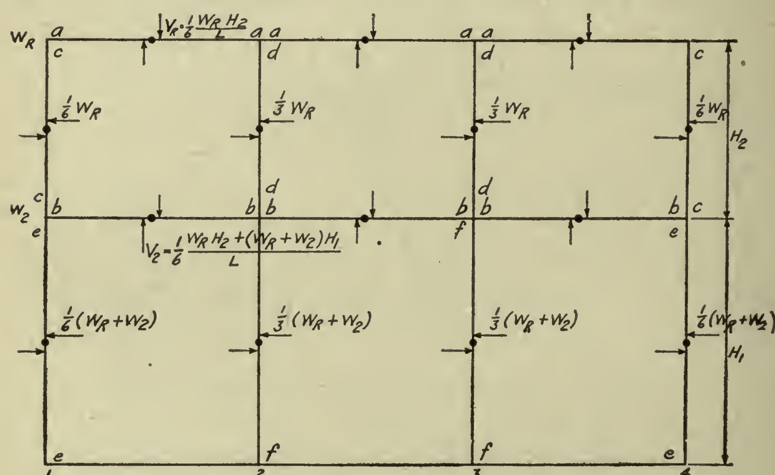


Fig. 184. Stresses in a Two-Story Rectangular Framework

PROBLEM

Assume a frame of 7 panels, supporting a wind load of 115,000 pounds. Let H equal 14 feet. Compute the maximum bending moments and draw the moment diagram.

Two-Story Framework. Next assume the case illustrated in Fig. 184. This shows the framework of a two-story building. The points of contraflexure occur at the points indicated by the black dots. The loads applied are W_R at the roof and W_2 at the second floor. The first-story frame serves as a foundation for the second-story frame. The horizontal shears which are transmitted through the points of contraflexure in the second-story columns are $\frac{1}{6} W_R$ and $\frac{1}{3} W_R$ as indicated; those transmitted through the points of contraflexure in the first-story columns are $\frac{1}{6}$ (W_R+W_2) and $\frac{1}{3}$ (W_R+W_2) as shown. The vertical shears transmitted through points of contraflexure in the roof girders are $V_R=\frac{1}{6}\frac{W_RH_2}{L}$, and those transmitted through the second-floor girders are

$$V_2=\frac{1}{6}\frac{W_RH_2+(W_R+W_2)H_1}{L}$$

(assuming panels of equal length). Then the bending moments are

at *a* in roof girders	$-\frac{1}{12}W_RH_2$
at *b* in 2nd floor girder	$-\frac{1}{12}[W_RH_2+(W_R+W_2)\ H_1]$
at *c* in columns	$+\frac{1}{12}W_RH_2$
at *d* in columns	$+\frac{1}{6}W_RH_2$
at *e* in columns	$+\frac{1}{12}(W_R+W_2)\ H_1$
at *f* in columns	$+\frac{1}{6}(W_R+W_2)\ H_1$

An important relation to be noted is that at any joint the sum of the moments in the members equals zero, or the sum of the moments in the column equals the sum of the moment in the girders. Thus at column *1*, 2nd floor

$$\frac{1}{12}W_RH_2+\frac{1}{12}(W_R+W_2)\ H_1-\frac{1}{12}[W_RH_2+(W_R+W_2)\ H_1]=0$$

at column *2*, 2nd floor

$$\frac{1}{6}W_RH_2+\frac{1}{6}(W_R+W_2)\ H_1-2\times\frac{1}{12}[W_RH_2+(W_R+W_2)\ H_1]=0$$

Extension of System in Either Direction. The method can now be applied to a frame of any extent, vertically and horizontally. Fig. 185 shows such a frame six panels in width and six stories and basement in height. The loads applied at the several floor levels are represented by W_1, W_2 . . . . W_R. The total shears in the several stories are represented by W_B', W_1', W_2'. . . . . . W_6'.

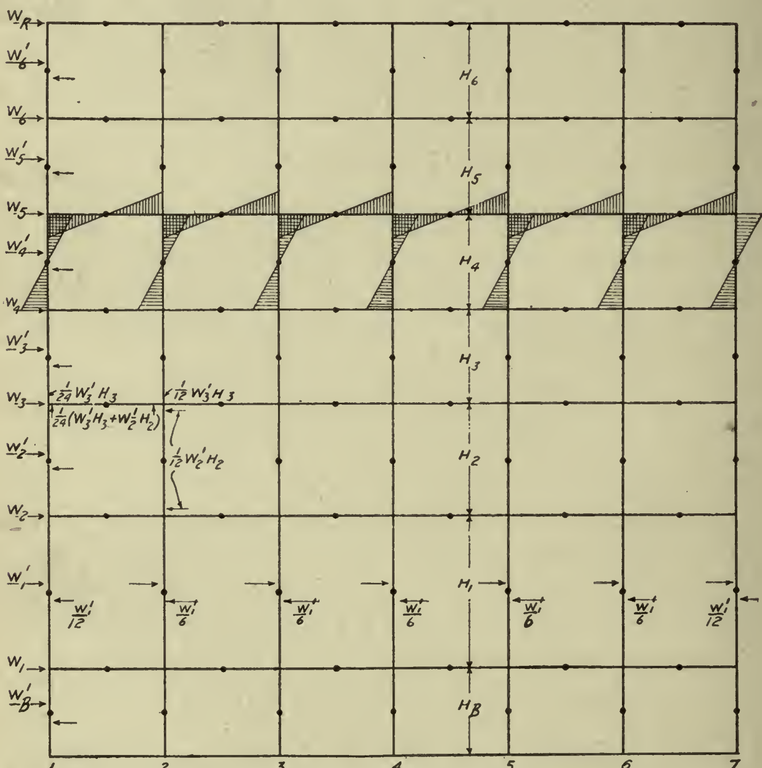


Fig. 185. Diagram of Wind Stresses in a Building Frame

The total shear in any story is the sum of all the loads applied at the floors above, thus,

$$W_2' = W_3 + W_4 + W_5 + W_6 + W_R$$

The total shear in any story is divided between the columns in that story in accordance with the rule given. This is illustrated in the figure by the values given in the first story.

The bending moments are illustrated at the third floor in the figure and the moments diagrams at the fifth floor.

The procedure can now be reduced to simple rules and formulas.

The bending moment in an intermediate column in any story equals the total shear in that story multiplied by the story height, and the product divided by two times the number of panels. This is expressed by the formula

$$M=\frac{W'H}{2n}$$

The bending moment in an outside column is one-half that in an intermediate column, or,

$$M=\frac{W'H}{4n}$$

The bending moment in a girder is the mean between thé bending moments in the column above and below the girder. It is expressed by the formula

$$M=\frac{1}{2}\left(\frac{W_a'H_a}{2n}+\frac{W_b'H_b}{2n}\right)=\frac{1}{4n}\left(W_a'H_a+W_b'H_b\right)$$

NOTE. a and b refer to two adjacent stories, as the third and fourth. The panel length does not affect the value of the bending moment.

Illustrative Example. Compute the bending moments at the first floor in the frame in Fig. 185. Assume that the loads applied above the first story sum a total of 66,000 pounds equal W_1', those above the basement story a total of 75,000 pounds equal W_B'. Let H_B equal 10 feet, and H_1 equal 16 feet. Then the bending moment is:

in an intermediate basement column

$$\frac{75{,}000\times 10}{2\times 6}=62{,}500 \text{ ft.-lb.}$$

in the intermediate first-story columns

$$\frac{66{,}000\times 16}{2\times 6}=88{,}000 \text{ ft.-lb.}$$

in the first-floor girders

$$\frac{62{,}500+88{,}000}{2}=75{,}250 \text{ ft.-lb.}$$

Axial Stresses. The axial stresses may be disregarded in most cases. They are usually small in proportion to the sections otherwise required for the members. The girders may be considered as being relieved from this stress by the floor construction. If there be no floor construction along the girders, the axial stress should be considered. In the intermediate columns the axial stress is zero if the panel

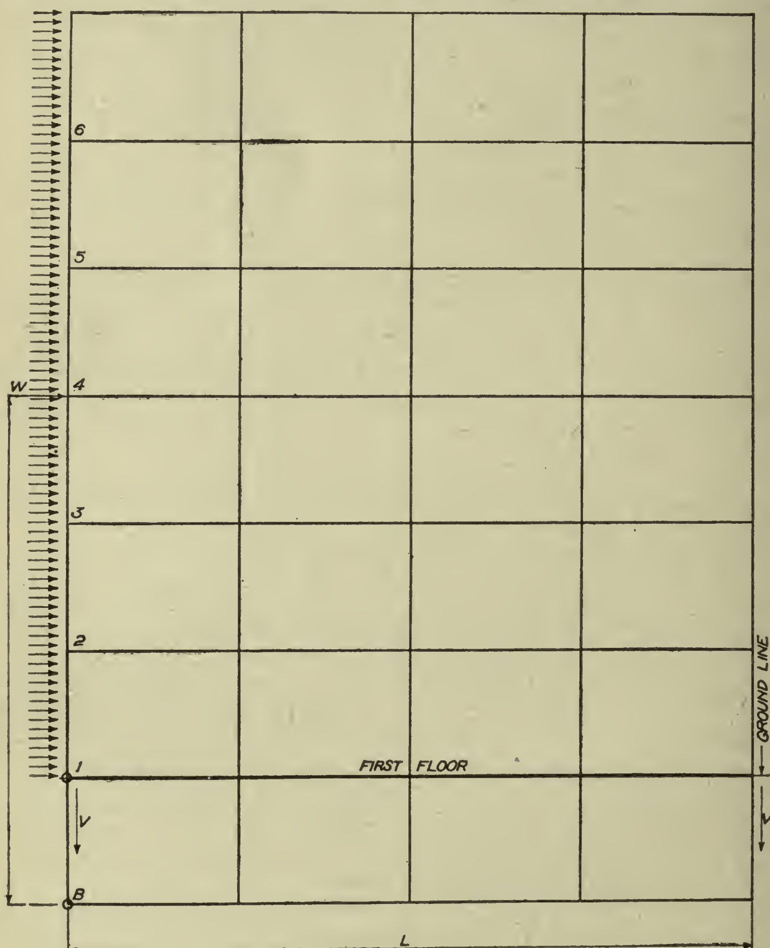


Fig. 186. Diagram of Overturning Stresses in a Building Frame

lengths are equal. In the outside columns the axial stress occurs, but here the bending moment is only one-half that in the intermediate columns, so the axial stress is usually not important; however, in tall, narrow buildings it may be important and should be computed. When required, it can be computed thus: In Fig. 186 the arrows represent the wind pressure on the framework shown. The resultant of this pressure is W, acting at mid-height of the exposed part of the structure. The axial stress V in the basement section of the end column is found by taking moments about the point B. The stress in the first-story section is found by taking moments about the point *1*.

PROBLEMS

1. Assign values to the structure illustrated in Fig. 186 and compute the axial stress in the second-story sections of the end columns.

2. In Fig. 185 assume the following values:

H_B	$=10'\text{-}0''$
H_1	$=16'\text{-}6''$
$H_2, H_3 --- H_5$	$=12'\text{-}6''$
W_1	$= 8{,}000\#$
W_2	$=14{,}500\#$
W_3, W_4, W_5, W_6	$=12{,}500\#$
W_R	$=10{,}000\#$

(a) Compute W_B', W_1', ------ W_6'.

(b) Compute the maximum bending moment for an interior column above and below each floor line.

(c) Compute the maximum bending moment in the girders at each floor.

(d) What is the bending moment in the second-floor girder at a point 1'-9" to the right of column *4*?

(e) Construct the moment diagram for column *7* from basement floor to roof.

DESIGN OF WIND-BRACING GIRDERS

In the preceding pages the method has been developed for determining the bending moments in wind-bracing girders and columns. It has been shown that the maximum bending moment occurs at the intersection of the column and the girder, and zero moment occurs at the center of the girder. Between these points the moment varies uniformly, as shown by the moment diagrams in Figs. 180, 182, and 185. By laying out the moment diagram to scale, the bending moment at any point may be measured.

End Connections for Riveted Girders. Heretofore in designing beams, end connections have been required to resist only vertical shear, but in the case of wind-bracing girders it is evident that the connection of the girders to the column is chiefly to resist the bending moment. This connection requires careful designing to insure effective results.

To illustrate the design, assume an example as follows: In Fig. 187 the distance center to center of columns is 20 feet; the max-

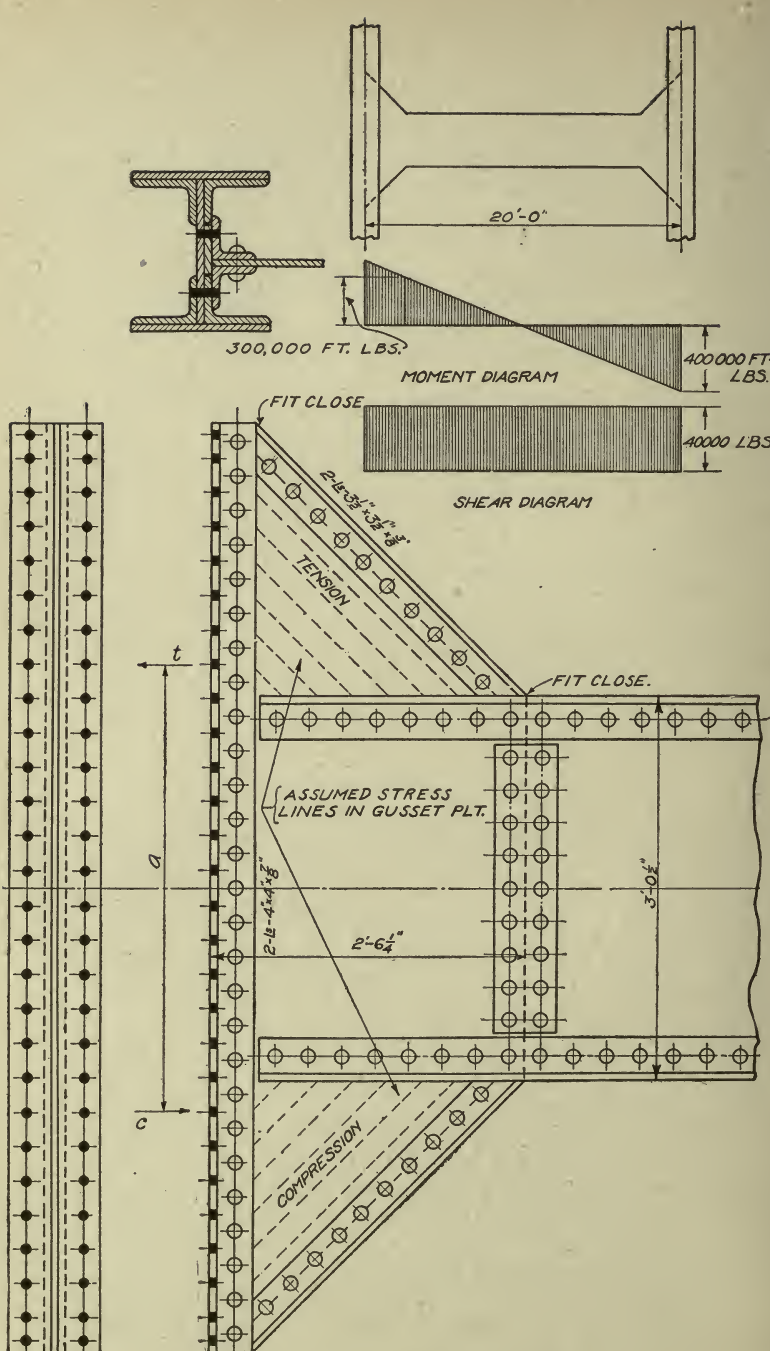


Fig. 187. Design of a Wind Bracing Girder

imum bending moment is 400,000 foot-pounds or 4,800,000 inch-pounds; the depth of girder is 3 feet $\frac{1}{2}$ inch back to back of angles. As stated on page 51, the unit stresses to be used are fifty per cent in excess of those allowed for gravity loads.

The girder connects to the web of the column. As the end of the girder thus lacks only about an inch of reaching to the column center, the maximum bending moment must be provided for, viz, 4,800,000 inch-pounds.

Rivets Connecting Girder to Column. The rivets through the end angles and column webs are field driven, $\frac{7}{8}$ inch diameter, and on the tension side of the girder (above the neutral axis in this case) are in tension. As in a beam, the unit fiber stress varies from zero at the neutral axis to a maximum at the extreme fiber; so the unit stress in these rivets varies from zero at the neutral axis to the maximum allowable amount at the farthest rivet.

Then, if the rivets are equally spaced, the average stress is one-half the maximum. The total resistance of the rivets is the average value of one rivet multiplied by the number of rivets in the tension (or compression) group represented by t (and c); the centers of gravity of the groups are at the points t and c. The moment arm is the distance a between t and c, and the resisting moment is $a \times t$ (or c).* The number of rivets required is determined by trial. The full value of a $\frac{7}{8}$-inch rivet, field driven, in tension is one and one-half times 6000 pounds or 9000 pounds. Several trials lead to the use of 28 rivets on each side of the neutral axis. The value of t is $\frac{9000 \times 28}{2}$ or 126,000 pounds. The moment arm a is 42 inches and the resisting moment of the joint is $126{,}000 \times 42$ or 5,292,000 inch-pounds, which is about ten per cent in excess of the bending moment.

PROBLEM

Design the above joint, using $\frac{3}{4}$-inch rivets spaced $2\frac{1}{2}$ inches.

Rivets Connecting End Angles to Gusset Plate. Now consider the rivets connecting the end angles to the gusset plate. The method is the same as that for the connections of the end angles to the column, except that the rivets are shop driven in double shear.

*This is not exact, for the rivets on the compression side do not act, the compression being resisted by the direct bearing of the end of the girder against the column. The error is on the safe side.

The required results can easily be obtained by comparison with field-driven rivets. With one row of rivets there will be one-half as many (less one). One shop rivet in double shear is good for 21,660 pounds. This is greater than the value of two rivets in tension (18,000 pounds), hence the proposed arrangement is satisfactory. It gives greater strength than is required.

The thickness of gusset plate required to develop the full shearing value of the rivets is $\frac{11}{16}$ inch. The thickness required for the actual stress is $\frac{9}{16}$ inch, which use. (See rivet tables in handbook.)

PROBLEM

What thickness of gusset plate is required for $\frac{3}{4}$-inch shop rivets?

Bending Stresses in Connecting Angles. No accurate determination can be made of bending stresses in connecting angles, so thickness must be adopted arbitrarily. If the gage line of the rivets is not more than $2\frac{1}{2}$ inches from the back of the angle, the thickness should be $\frac{5}{8}$ inch. In many cases wide angles with large gage distance must be used in order to match the gage lines in the column. A thickness of 1 inch seems to be safe for a gage distance of 4 inches. Intermediate values may be interpolated.

Gusset Plate. The slope of the gusset plate should be about 45 degrees, but may vary to suit conditions, such as clearance from windows, etc. Stresses in the gusset plate may be imagined to act along the dotted lines shown in the figure. On the tension side of the girder the plate is in tension, and on the compression side in compression. The thickness of plate required for rivet bearing is sufficient to give the necessary strength on the tension side, but on the compression side stiffener angles may be required. These angles can be designed according to rules similar to those given for the stiffeners of plate girder webs, p. 148. They should be used when the length of the diagonal edge of the plate is more than thirty times the thickness. The leg of the angle against the plate should be of suitable width for one row of rivets, say 3 inches, $3\frac{1}{2}$ inches, or 4 inches. The outstanding leg may vary from 3 to 6 inches. A thickness of $\frac{3}{8}$ inch is suitable usually; it may be made more or less to be consistent with size and thickness of the main members of girder. For the case illustrated use 2Ls $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$.

Girder Section. The critical section of the main girder is at the end of the gusset plate (because there are no gravity loads). The gusset plate being 2′-6″ wide, the bending moment at this point, as determined from the moment diagram, is 300,000 foot-pounds, or 3,600,000 inch-pounds, Fig. 187.

It is usually economical to make the girder as deep as conditions will permit. In most cases it is limited by the windows above and below. For this case 3′-0½″ back to back of angles is assumed.

The section is determined by the methods given for riveted girders, p. 141, using the increased unit stresses previously mentioned. Note that the web is spliced at the point under consideration.

The spacing of rivets that connect the flange angles to the web plate is determined as in riveted girders, p. 149. As the bending moment varies uniformly from the center to the end, the rivets are equally spaced. This spacing may be continued for connecting the flange angles to the gusset plate. But there must be enough rivets through the gusset plate to transmit all of the stress which is in the flange angles at the edge of the gusset. Connecting angles may be needed to assist in connecting the flange angles to the gusset plate.

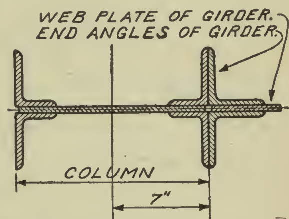


Fig. 188. Section of Connection of Girder to Column

PROBLEMS

1. Design the girder section, flange riveting, and web splice, Fig. 187.

2. Make drawing at 1-inch scale showing side elevation, end elevation, and section of the girder. (Use the design with ¾-inch rivets.) Show rivet spacing.

Other Forms of End Connections. Fig. 188 shows a girder connection differing from the previous case in that the column is turned in the other direction. The connection is designed in just the same manner but the amount of the bending moment is somewhat less than the maximum because it is some distance away from the center of the column. The actual amount can be computed or scaled from the moment diagram.

PROBLEM

What is the bending moment at the end of the girder shown in Fig. 188, the moment at the center of the column being 400,000 foot-pounds and the distance, center to center of columns, 16 feet?

In Fig. 189 the web of the girder connects directly to the flange of the column. This form of connection is suitable for girders which are deep in proportion to the bending moment which they must resist. The method of designing the connection is the same as that explained for Fig. 187, except that the rivets are in single shear instead of tension, and that the rivets are not evenly spaced, hence the average resistance may not be one-half the maximum. The value of each rivet can be measured from the diagram at m in the

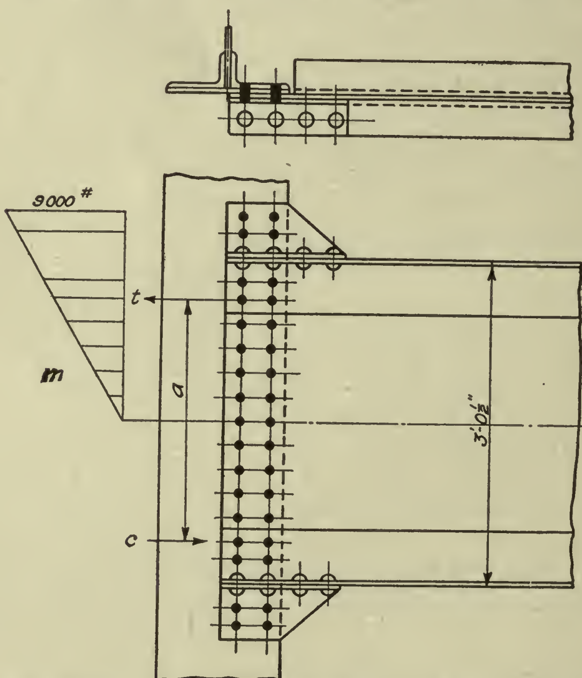


Fig. 189. Details of Connection of Girder Directly to the Face of the Column

figure. Having the values of the several rivets, the center of gravity of each group, i. e., the positions of the resultants t and c, can be found in the usual way.

PROBLEM

Compute the resisting moment of the connection shown in Fig. 189. Use ¾-inch rivets, and assign suitable spacing for them. Design the girder section corresponding to this resisting moment.

When the form of connection shown in Fig. 189 is not adequate, the gusset plate can be used connecting directly to the flange of the column. It involves no principles or methods different from those already explained.

End Connections for I-Beam Girders. I-beam connections for resisting bending are illustrated in Figs. 190, 191, and 192.

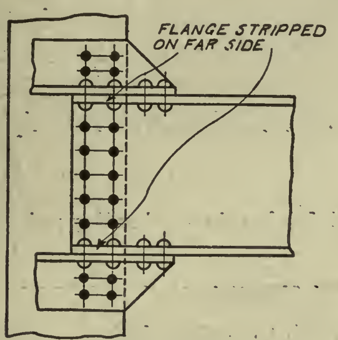


Fig. 190. Connection of I-Beam to Flange of Column For Wind Bracing

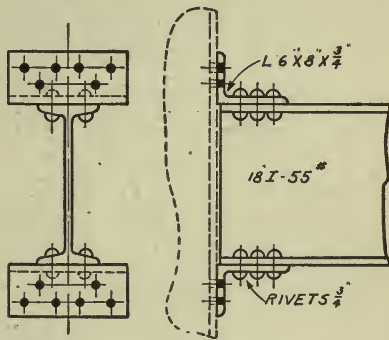


Fig. 191. Connection of I-Beam to Side of Column for Wind Bracing

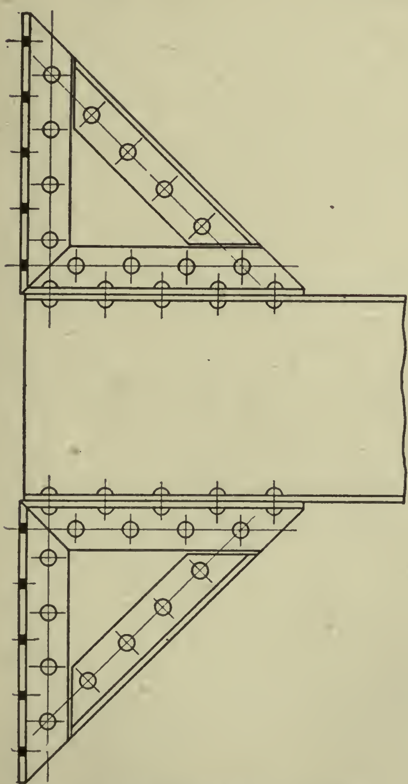


Fig. 192. Bracket Connection of I-Beam for Wind Bracing

The detail in Fig. 190 is similar to the connection shown in Fig. 189. It can develop only a small part of the capacity of the beam.

The detail in Fig. 191 also can develop only a part of the capacity of the beam, but it is available for making use of the floor girders in the upper part of the building for resisting wind stresses. The strength of this connection is limited by the bending resistance of the connecting angles or the strength of the rivets.

Problem

Compute the bending resistance of the connection shown in Fig. 191.

Bracket Connection. The connection in Fig. 192 can be made to develop the entire net bending resistance of the beam (deducting for rivet holes in the flanges). The connection of the brackets to the column is designed in the same manner as described for the gusset plate connection. The average value of the rivets is determined from the diagram as at m, Fig. 189. In the connection of the brackets to the beam, all the rivets are figured at the maximum value. Their resisting moment is their total shear value multiplied by the depth of the beam.

PROBLEM

Design a bracket connection that will develop the net bending resistance of a 24″ I 80#.

COMBINED WIND AND GRAVITY STRESSES IN GIRDERS

The girders which are usually used to resist wind stresses are also subjected to gravity stresses in supporting walls and floors. It is necessary, therefore, to determine the combined effect before the member can be designed.

Moment Diagram for a Restrained Beam. In the discussion of beams, it was considered that the ends rested freely on the supports. With these conditions the beam under a gravity load tends to deflect in the form of a simple curve and its moment diagram lies entirely below the axis *o-o*, Fig. 193-a. If the beam is restrained by rigid connections at the ends, as illustrated in Fig. 192, it tends to deflect in the form of a compound curve and the moment diagram, Fig. 193-b, lies both above and below the axis. The part of the diagram above the axis represents negative moment and the part below, positive moment. The total depth of the moment diagram is $\frac{1}{8} W L$ (for a uniformly distributed load) in each case.

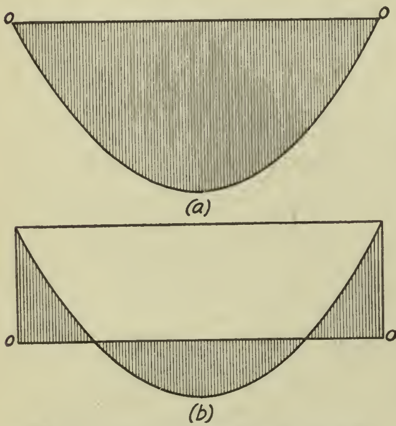


Fig. 193. Moment Diagrams (a) of Simple Beam; (b) of Restrained Beam

Positive and Negative Moments. The division of the moment diagram of a restrained beam between positive and negative moments depends on a number of conditions. The conditions usually assumed as ideal are that the beam is of constant cross section from end to end and that the end connections are absolutely rigid. Then the bending moment at the ends is $-\frac{1}{12} W L$, and at the middle is $+\frac{1}{24} W L$.

If the section of the beam at mid-span is less than at the ends, as is the case when the connections are made by deep gusset plates or brackets, the positive moment is less and the negative moment greater than the above values. The extreme case would be when a beam had no bending resistance at the center (as if hinged), in which case the two halves would act as cantilevers; there would be no positive moment and the negative moment would equal $\frac{1}{8} W L$ (W being the total load on the span).

The assumed ideal condition of absolute rigidity at the ends is not realized because the columns must deflect laterally under load. This lack of absolute rigidity tends to decrease the negative moment and to increase the positive moment. The same effect is produced if the connection is not sufficient to develop the strength of the beam section, as in the examples shown in Figs. 190 and 191. In the extreme case when the columns or the connections are extremely weak in bending resistance, the negative moment approaches zero and the positive moment approaches $\frac{1}{8} W L$.

It is not practicable to determine definitely the amount of negative and positive moments for a given case, so arbitrary values must be adopted. The designer generally should assume that the moments from the gravity loads are $-\frac{1}{12} WL$ at the ends and $+\frac{1}{24} WL$ at mid-span, and should design the end connections and the beam section accordingly. But a less value may be used at the ends and a corresponding greater value at the center if it is not possible to make end connections strong enough to resist the larger value.

Bending Moments for Combined Loads. Now consider the bending moments resulting from the combined action of gravity and wind loads. In Fig. 194, let a be the moment diagram for a wind load and b the moment diagram for a gravity load. Then the total effect is represented by c, which is the moment diagram for

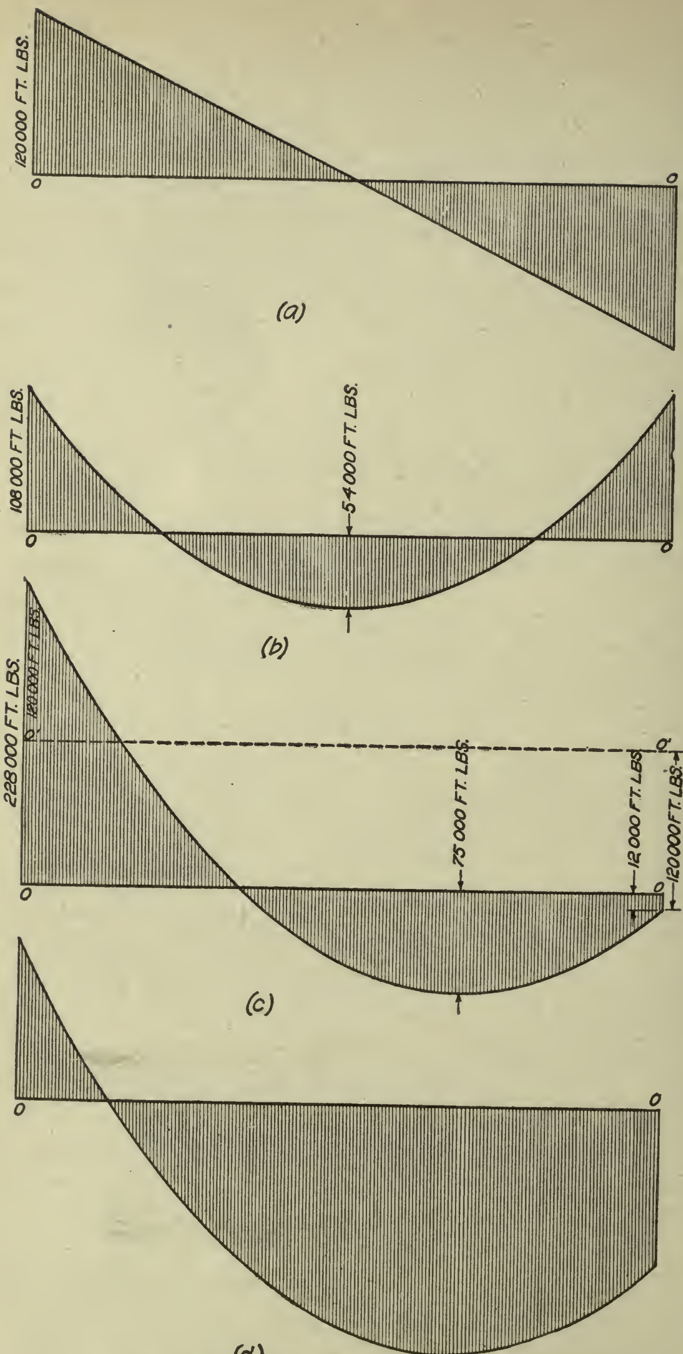


Fig. 194. Moment Diagrams for Combined Loads
(a) Wind Load (b) Gravity Load
(c) Combined Wind and Gravity Loads
(d) Maximum Bending Moment Diagram

the combined loads. This moment diagram *c* is constructed by adding together the moments used in constructing the diagrams *a* and *b*.

End Connections Designed to Resist Wind Loads. Diagram *c*, Fig. 194 shows a very large resultant negative bending moment at the left end of the diagram, and a very small resultant positive bending moment at the right end. If the respective end connections be designed to resist these moments, i. e., the left end with a very heavy connection and the right end with a very light connection (in this case practically a hinged joint), then the distribution of stresses probably would be as represented in diagram *c*. But, since the wind may act from either direction, the two end connections are made alike; the columns at the two ends are probably of about equal size and stiffness; then it is reasonable to assume that the deflections, and hence the resistance developed at the two ends, will be equal.

For this condition it is evident that diagram *c* does not represent the actual distribution of moments. To have a diagram which will represent it, the curve must be shifted so that the negative moment at the left end equals the positive moment at the right end. This gives diagram *d*. The same diagram results directly by combining diagram *a* of Fig. 193 with diagram *a* of Fig. 194. It will be noted that the bending moments at the ends equal the bending moments from the wind loads. Hence, *the end connections in all cases are designed to resist the wind load moments.*

Maximum Bending Moment. The bending moment at the center of the span equals the bending moment of the gravity load computed for an unrestrained beam. However, the maximum positive bending is not at the center, but some distance to one side (to the right in this case) and its amount can be determined by constructing the diagram *d*. *The value thus determined governs the cross section of the girder.*

As has been stated, the unit stresses allowed for the combined loads are 50 per cent larger than those for the gravity load alone. The resulting section designed for the maximum positive bending moment from diagram *d* will always be larger than the section required by the negative moment of gravity load from diagram *b* and more than twice the section required by the maximum positive bending moment from diagram b, diagram b being the moment diagram for the gravity loads on a restrained beam, when the wind is not acting. Note, however, that the section required is less than would be required for the gravity load on a simple (unrestrained) beam, diagram a, Fig. 193.

PROBLEMS

1. In Fig. 194 assume values given for diagrams a and b. Determine the maximum positive and negative values for diagram d. Construct diagram d accurately to scale.

2. Design a girder of the type shown in Fig. 187 from the moment diagram d in Fig. 194.

EFFECT OF WIND STRESSES ON COLUMNS

Combined Direct and Bending Stresses. The bending moment on the column due to wind loads produces the same sort of stresses as result from the bending moment due to eccentric loads or any other cause producing flexure. The extreme fiber stress is computed from the formula

$$S=\frac{Mc}{I}$$

This stress is added to the stresses resulting from the direct and eccentric loads on the column to give the maximum fiber stress.

Fig. 195. Diagram of Combined and Direct Stress

The combination of the direct and the bending stress is illustrated in Fig. 195. The stress from the direct load is represented by the rectangle $a\ b\ c\ d$ and the unit stress by $a\ b$. The stress from bending is represented by the triangles $b\ b'o$ and $c\ c'o$, the extreme fiber stress being $b\ b'$ in compression and $c\ c'$ in tension. Then the maximum fiber stress is on the compression side and is $a\,b+b\,b'$. Thus $b\ b'$ represents the increase in stress due to the wind load. If, as is usually the case, $b\ b'$ amounts to less than half $a\ b$, the column section required for the direct load need not be increased on account of the wind stress, because of the increased units allowed for combined stress. But if $b\ b'$ exceeds one-half of $a\ b$, the combined stress will govern the design using the increased unit stress.

On the tension side of the column, the wind stress will very rarely be great enough to overcome the direct compression. And if there should be a reversal of stress, there cannot be tension enough to require any addition to the section. It frequently occurs that the wind bracing girder connects to the column in such a position that one side of the column must resist practically all the wind stress. Such a case is illustrated in Fig. 189. With these conditions only one-half the column section should be used in computing the resulting extreme fiber stress.

Design of Column for Combined Stresses. The procedure in designing the column section, when the combined wind and gravity loads govern, is the same as has been given for columns with eccentric loads, p. 174. The method there given for computing the concentric equivalent load also applies, as well as the formula

$$W_w' = W'\frac{e\,c}{r^2}$$

As applied to wind load (refer to Fig. 196) W_w' is the equivalent concentric load, i. e., the direct load that would produce the same unit stress; W' is the horizontal shear which is assumed to be carried by the column under consideration and is assumed to be applied at the point of contraflexure of the column (see Fig. 185); e is the moment arm expressed in inches, hence $W'e$ is the bending moment in inch-pounds at the section under consideration; c is the distance from the neutral axis of the column to the extreme fiber on the compression side; r is the radius of gyration of the column in the direction under consideration. The critical section of the column is at the top of the bracket, as the bracket has the effect of enlarging the column section, so the distance e is measured to that point.

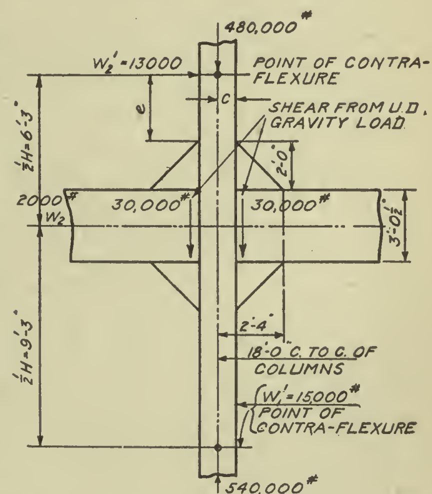


Fig. 196. Details of a Problem in Wind Bracing

To illustrate the use of the formula assume the following data:

Direct or gravity load on column is 600,000 pounds; W' is 10,000 pounds; e is 30 inches; c is 7 inches; and r is 3.5 inches. Then

$$W_w' = \frac{10{,}000 \times 30 \times 7}{3.5 \times 3.5} = 171{,}400\#$$

As this is less than half the gravity load it is neglected.

PROBLEM

In Fig. 196 are given the essential dimensions and the loads on the columns in the first and second story of a building and the girders at the second floor.

(a) Design the columns and girders.

(b) Write a complete record of all computations.

(c) Make a drawing of the joint at $\frac{3}{4}$-inch scale.

FORT DEARBORN HOTEL, CHICAGO
Courtesy of Holabird and Roche, Architects, Chicago

STEEL CONSTRUCTION

PART IV

PRACTICAL DESIGN

SIXTEEN-STORY FIREPROOF HOTEL

Having studied the stresses and the design of individual steel members, attention will now be given to the problems which arise in the design of the structural framework of a building.

It is assumed that the student now understands how to compute stresses and how to design individual members of the framework; therefore, detailed computations of these operations in most cases are not given. Nor are references given to the preceding parts of the work, except in a few cases, it being left to the student to seek these references for himself if he needs them. This applies also to the tables and diagrams in this book and in the handbooks.

Description of Building*. The building selected for the purpose of illustrating the practical problems of design has been taken because it gives an unusually large number of special conditions. For this reason it cannot be considered as a typical case. Its framing differs from that most commonly seen in buildings because steel joists are not used.

The building is designed to be used as a hotel. It has sixteen stories and an attic above street level and a basement below street level. It also has a sub-basement over part of the area to provide space for a power plant. The basement extends under the sidewalk on two sides of the building.

The building occupies the entire lot, except for a light court above the third-floor level. Fireproof construction is used throughout. The framework consists of structural steel columns and girders. The floor construction consists of reinforced concrete slabs and joists, with tile fillers between the joists. In most of the

*The Fort Dearborn Hotel, Chicago, Illinois; *Holabird and Roche, Architects.*

building the concrete slabs form the finished floor. Partitions in general are three-inch hollow tile, plastered on both sides. They are fixed in position (this has some bearing on the arrangement of girders). The foundations are cylindrical concrete piers extending to rock. The basement walls are of reinforced concrete. The walls above grade are brick with terra cotta trimmings.

Plates A to X give the complete structural framing plans, and a part of the architectural floor plans and elevations, which are sufficient for this problem; but additional architectural details would be required for making the complete design.

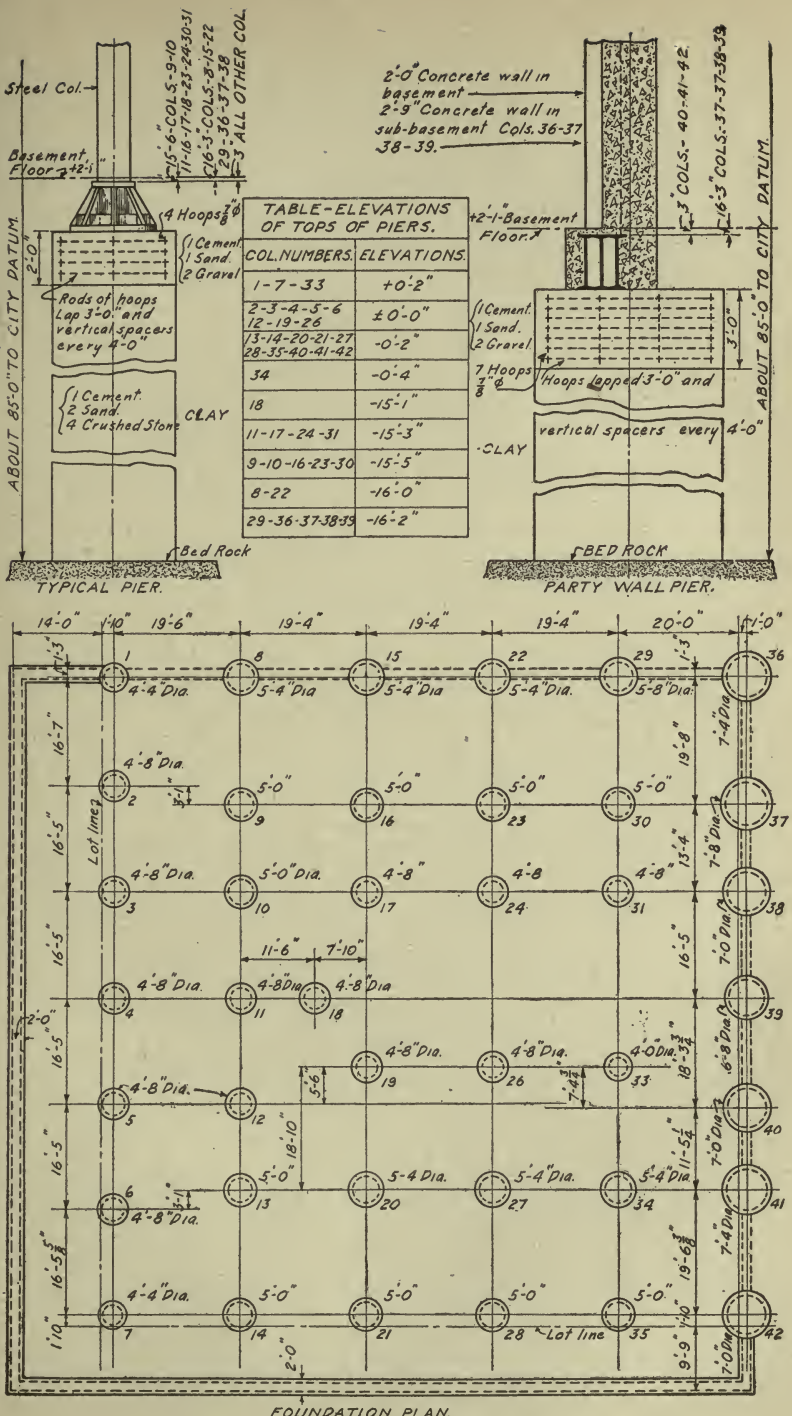


TABLE-ELEVATIONS OF TOPS OF PIERS.

COL. NUMBERS.	ELEVATIONS.
1-7-33	+0'-2"
2-3-4-5-6 12-19-26	±0'-0"
13-14-20-21-27 28-35-40-41-42	-0'-2"
34	-0'-4"
18	-15'-1"
11-17-24-31	-15'-3"
9-10-16-23-30	-15'-5"
8-22	-16'-0"
29-36-37-38-39	-16'-2"

Plate A. Foundation Plan, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

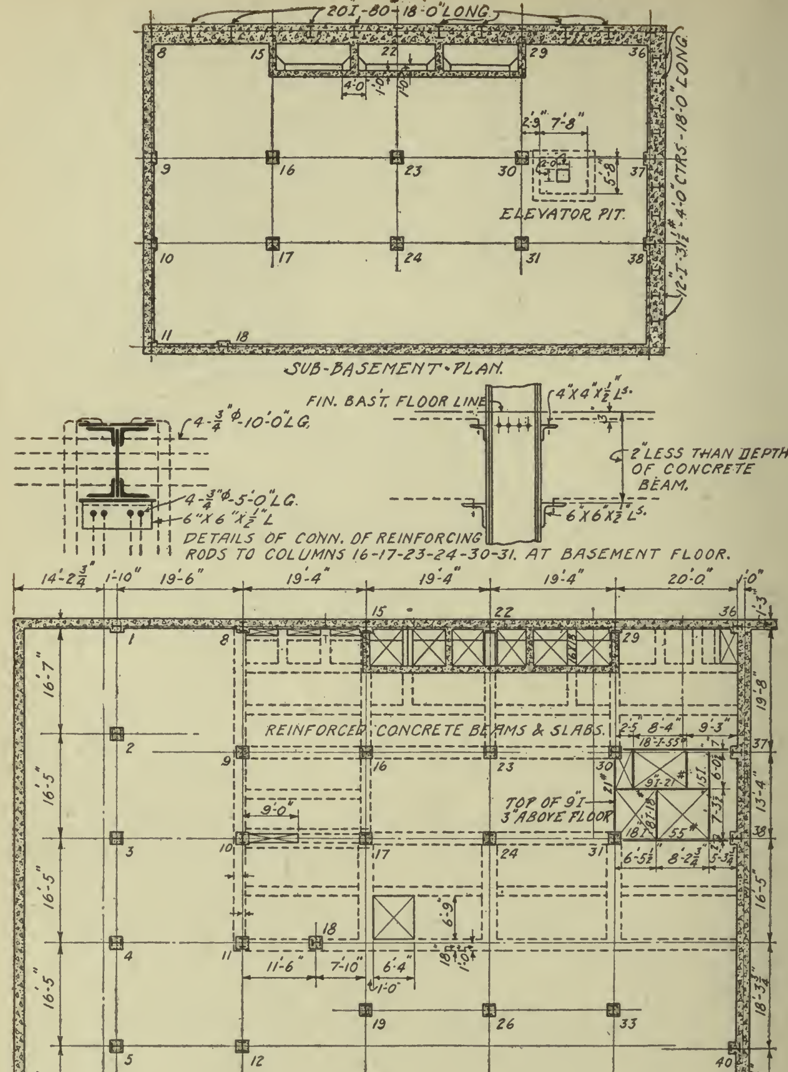


Plate B. Basement and Sub-Basement Plans, Fort Dearborn Hotel
Courtesy, Holabird & Roche, Architects

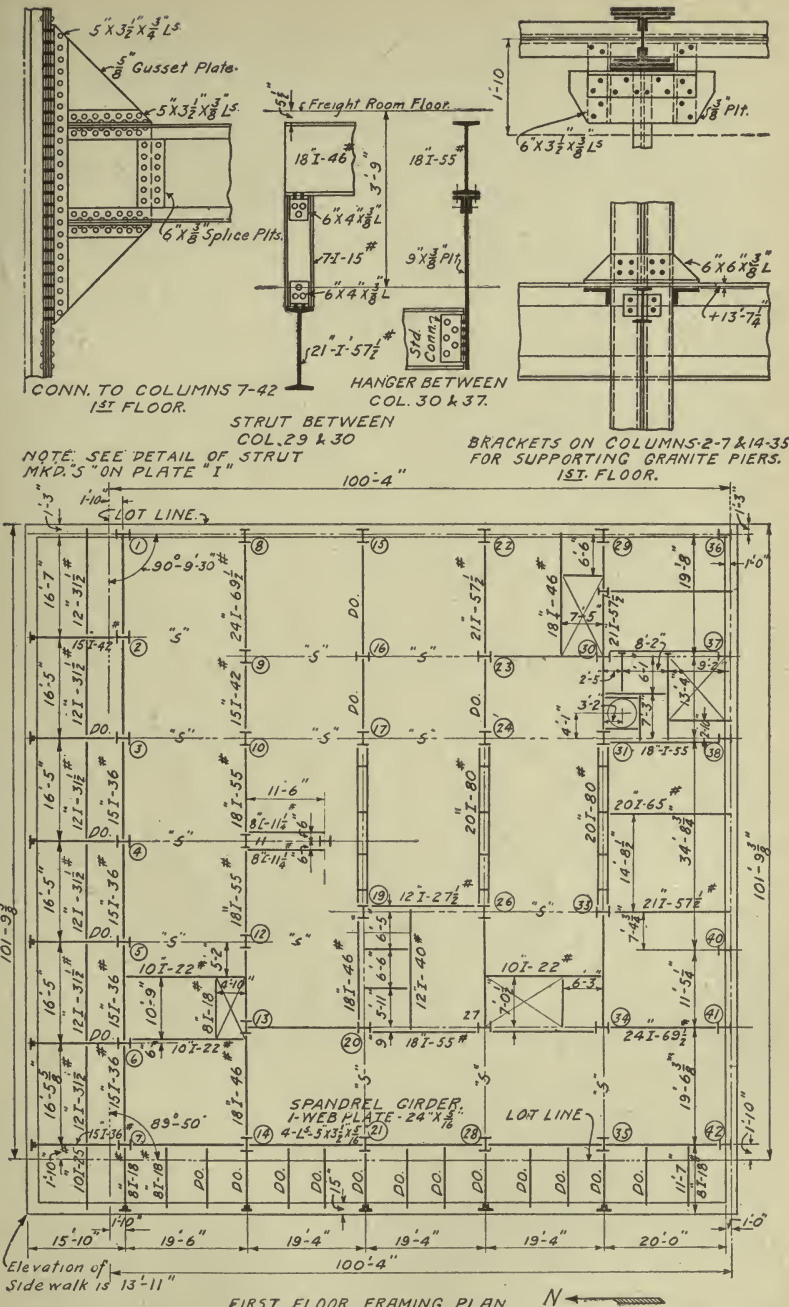
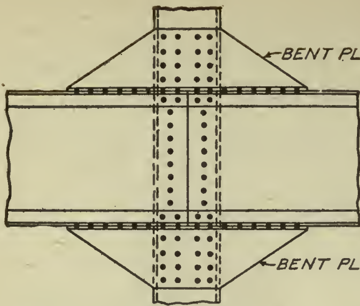
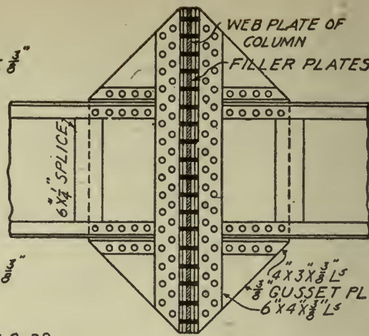


Plate C. First Floor Framing Plan and Details of Beam Connections, Fort Dearborn Hotel

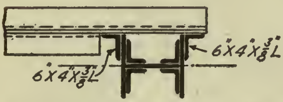
Courtesy, Holabird & Roche, Architects



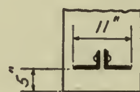
SPANDREL CONNECTIONS TO COLUMNS 8-29
2ND. FLOOR.



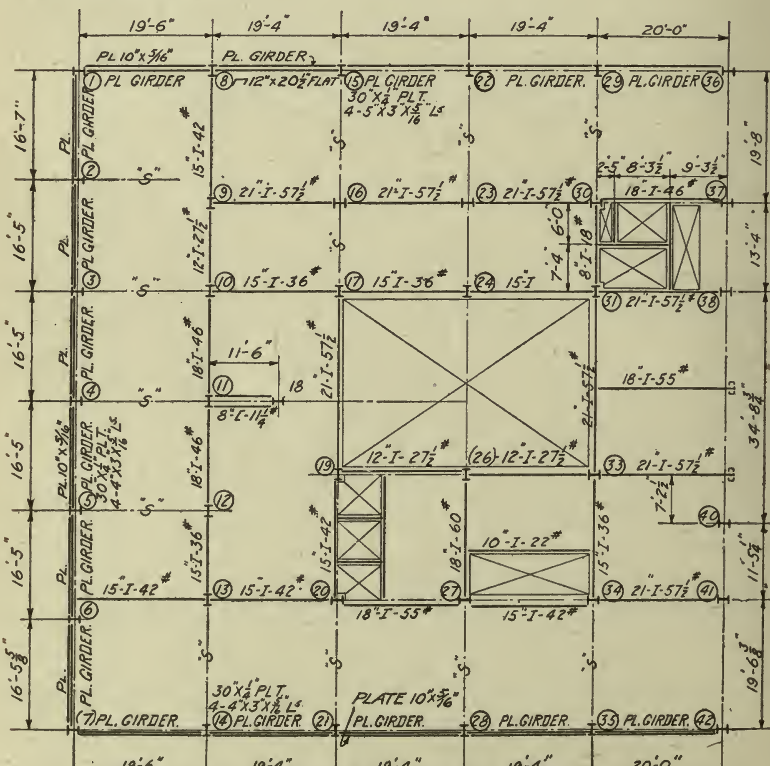
SPANDREL CONNECTIONS TO COLUMNS 14-35
2ND FLOOR.



CONNECTION TO COLUMN 36
2ND FLOOR.



2-5"X3"X3/8"X12'-6" L^s over fresh-air intake. Col. 8-15-1 Reqd.
2-5"X3"X3/8"X4'-0" L^s over door opening. Col 29-36-1 Reqd.



2ND. FLOOR FRAMING PLAN.

Plate D. Second Floor Framing Plan and Spandrel Connections, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

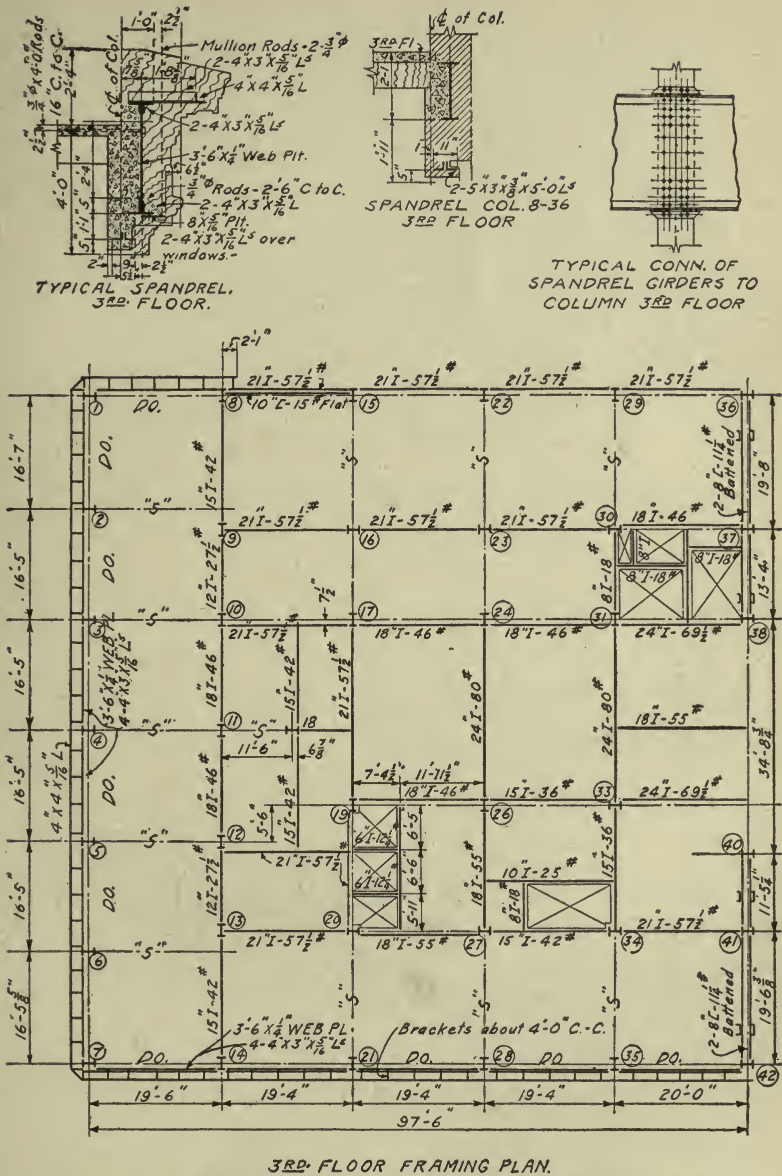


Plate E. Third Floor Framing Plan and Spandrel Sections, Fort Dearborn Hotel
Courtesy, Holabird & Roche, Architects

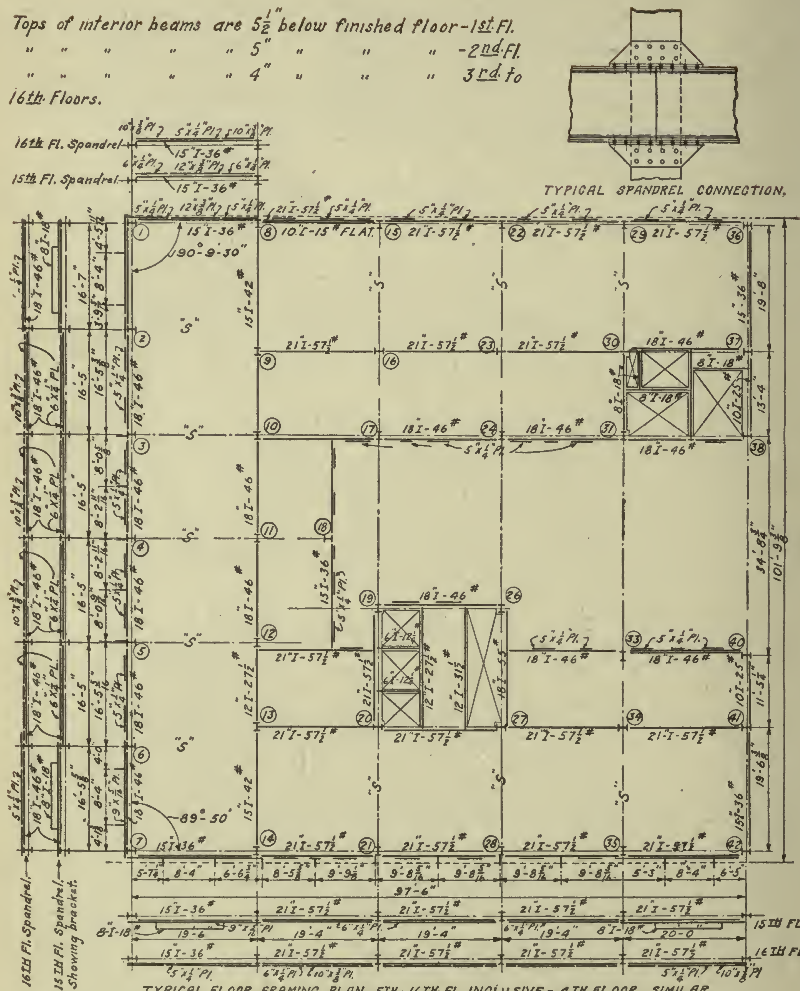


Plate F. Typical Floor Framing Plan, Fifth to Sixteenth Floors Inclusive, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

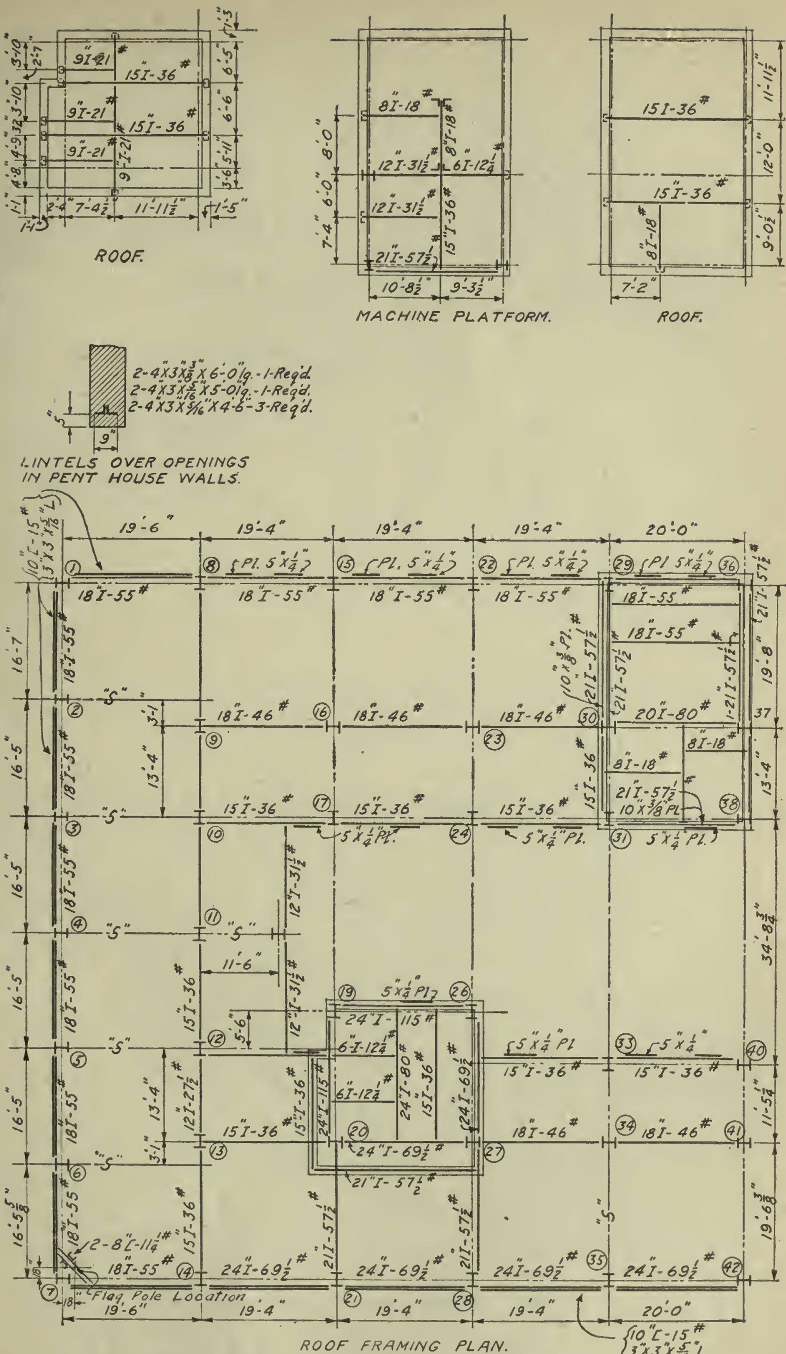
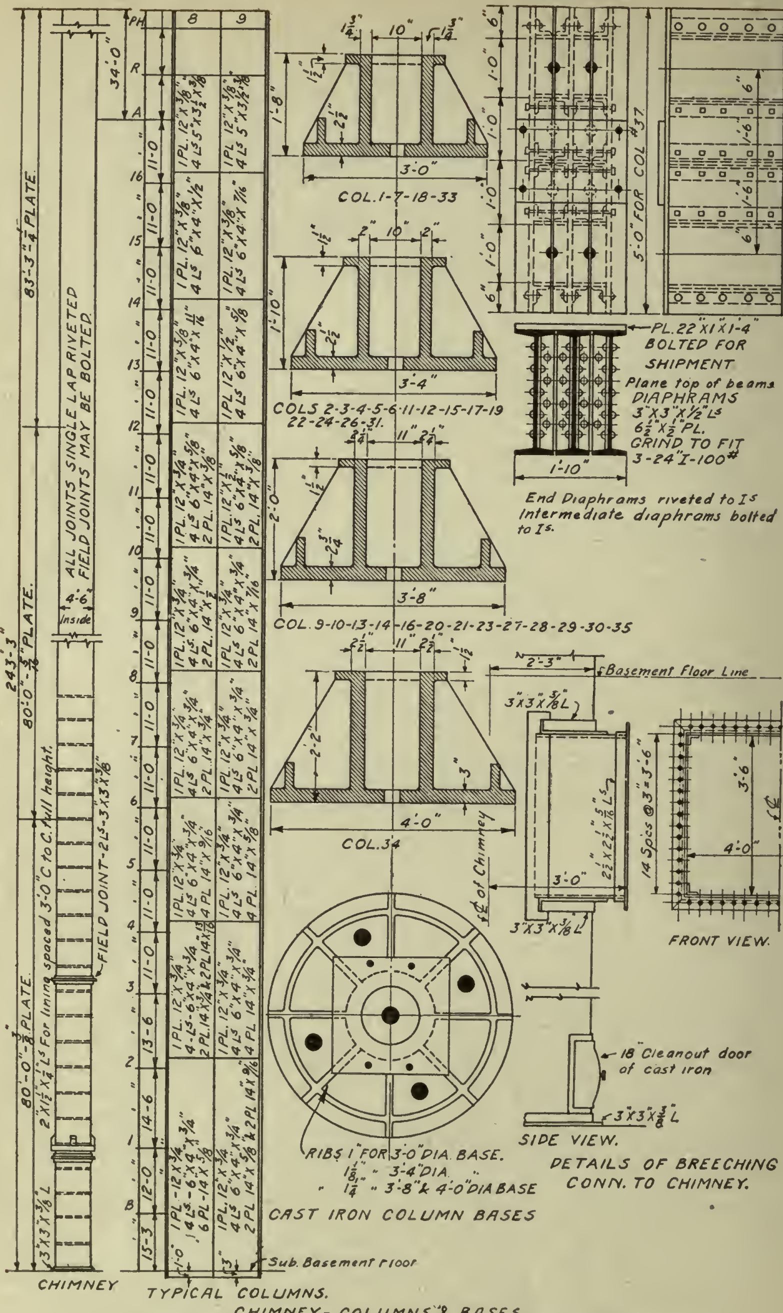


Plate G. Roof Framing Plan, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects



CHIMNEY - COLUMNS & BASES.

Plate H. Details of Chimney, Columns, and Bases, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

278

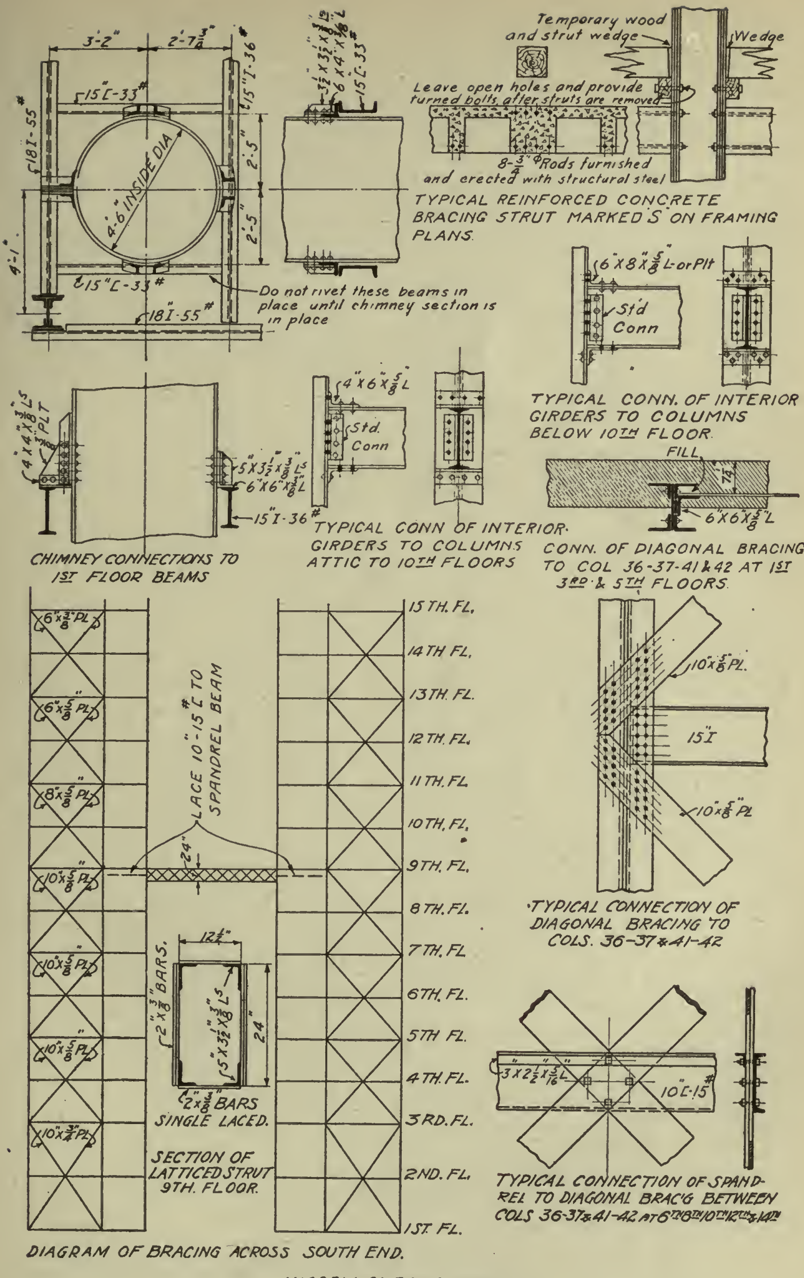


Plate I. Miscellaneous Details, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

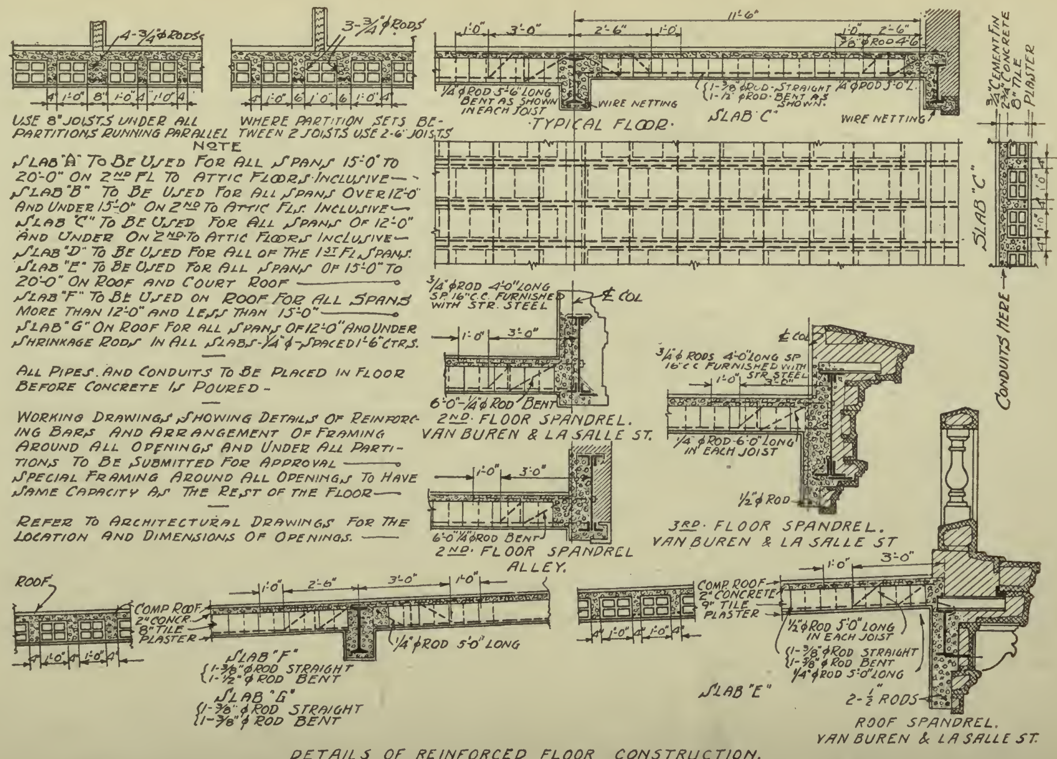


Plate J. Floor Sections Showing Details of Reinforced Concrete Floor Construction, Fort Dearborn Hotel
Courtesy, Holabird & Roche, Architects

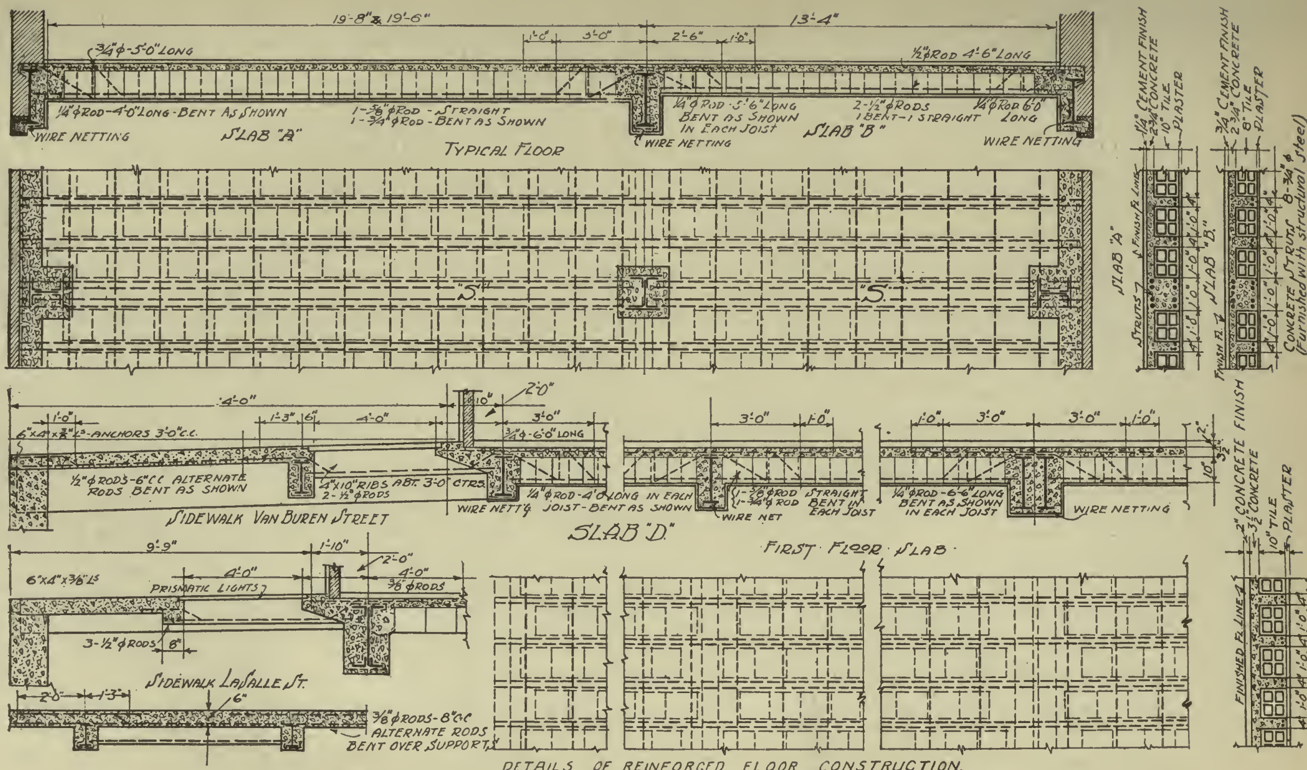
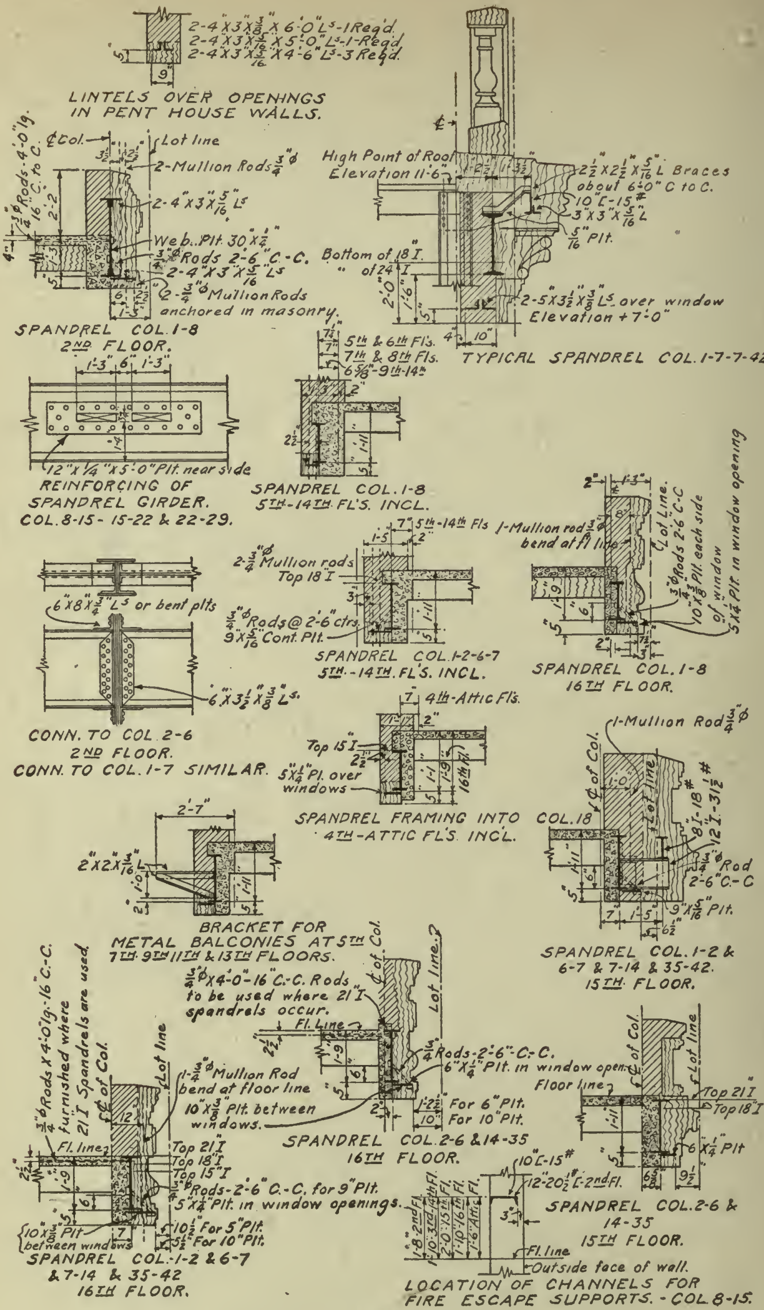


Plate K. Floor Sections Showing Details of Reinforced Concrete Floor Construction, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

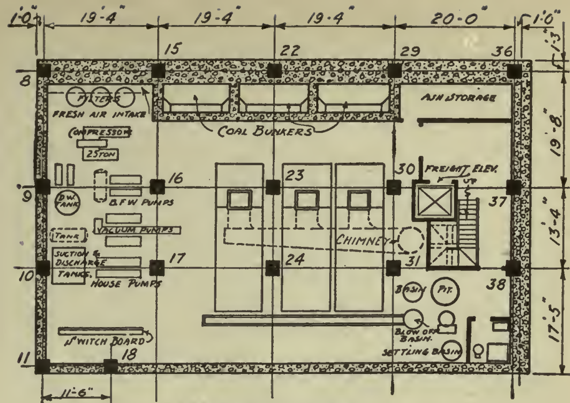


TYPICAL SPANDREL DETAILS.

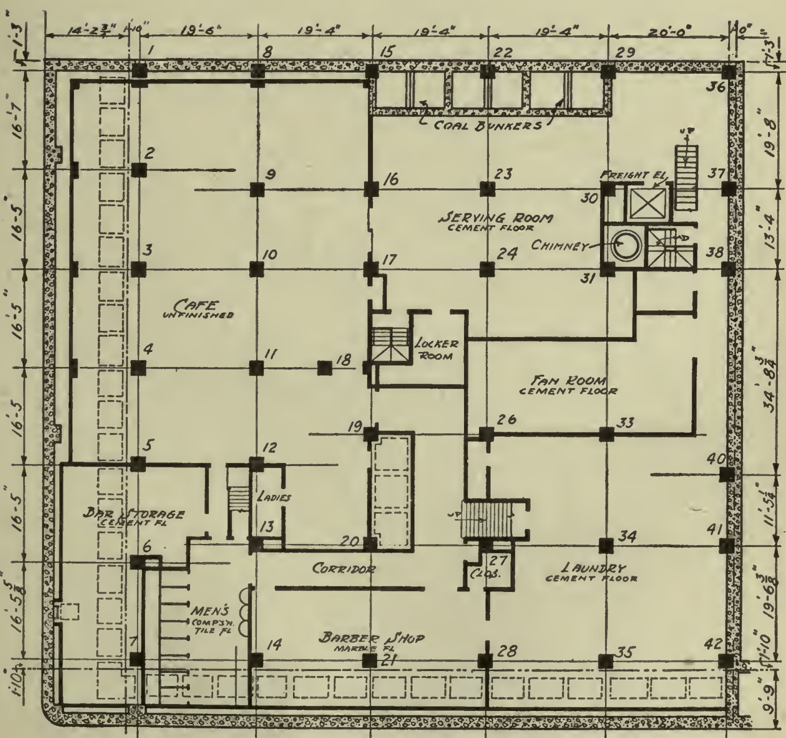
Plate L. Typical Spandrel Details, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

282



SUB BASEMENT PLAN



BASEMENT PLAN

Plate M. Basement and Sub-Basement Plans, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

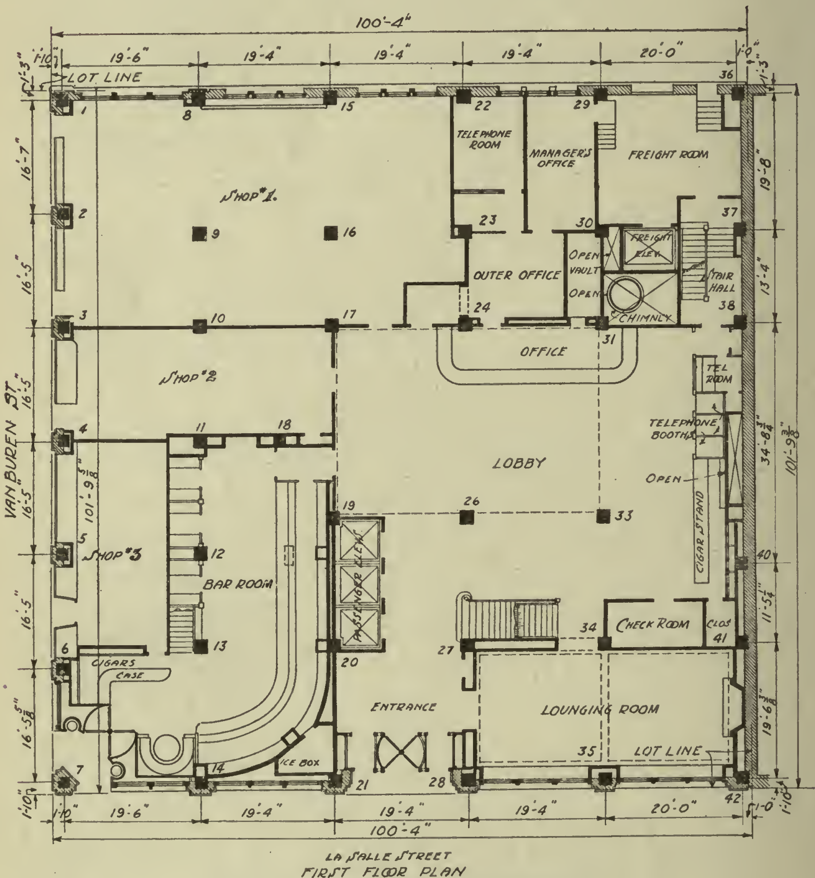


Plate N. First Floor Plan, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

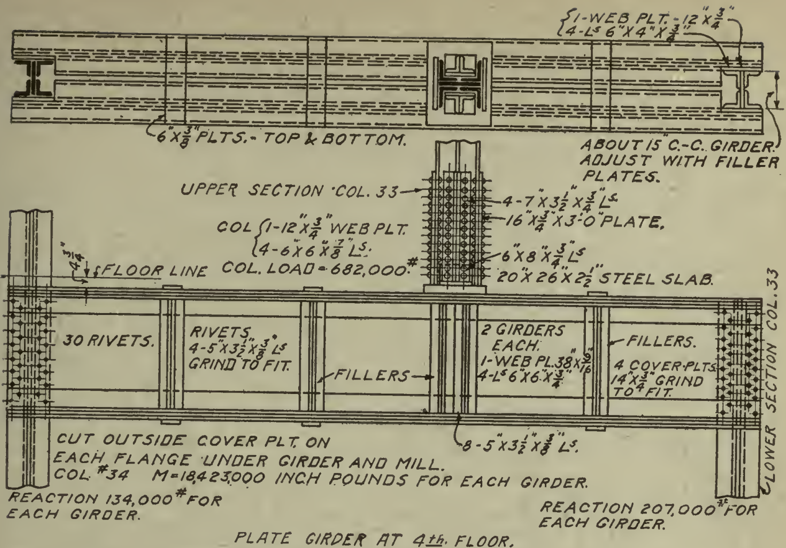


Plate O. Details of Special Plate Girder at Fourth Floor Showing Offset of Column 33, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

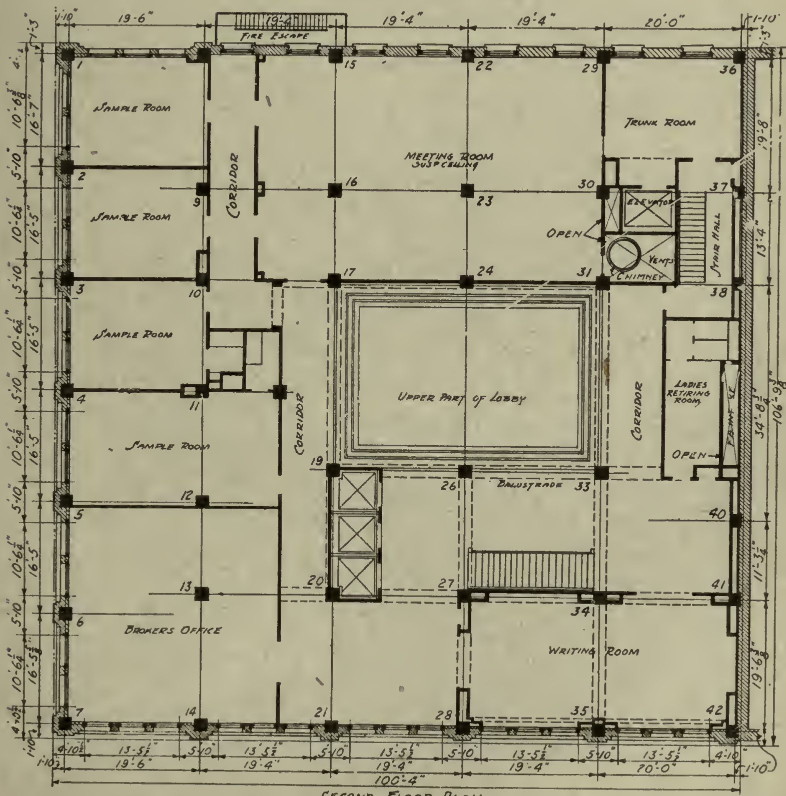


Plate P. Second Floor Plan, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

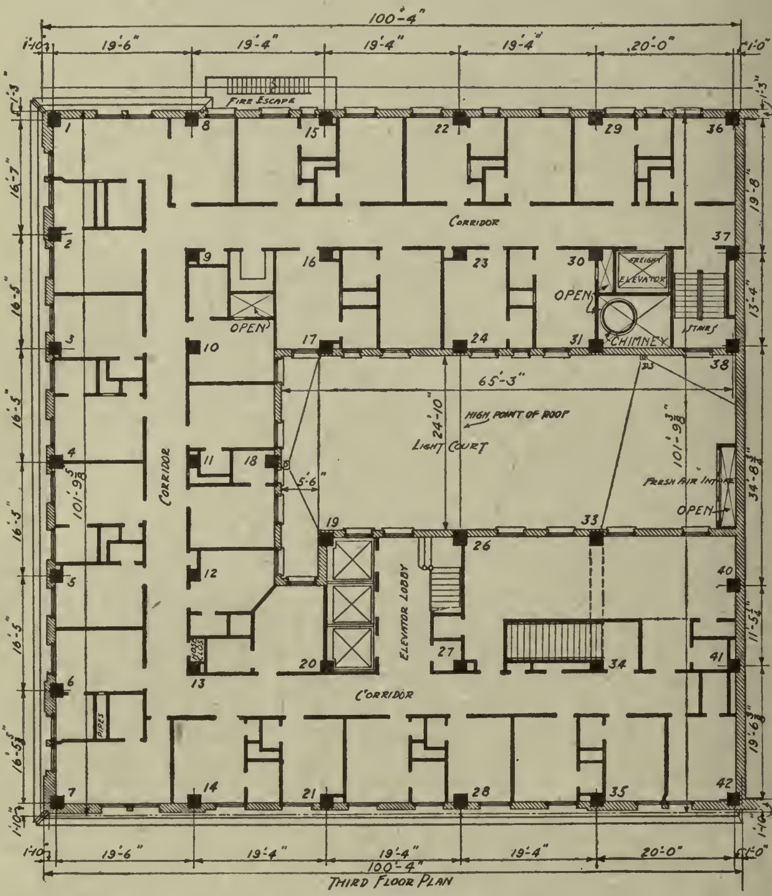


Plate Q. Third Floor Plan, Fort Dearborn Hotel
Courtesy, Holabird & Roche, Architects

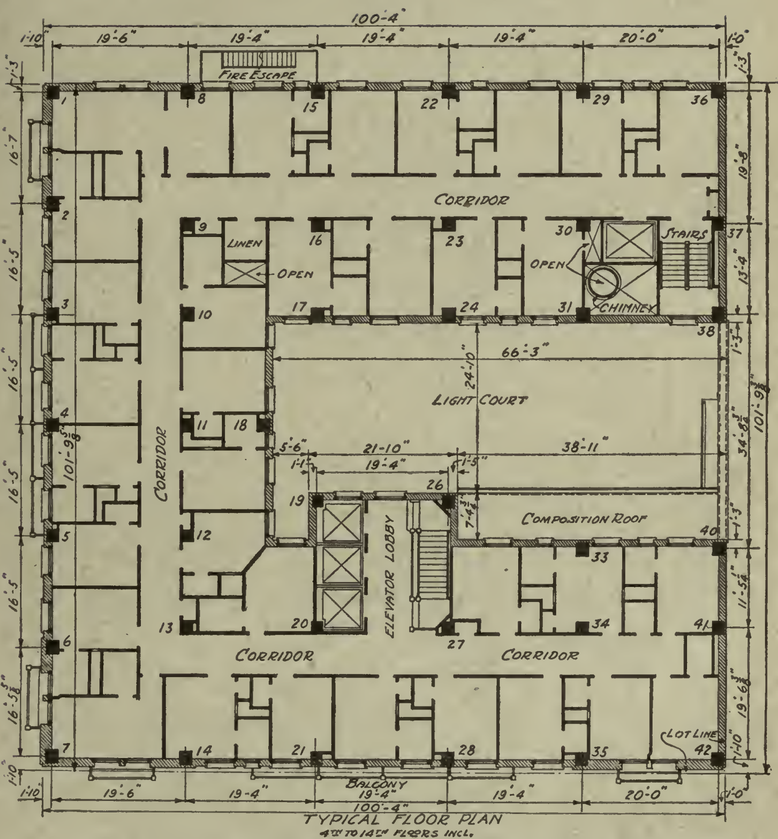
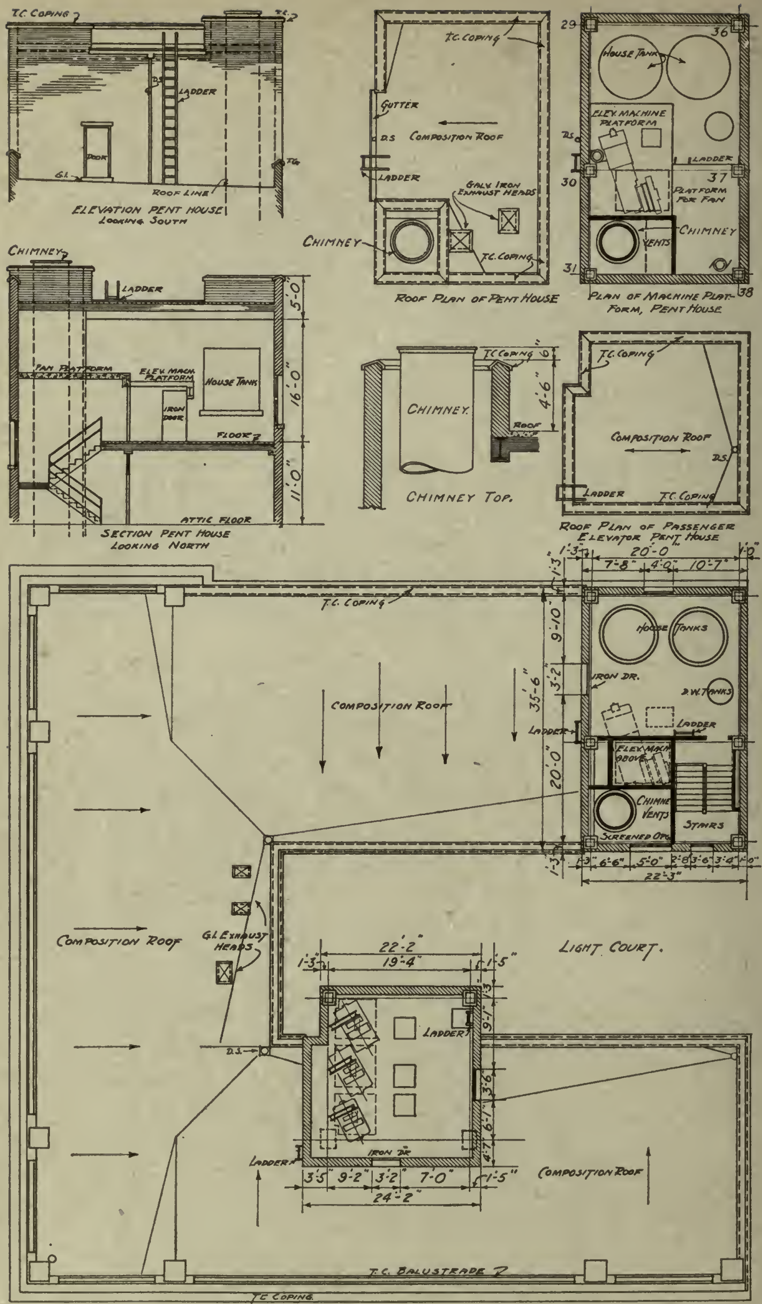


Plate R. Typical Floor Plan, Fifth to Fourteenth Floors, Fort Dearborn Hotel
Courtesy, Holabird & Roche, Architects

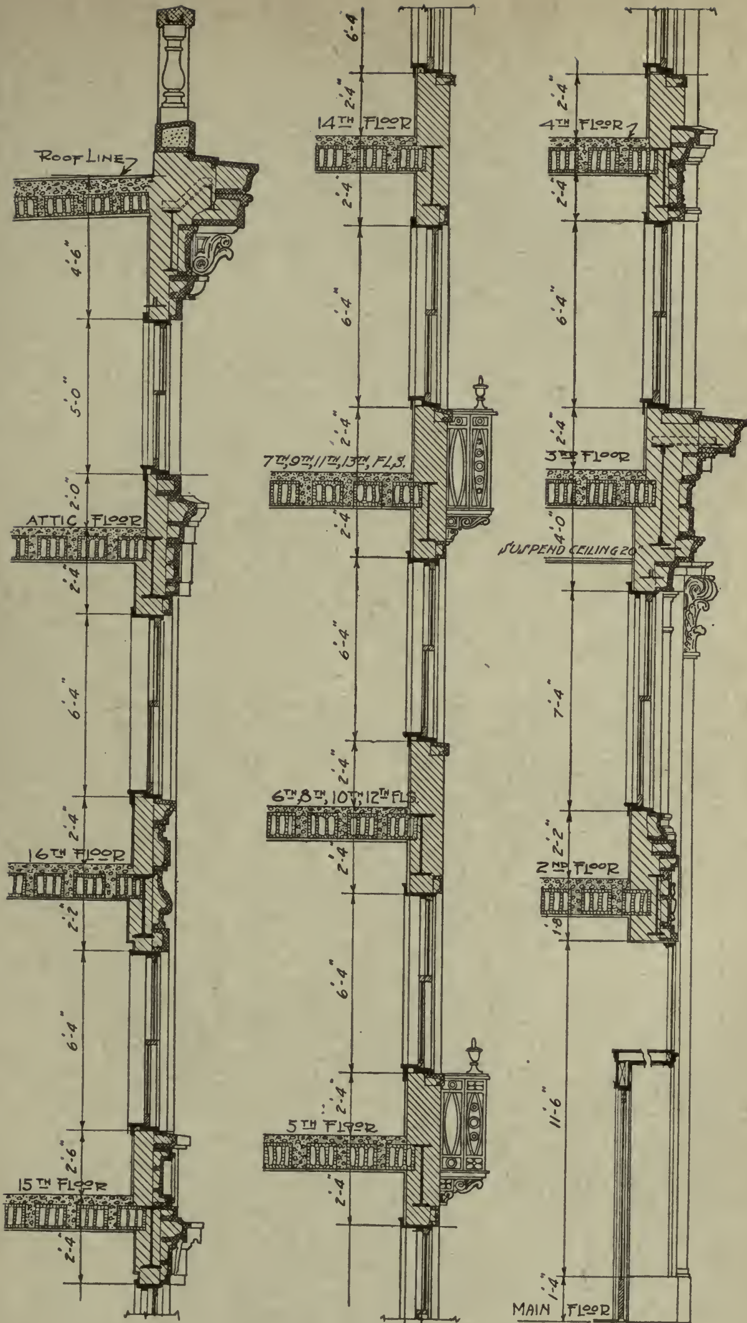


·ROOF·PLAN·

Plate S. Roof Plan, Including Vertical and Horizonta Sections of Penthouse, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architec

288



TYPICAL SPANDRELS.

Plate T. Sections of Typical Spandrels, Fort Dearborn Hotel
Courtesy, Holabird & Roche, Architects
289

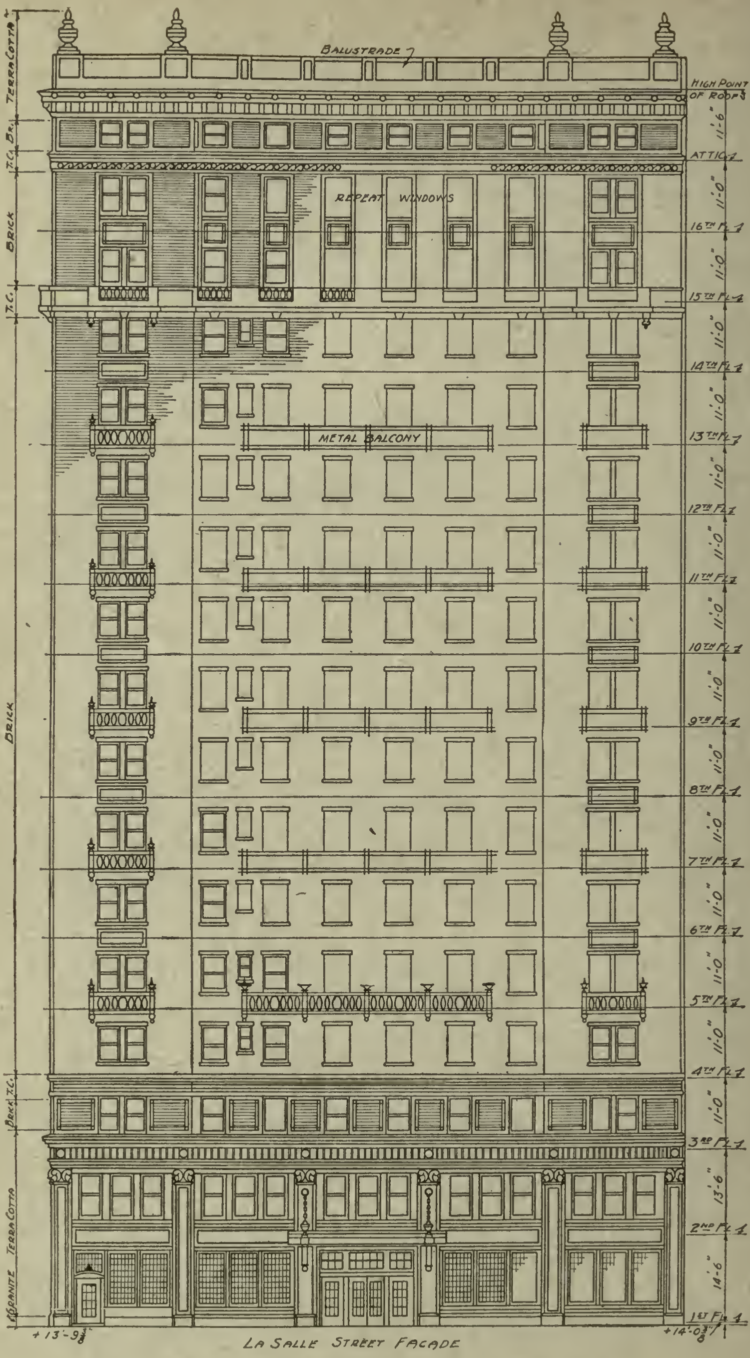
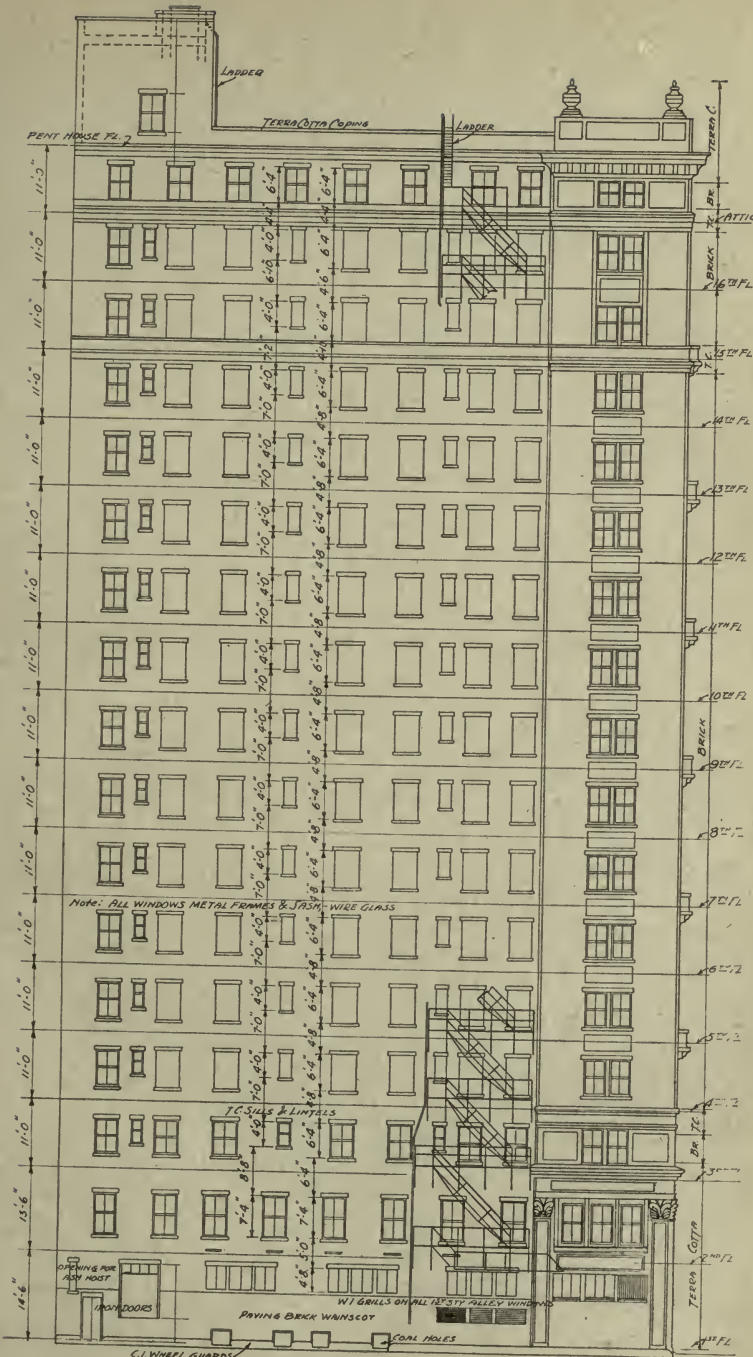


Plate U. LaSalle Street Front Elevation, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

290



ALLEY ELEVATION

Plate V. Alley Elevation, Fort Dearborn Hotel.

Courtesy, Holabird & Roche, Architects

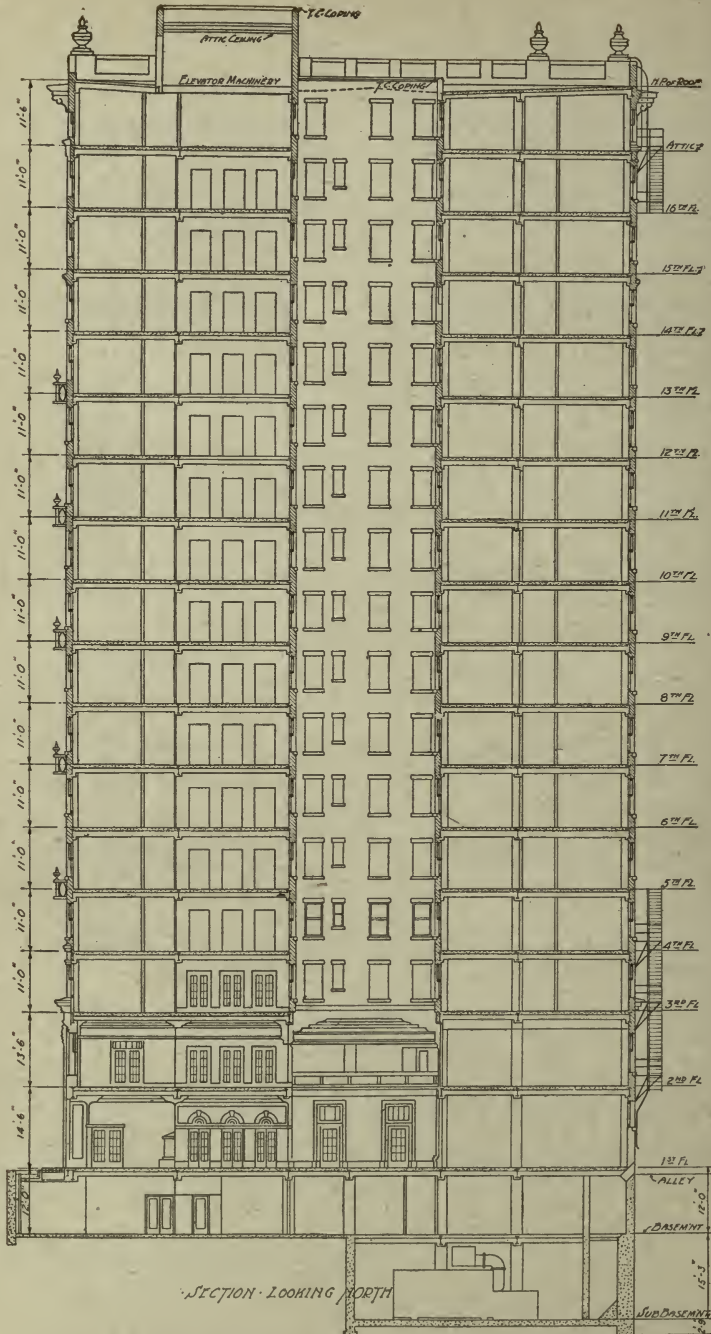
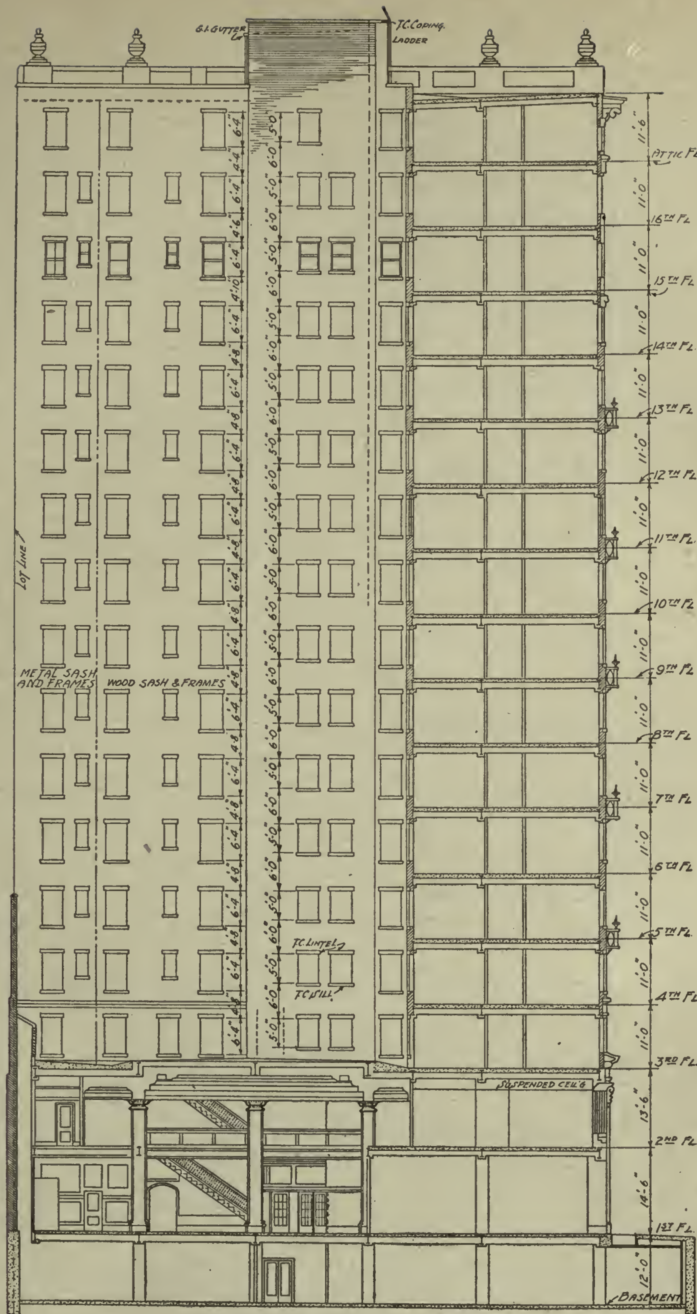


Plate W. Section of Building Looking North, Fort Dearborn Hotel
Courtesy, Holabird & Roche, Architects



SECTION LOOKING WEST.

Plate X. Section of Building Looking West, Fort Dearborn Hotel

Courtesy, Holabird & Roche, Architects

FIREPROOFING

Choice of Concrete. The general subject of fireproofing is discussed elsewhere in this book. For this building concrete is used for fireproofing the steel. It is selected because it protects the steel from corrosion, adds to the strength of the columns, and can be placed easily, in connection with the concrete used in the floor construction.

Thickness Required. The fireproofing affects the steel design through the weight of the material to be supported, and through the locations of steel members in relation to the openings, as allowance must be made for the thickness of fireproofing. The thicknesses required are*

For exterior columns	4″
For interior columns	3″
On the bottom and sides of beams	2″
On the outside of spandrels	4″
Beyond the edge of shelf angles and plates supporting outside brickwork	2″

For the last two items, the brick covering is the fireproofing, but for the columns the brick covering is not counted as fireproofing.

Effect on Position of Exterior Columns, Etc. The requirements for thickness of fireproofing control the position of exterior columns, spandrel beams, and beams around openings in floors. For example, assuming that the steel columns will be 14 inches square, the smallest distance that can be used from face of building to center of columns is made up of

One course of brick	4″
Concrete fireproofing	4″
One-half of column width	7″
Total	15″

This value is adopted for the columns along the alley and court walls, but along the street fronts a greater distance must be had to suit the architectural designs, 1 foot 10 inches being used. The columns should be placed as close to the outside of the building as possible, to keep the eccentricity small and also to make the projection of the columns into the rooms as small as practicable.

In general, the spandrel girders are placed as near the outer face of the wall as the fireproofing requirements will permit, that is,

*To comply with the Chicago Building Ordinance.

with the edge of the flange 2 inches from the face of the wall. In order to provide support farther out, shelf angles or plates are used, projecting no nearer to the face of the wall than 2 inches. The outer 2 inches of the flange angles of a girder may be considered as shelf angles, if the area of this portion of the angles is not required for the girder section; and in such a case the girder is placed 2 inches nearer the face of the building than otherwise would be done.

Fireproofing Around Openings. Around openings, the specifications require 2 inches for fireproofing and usually 1 inch is needed for plaster, stair facia, or other finish. To these must be added the half width of beam to get the distance from finished edge of opening to center of beam. The actual amount required varies for different sizes of beams. It is usually convenient to use the next larger whole number of inches. In most cases 6 inches will suffice for the distance from center of beam to finished opening.

LOADS

Classification of Loads. The structural frame of the building must support the weight of all materials of construction, called the "dead loads"; and the loads of all kinds that may be imposed on the finished structure, called the "live loads". Dead loads are, in all cases, gravity loads, that is, they act vertically. Live loads are gravity loads in most cases. (Belt-driven machinery may cause loads in lateral directions.) In addition to the gravity loads, the framework must resist wind pressure.

A design cannot be more accurate than the loads upon which it is based. It is, therefore, of first importance that the loads used be as accurate as practicable.

Dead Loads. The so-called dead loads, that is, fixed or immovable loads, consist of the weight of all the materials of construction. The quantities must be estimated from the architectural plans and the structural plans as they develop.

Unit Weights. The unit weights of some materials will vary according to locality and the weights of some will vary because of a difference in quality. The following values may be used as averages for ordinary conditions. Weights which are likely to vary with quality, location, or any other cause should be verified or corrected by the designer.

WEIGHTS OF MATERIALS OF CONSTRUCTION

White pine, spruce, hemlock, per ft., board measure	3 lb.
Yellow pine, fir, per ft., board measure	4 lb.
Oaks, maple, per ft., board measure	5 lb.
Brick masonry, pressed or paving, per cu. ft.	140 lb.
Brick masonry, hard common, per cu. ft.	120 lb.
Brick masonry, hollow, per cu. ft.	90 lb.
Sandstone or limestone rubble, per cu. ft.	140 lb.
Sandstone or limestone cut facing, per cu. ft.	150 lb.
Granite, per cu. ft.	160 lb.
Stone concrete, per cu. ft.	144 lb.
Cinder concrete, per cu. ft.	96 lb.
Cinder fill (without sand and cement) per cu. ft.	72 lb.
Mortar and plaster, per cu. ft.	120 lb.
Ornamental terra cotta, backed and filled with common brick, per cu. ft.	120 lb.
Marble, per cu. ft.	175 lb.
Floors, marble, tutti colori, and similar, per sq. ft.	12 lb.
Windows (glass, frames, and sash), per sq. ft.	5 lb.
Roofing, composition, per sq. ft.	5 lb.
Roofing, gravel, per sq. ft.	10 lb.
Roofing, slate, per sq. ft.	10 lb.
Roofing, tile, per sq. ft.	10 lb.
Roofing, shingle, per sq. ft.	3 lb.
Sheet metal roofing, cornice, etc, per sq. ft.	3 lb.
Partition tile, 3 in. thick, per sq. ft.	14 lb.
Partition tile, 4 in. thick, per sq. ft.	15 lb.
Partition tile, 6 in. thick, per sq. ft.	22 lb.
Partition tile, 8 in. thick, per sq. ft.	28 lb.
Partition tile, 10 in. thick, per sq. ft.	32 lb.
Floor flat arch (average of set) 8 in. thick, per sq. ft.	28 lb.
Floor flat arch (average of set) 10 in. thick, per sq. ft.	32 lb.
Floor flat arch (average of set) 12 in. thick, per sq. ft.	36 lb.
Floor flat arch (average of set) 14 in. thick, per sq. ft.	40 lb.
Floor flat arch (average of set) 16 in. thick, per sq. ft.	46 lb.
Floor segmental arch tile (average per set) 6 in. thick at crown, per sq. ft.	28 lb.

Mortar for tile arch floors, per sq. ft.	3 lb.
Book tile 2 in. thick, per sq. ft.	12 lb.
Book tile, 3 in. thick, per sq. ft.	14 lb.
Beam tile (when not included with arch tile), per sq. ft.	12 lb.
Gypsum partition blocks, 3 in. thick, per sq. ft.	10 lb.
Gypsum partition blocks, 4 in. thick, per sq. ft.	12 lb.
Gypsum partition blocks, 5 in. thick, per sq. ft.	14 lb.
Gypsum partition blocks, 6 in. thick, per sq. ft.	16 lb.
Plaster on brick, concrete, tile, or gypsum, per sq. ft.	5 lb.
Plaster on lath, per sq. ft.	7 lb.
Suspended ceiling complete, per sq. ft.	10 lb.
Steel bar 1 in. square, 1 ft. long, per lineal ft.	3.4 lb.
Steel plate 1 ft. square, 1 in. thick, per sq. ft.	40.8 lb.
Cast iron, bar 1 in. square, 1 ft. long, per lineal ft.	3.125 lb.
Cast iron, per cu. in.	.26 lb.

The following items may vary considerably in weight but the values given may be used for preliminary computations, or when the quantities are small:

Iron stair construction, per sq. ft.	50 lb.
Concrete stair construction, per sq. ft.	150 lb.
Wood stair construction, per sq. ft.	20 lb.
Sidewalk lights in concrete, per sq. ft.	30 lb.
Reinforcment of concrete, per cu. ft.	6 lb.
Total weight of reinforced concrete, per cu. ft.	150 lb.
Steel joists, per sq. ft. of floor	6 lb.
Steel girders, per sq. ft. of floor	4 lb.
Partition, tile plastered, per sq. ft.	25 lb.
Same in hotels, per sq. ft. of floor	35 lb.
Same in office buildings, per sq. ft. of floor	25 lb.

Live Loads. Live loads are the temporary or movable loads in a building. They include furniture, merchandise, and people. The amount of live load depends on the purpose for which the building is used, and for a given purpose may vary greatly from time to time and from one part of the building to another. The amount to be used is a matter of judgment, unless an arbitrary weight is established by law. In most cities the building ordinances fix the minimum live loads for various buildings according to their use. The requirements of the Revised Building Ordinances of the City of Chicago, adopted December 8, 1910, are as follows:

Stores, light manufacturing, stables, and garages	100 lb.
Office buildings, hotels, and hospitals	50 lb.
Dwellings, small stables, and private garages	40 lb.
Churches and halls	100 lb.
Theaters	100 lb.
Apartment houses	40 lb.
Department stores	100 lb.
Schools	75 lb.
Roofs	25 lb.

These loads are to be applied per square foot to the actual floor area of the building.

In designing the floor slabs and joists, the full amount of the live load is used. For girders, the live load may be reduced 15 per cent. For columns, the load for the top floor is reduced 15 per cent and for each successive floor downward the reduction is increased 5 per cent till 50 per cent is reached; this final value is used for the remaining floors. This method of reducing the loads on columns is allowed in Chicago. Other similar methods are used in other cities. The designer must use his judgment as to the propriety of making the reductions.

Special Loads. In addition to the live load, which is assumed to be uniformly distributed over the floor, there may be special loads, such as elevators, machinery, water in tanks, coal in bins, space for storage of special materials, etc. The weight of water is 62.5 pounds per cubic feet, or $8\frac{1}{3}$ pounds per gallon; of bituminous coal, 50 pounds per cubic feet; of anthracite coal, 60 pounds per cubic feet.

The weights of elevators are usually given by the manufacturer for the particular situation. An impact allowance of 100 per cent is applied to these weights in designing the beams and their connections to the columns, but only the actual weights need be allowed on the columns.

Loads on the Building Illustrated. In the Fort Dearborn Hotel the following live loads are used:

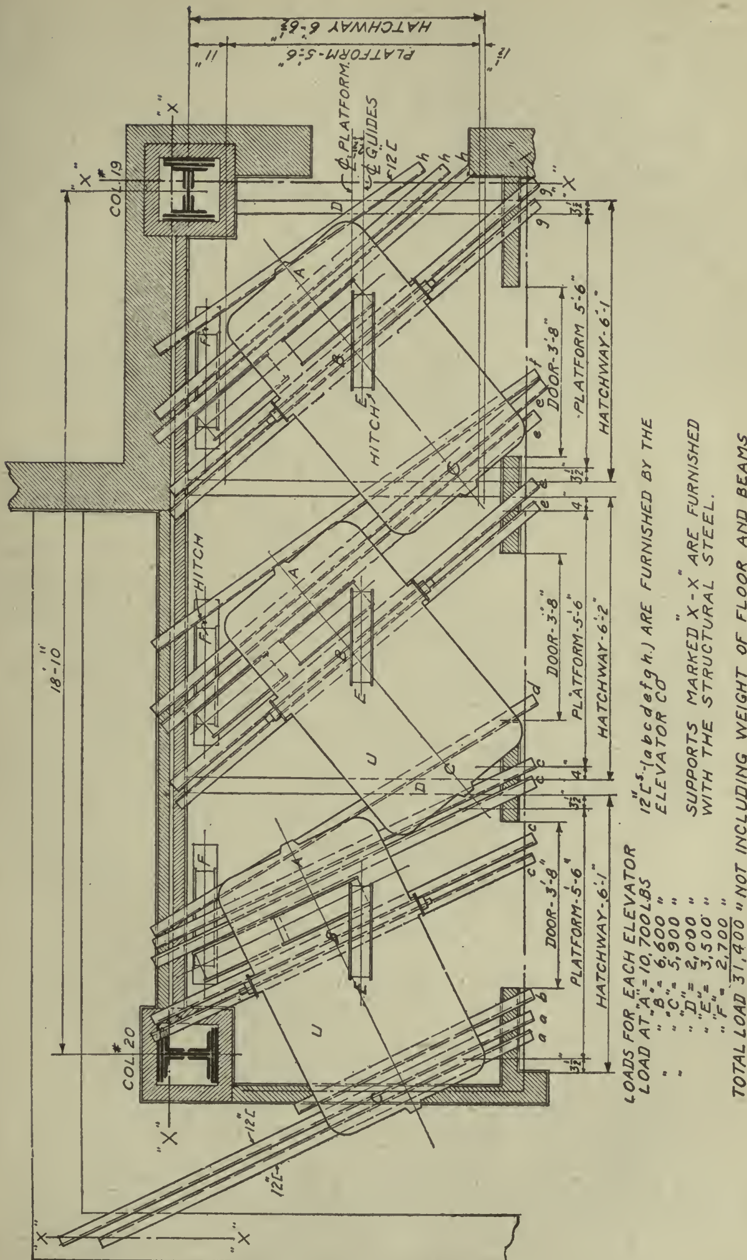


Fig. 197. Details of Passenger Elevator Machines and Supports

For the roof, per sq. ft.	25 lb.
For 2nd to 16th floor, per sq. ft.	50 lb.
For 1st floor, per sq. ft.	100 lb.
For sidewalks, per sq. ft.	150 lb.
For freight receiving room, per sq. ft.	150 lb.
For stairs, per sq. ft.	100 lb.

The special loads are the elevator loads as indicated in Figs.

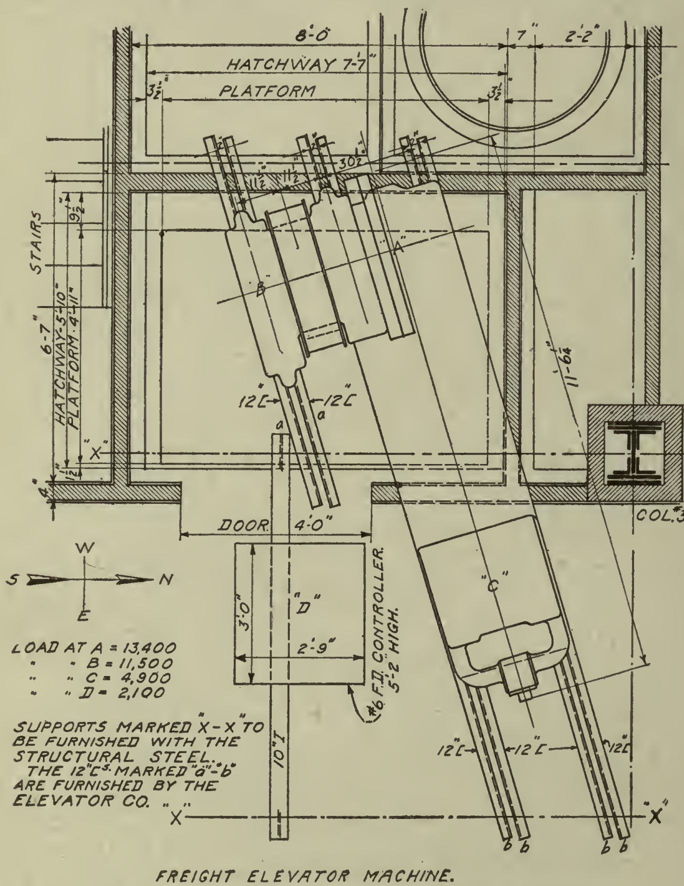


Fig. 198. Details of Freight Elevator Machine and Supports

197 and 198 and water-tank loads shown on the plans of the penthouse, Plate S.

The dead loads are computed in connection with the various members supporting them, from the unit values previously given.

The wind load is taken at 20 pounds per square foot of the exposed area of the building.

TYPE OF FLOOR CONSTRUCTION

Two types of floor construction are suitable for this building; the flat tile arch between steel I-beam joists, Fig. 199, and a

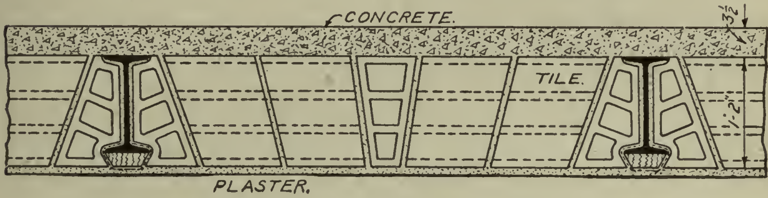


Fig. 199. Section of Flat Tile Arch Floor

combination tile and reinforced concrete spanning from girder to girder, Fig. 200, and Plates J and K. Other types might be considered but have been rejected as not being suitable for the particular requirements of this building. It is evident at once that the type using joists requires more steel than the other, but in order to make a complete comparison of costs it is necessary to make preliminary designs of the steel required for typical panels for each type.

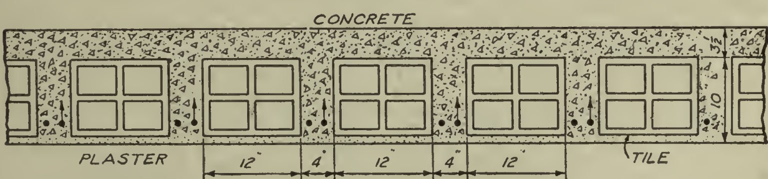


Fig. 200. Section of Reinforced Concrete and Tile Floor

Tile Arch Floor. Considering first the flat tile arch, the loads per square foot of floor on joists are

Tile arch set in place 14 in. deep	43 lb.
Concrete $3\frac{1}{2}$ in. deep	42 lb.
Steel joists	6 lb.
Plaster	5 lb.
Partitions	35 lb.
Total dead load	131 lb.
Live load	50 lb.
Total load on joists	181 lb.

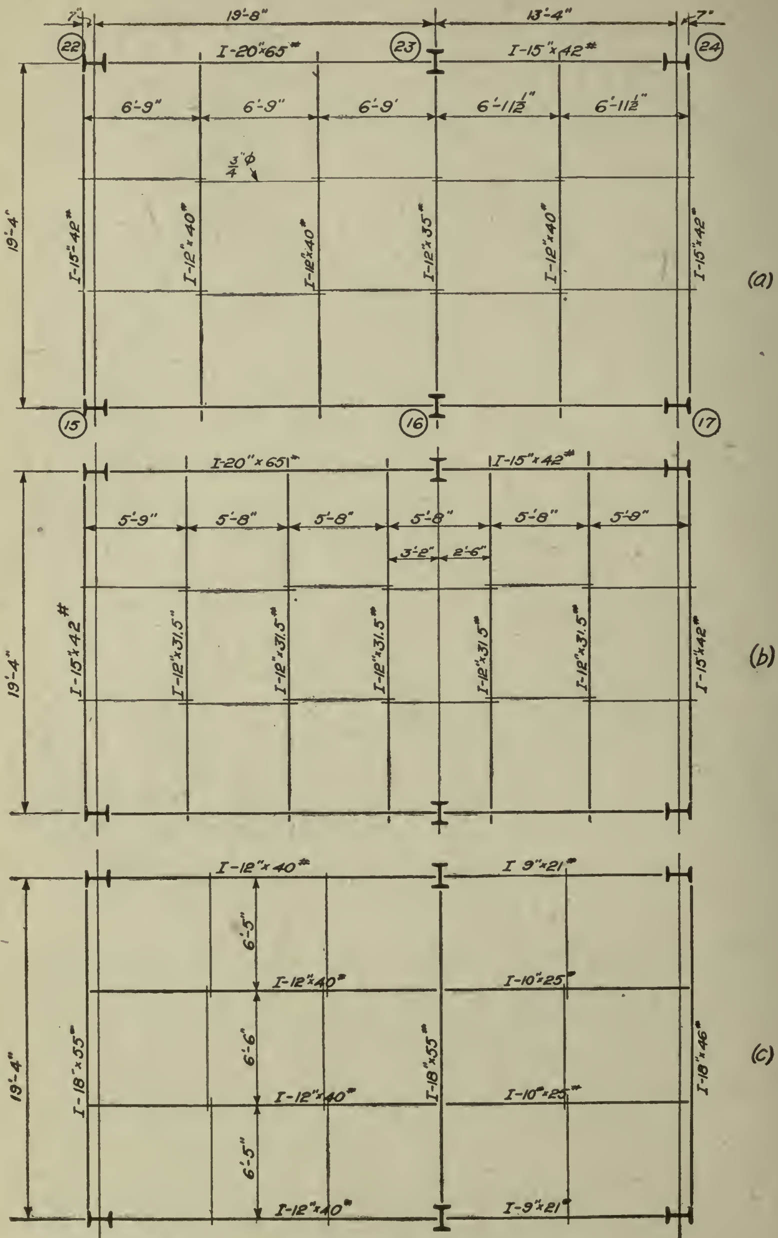


Fig. 201. Diagram Showing Framing for Tile Arch Floor

The loads per square foot of floor, as applied to the girder are

Total dead load of floor as above	131 lb.
Steel girder	4 lb.
Fireproofing on girder	2 lb.
Total dead load	137 lb.
Live load 85% of 50 lb.	43 lb.
Total load on girders	180 lb.

Therefore, 180 pounds per square feet may be used for both joists and girders.

The allowance for partitions is determined by computing the total quantity and weight on one floor and dividing by the number of square feet of floor area.

The depth of the joists is assumed for trial to be 12 inches. The joists may be spaced as far apart as 8 feet, but a closer spacing is preferred. They may be arranged in the three ways shown in Fig. 201.

The beams *15-22* and *17-24* support the wall load as well as the floor load. The amount of the wall load is calculated as follows:

Gross wall area 11′-0″×19′-4″	212 sq. ft.
Less windows 2×6′-4″×4′-0″	51 sq. ft.
Net wall area	161 sq. ft.

Weight of material composing wall is

4 -inch pressed brick weighing 140 lb., per cu. ft.	47 lb.
4-inch common brick weighing 120 lb., per cu. ft.	40 lb.
$4\frac{1}{2}$-inch hollow brick weighing 90 lb., per cu. ft.	34 lb.
Total weight per sq. ft. of wall area	121 lb.

Using even figures, the weight of wall on the spandrel beam is

$$160 \times 120 = 19{,}200\#$$

Scheme a. In scheme *a*, Fig. 201, the sizes of beams required to support the loads computed above are as marked on the diagram. The lengths used in computing are the actual lengths of the beams, that is, allowance is made for the width of column. Thus the joist between columns *16* and *23* is taken at 18′-2″ long, and because it is shorter than the other joists it is made lighter.

Scheme b. Scheme *b*, Fig. 201, is similar to scheme *a*, the only difference being in the spacing and, consequently, in the weight of the joists. It has the advantage of using joists all alike and equally spaced. It has the disadvantages of greater weight (slight), greater number of pieces to be handled, and of not providing a direct brace between columns *16-23*.

Scheme c. In scheme *c*, Fig. 201, the direction of the joists differs from that in the other schemes. It has the disadvantages of a greater variety of sizes of joists and of throwing a heavy load on the spandrel girders which have eccentric connections to the columns. Its advantage (which is not apparent from the sketches but is shown on the architectural plans of the building) is that the girders do not cross the corridor which extends along the middle of the building alongside of columns *16-23*.

The weights of the steel in the three schemes differ so little that this feature would not govern. Scheme *a* seems to be the best one because it has the least number of pieces to handle, braces all columns in both directions, and loads the columns with the least eccentricity.

PROBLEMS

1. Estimate the weights of steel in the panels shown in Fig. 201 for schemes *a*, *b*, and *c*.

2. Check the sizes of I-beams used in schemes *a*, *b*, and *c*.

Combination Tile and Concrete Floor. Now consider the type of floor construction shown in Fig. 200, that is, the combination tile and concrete. There being no steel joists, the weight per square foot as applied to the girders is estimated as follows:

Concrete slab $3\frac{1}{2}$ in.	42 lb.
Concrete joists $4'' \times 10''$, $40\# \times \frac{3}{4}$	30 lb.
Tile $10'' \times 12''$; $32\# \times \frac{3}{4}$	24 lb.
Plaster	5 lb.
Reinforcing steel	3 lb.
Girder steel	4 lb.
Girder fireproofing	10 lb.
Partitions	35 lb.
Total dead load	153 lb.
Live load, 85% of 50 lb.	43 lb.
Total load	196 lb.

On the narrow panels the tile fillers are 8 inches deep, the resulting saving in weight of tile and concrete and concrete joists being 9 pounds. This leaves a total weight of 187 pounds per square foot on these narrow panels.

Two schemes for the arrangement of girders are shown in Fig. 202. In both cases the spandrel beams have the same wall load as computed in connection with the tile arch type of floor, viz, 19,200 pounds. The sizes of beams required are marked on the diagrams. Note that in scheme *a* the lighter load applies in the narrow panel, whereas in scheme *b* the heavier load must be used in both panels.

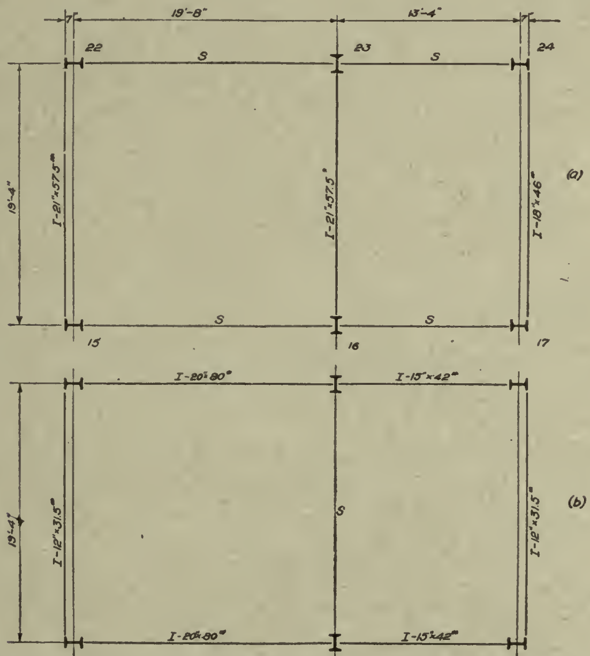


Fig. 202. Diagram Showing Framing for Combination Tile and Concrete Floor

The members marked *S* are struts which support only narrow strips of floor load but are required to brace the columns in the direction in which girders do not occur. For this purpose light I-beams or H-sections are commonly used, but in this case reinforced concrete is used.

Neither scheme has any definite advantage in weight of steel. Scheme *a* is adopted because the arrangement is better suited to the plan of the floor. The girder *16-23* is alongside the corridor and is covered by the partition. No girder crosses the corridor. The use of the larger spandrel beams assists in bracing the building. A definite disadvantage is that the spandrel beams, carrying large loads, have eccentric connections to the columns.

PROBLEM

Check the sizes of beams given in Fig. 202.

Selection of Floor Type. The selection of the type of floor construction is affected by a number of items in addition to the cost of the steel, which cannot be considered in detail here. Some of them are: the effect of difference in weight on the cost of the columns; the effect of the difference in weight on the cost of foundations; the relative cost of the floors; the thickness of the floor construction; and soundproofness. In this particular case the cost of the steel is the most important item.

The combination type is used for this building on account of its economy, all conditions being considered, Plates J and K.

FRAMING SPECIFICATIONS

Arrangement of Girders. Some attention has already been given to the arrangement of the girders in the discussion of typical floor panels, but this arrangement really needs to be considered in its relation to the entire building. Refer to the architectural and the framing plans of the typical floors, Plates R and G.

Exterior. It is necessary of course to have girders around the entire perimeter of the building to support the walls.

Interior. The next thing to settle is whether the interior girders shall be parallel to or perpendicular to the outside lines of the building. The former arrangement is used. It is to be noted that the girders and their covering project several inches below the ceiling line, hence it is important to place them so that they interfere as little as practicable with the interior arrangement. In the plan adopted the principal lines of girders are along the side of the corridors and thus can be partially or wholly concealed. They cross the corridors only at two places.

The arrangement used gives practically a set of duplicate floor panels along the outside walls of the building and another along the court walls. The other plan would be nearly as good in this respect. However, columns *2* and *6* are not opposite the columns in the next row so that if girders perpendicular to the outside lines were used, they would be connected at one end to the columns mentioned but would require cross girders to support the other ends. Having main lines of girders east and west, and also north and south, is advantageous in bracing the building.

Special Cases. On the first floor, Plate C, girders are required between columns *17-19* on account of the length of span. Along the east and south sides no wall girders are required because the basement walls can be used to support the first-story walls, hence along these two sides the girders are placed perpendicular to the side lines. Other interior girders are placed so as to give the greatest possible uniformity in the floor construction.

Around openings, such framing is used as may be needed. No instruction is necessary for this, as the framing required can easily be determined from the conditions in each case.

Each building has its special conditions affecting the placing of the girders. Flat ceilings, permitting no projecting beams, may compel the placing of girders on the short spans and perhaps the use of double girders. The use of reinforced concrete floors with rods in two directions requires girders on all four sides of the panels. Pipe shafts in line with the columns in one direction may require the placing of the girders in the other direction. Columns in rows in one direction, only, limit the girders to those lines.

Arrangement of Joists. Having established girder lines, the joists, if used, are spaced as uniformly as practicable. A joist should connect to each column in order to brace it, and the intervening panels should be divided into a number of equal spaces. Their spacing is governed in most cases by the type of floor construction; for the style of construction adopted no steel joists are required.

Beam Elevations. The elevations of beams are given in reference to the elevations of the floors. The distance from the floor lines to the top of the beams is governed by the floor construction. The items entering into this dimension are: the thickness of flooring, whether of wood, marble, tutti colori, etc.; the mortar bed for setting marble and similar floors; the thickness of the wood nailing strips for wood floors; the space for electrical and other conduits.

The minimum thickness of concrete floors over beams should be 3 inches to allow space for conduits and to prevent cracks. Other floors require from 3 to 6 inches, depending upon conditions.

In flat tile arch construction the total thickness is fixed by the depth of the typical joist. All beams deeper than this will be placed flush on top, and all beams shallower flush on the bottom. Thus, if the typical joist is 12 inches, the girder, which probably is deeper, will be placed flush with the top of the joist and will project below the ceiling line; other joists and framing around openings which may be 8-, 9-, or 10-inch beams will be placed flush with bottom to provide bearing for the skew back of the arch at the proper level.

For combination tile and concrete, and for concrete floors, all the beams will be placed flush on top except such as may require a different elevation to suit some special condition.

Spandrel beams, being embedded in the walls, are not governed by the elevation of the floor. In many cases these beams serve as the lintels over the windows and their elevations are fixed accordingly. This is shown in the spandrel sections, Plates L and T.

For flat roofs, the beams may be set on slopes parallel to the roof surface, or may be set level, depending on whether the roof or the ceiling has the greater control.

Arrangement of Columns. *Location.* It is desirable that the columns be arranged in rows across the building in both directions, but this may be prevented by the arrangement of the rooms in the building. The column spacing is also affected by the design of the exterior; the layout determined by the architectural requirements governs in most cases. Thus in the problem the position of column *18* is fixed by the light court wall; of columns *19* and *26* by the space required for elevators and stairs, Plate R; of column *33* in the lower part of the building, to suit the arrangement of rooms in the first story, Plate N, it being offset at the fourth floor, Plates Q and R, on account of the light court wall. The spacing of the columns along the west façade conforms to the architectural treatment, an odd number of panels being used to allow an entrance at the center. The spacing along the north façade is governed chiefly by the interior divisions.

Distance from Building Line. The distances of the columns from the building lines are governed by the fireproofing, as has been explained. They are 1′-10″ along the north and west façades, 1′-3″ along the alley and court, and 1′-0″ along the south side. This latter value is used because provision is made for a building on the adjoining lot which will supply any additional protection needed.

DESIGN OF STEEL MEMBERS

Design of Beams. The spacing of columns, arrangement of girders, and type of construction being settled, the next step is the design of the beams.

Joists. There are no joists except in a few cases and these can better be classed as special beams. Joists when used are almost invariably simple beams with uniformly distributed loads. Therefore, having computed the total load per square foot of floor, and having fixed the span and spacing, the total load on the beam is the product of these three quantities, and from it the size of beam is taken from the tables. Or, if the size has been selected, the capacity for the given span can be taken from the tables; and from this the floor area which it will support, and then the maximum spacing can be determined. The length of span and of load area used is the distance, center to center, of girders if the joist frames between girders, and the actual length of the joist if it connects to columns.

Girders. The typical girders were designed in connection with the preliminary study of the floor construction. The special cases remain to be designed. For example take girders *8-9* and *10-11*.

Girder *8-9* typical floor, Plate F, span 18′-6″. Load area on one side only.

$$\text{Total load u. d.} \qquad 18'\text{-}6'' \times 10'\text{-}0'' \times 196\# = 36{,}260\#$$

This requires a 15″ I 42#

Girder *10-11* typical floor, span 15′-3″. Heavier slab north side, lighter span south side.

$$\text{Total load u. d.} \begin{cases} 15'\text{-}3'' \times 10'\text{-}0'' \times 196\# = 29{,}890\# \\ 15'\text{-}3'' \times \ \ 6'\text{-}0'' \times 187\# = \underline{17{,}110}\# \end{cases}$$
$$47{,}000\#$$

This requires an 18″ I 46#*

On the first floor all the slabs are built with 10-inch tile and provision is made for a marble or a tutti colori floor. The live load allowance is 100 pounds per square foot. The partition allowance can be reduced to 20 pounds per square foot because of the larger rooms. Therefore, the load per square foot carried by the girder is

*Light weight Carnegie beam. These special beams are not always available.

Marble floor	10 lb.
Mortar	10 lb.
Concrete slab $3\frac{1}{2}''$	42 lb.
Concrete joists $4''\times10''$, $40\#\times\frac{3}{4}$	30 lb.
Tile $10''\times12''$, $32\#\times\frac{3}{4}$	24 lb.
Plaster	5 lb.
Reinforcing steel	3 lb.
Girder steel	4 lb.
Girder fireproofing	10 lb.
Partitions	20 lb.
Total dead load	158 lb.
Live load 85% of 100 lb.	85 lb.
Total load	243 lb.

Applying this to girder *8-9*, which has a span 18′-6″, gives

$$\text{Total load u. d.} \qquad 18'\text{-}6''\times19'\text{-}5''\times243\#=87{,}480\#$$

This requires a 24″ I $69\frac{1}{2}\#$

PROBLEMS

1. Design girder *9-10*, typical floor; girder *17-19*, first floor; and girder *13-20*, first floor, Plates F and C.

2. Compute the total load per square feet of floor in the freight room on the first floor (panel *29-30-37-36*). Floor, a reinforced concrete slab 8 inches thick. See Plates C and N for construction of floor. No partitions. Live load 150 pounds. Design the beam across the center of the panel.

3. Compute the load on the roof girders, and design girders *8-9*, *9-10*, and *10-11*. (See Plates G and J.)

Spandrel Girders. The spandrel girders in this design carry in most cases one-half panel of floor load and a panel of wall. The spandrel girders of the typical panels of the typical floors were designed in the study of the floor types.

The spandrel girder *1-8*, typical floor, carries only the wall load; this is practically uniformly distributed. The wall in this panel is 17 inches thick; its weight per square foot of surface is computed thus:

4 in. pressed	brick, 140 lb., per cu. ft.		47 lb.
$8\frac{1}{2}$ in. common	brick, 120 lb., per cu. ft.		85 lb.
$4\frac{1}{2}$ in. hollow	brick, 90 lb., per cu. ft.		34 lb.
			166 lb.

The wall surface is the panel area less the window area, viz,

$11'\text{-}0''\times18'\text{-}4''$	201 sq. ft.
Less $2\times3'\text{-}6''\times6'\text{-}0''$	42 sq. ft.
Net area	159 sq. ft.

Therefore the weight on the girder is

$$166 \times 159 = 26{,}400\#$$

The span is 18′-6″. This requires a 15″ I 36#. More exact computations would take into account the position of the windows, weight of concrete around beams, and weight of girder, but would not change the result in this case.

The effect of the wind stresses on the spandrel girders is considered later in the text.

PROBLEMS

1. Design spandrel girder *1-2*, typical floor.
2. Design spandrel girder *10-17*, typical floor.
3. Design spandrel girder *18-17*, typical floor.

Special Beams. Special beams are required around elevators and stairs, and for the support of elevator machinery, chimney, penthouses, and tanks.

Panel *30-31-38-37*. The panel *30-31-38-37* contains several special features, viz, a stairway, an elevator shaft, a chimney and vent space, and a pipe shaft. There is only a small section of floor in the panel, adjacent to column *37* on the typical floor.

In the north half of the panel the 8-inch I-beams support only partitions. None of them are fully loaded, but this size is considered the minimum for this situation.

The stair load may be taken at 50 pounds per square foot for the dead load and 100 pounds per square foot for the live load. It is supported by the 8-inch I-beam near column *37*, and the spandrel beam *31-38*. The latter beam cannot be placed at the floor level because the windows just above the stair landing interfere, so it must be placed near the level of the stair landing.

Framing around stair wells should be so designed that the weight of the stair can be supported from either the sides or the ends. In some cases the entire stair load is carried by the stringers to the beams at the ends of the well and in other cases hangers and struts transmit the loads to the side beams. Usually this cannot be determined by the structural steel designer unless he designs the stair.

PROBLEM

Design the cross beam near the middle of panel *30-31-38-37*, typical floor.

Panel *19-20-27-26*. The special framing in the panel *19-20-27-26*, Fig. 197 and Plate F, provides for elevators and stair. It presents no unusual features.

	Pent House	Attic	16th Sty.	15th Sty.	14th Sty.	13th Sty.	12th Sty.	11th Sty.	10th Sty.
Floor Live Load	....	4,800	8,100	7,600	7,100	6,700	6,200	5,700	5,200
Floor Dead Load	....	17,100	29,000	29,000	29,000	29,000	29,000	29,000	29,000
Wall Load, 8–1	....	e 16,000	e 13,300	e 12,400	e 12,400	e 12,400	e 12,400	e 12,400	e 12,400
Wall Load, 8–15	....	e 6,300	e 8,600	e 8,500	e 8,500	e 8,500	e 8,500	e 8,500	e 8,500
Column and Covering	....	7,600	7,600	7,600	7,600	7,600	7,600	7,600	7,600
Total for Story	....	51,800	66,600	65,100	64,600	64,200	63,700	63,200	62,700
Accumulated Total	....	51,800	118,400	183,500	248,100	312,300	376,000	439,200	501,900
Eccentric Effect	....	30,000	30,000	30,000	30,000	30,000	30,000	30,000	30,000
Total	....	81,800	148,400	213,500	278,100	342,300	406,000	469,200	531,900
Column Section	....	1 Pl. $12\times\frac{3}{8}$ 4 Ls $5\times3\frac{1}{2}\times\frac{3}{8}$		1 Pl. $12\times\frac{3}{8}$ 4 Ls $6\times4\times\frac{1}{2}$		1 Pl. $12\times\frac{5}{8}$ 4 Ls $6\times4\times\frac{11}{16}$		1 Pl. $12\times\frac{3}{4}$ 4 Ls $6\times4\times\frac{5}{8}$ 2 Pl. $14\times\frac{3}{8}$	

Fig. 203. Schedule of Loads for Column 8. Floor Area 190 square feet

	9th Sty.	8th Sty.	7th Sty.	6th Sty.	5th Sty.	4th Sty.	3rd Sty.	2nd Sty.	1st Sty.	Basement	Sub B'sm't
Floor Live Load	4,800	4,800	4,800	4,800	4,800	4,800	4,800	4,800	4,800	e 9,500	......
Floor Dead Load	29,000	29,000	29,000	29,000	29,000	29,000	29,000	29,000	32,300	e 30,000	......
Wall Load, 8–1	e 12,400	e 12,400	e 12,400	e 12,400	e 12,400	e 12,400	e 13,500	e 14,600	e 17,800	....	......
Wall Load, 8–15	e 8,500	e 8,500	e 8,500	e 8,500	e 8,500	e 8,500	e 8,500	e 8,500	e 11,200	....	......
Column and Covering	7,600	7,600	7,600	7,600	7,600	7,600	7,600	9,300	10,000	8,300	....
Total for Story	62,300	62,300	62,300	62,300	62,300	62,300	63,400	66,200	76,100	47,800	.....
Accumulated Total	564,200	626,500	688,800	751,100	813,400	875,700	939,100	1,005,300	1,081,400	1,129,200	....
Eccentric Effect	30,000	30,000	30,000	30,000	30,000	30,000	30,000	30,000	6,900	46,500	....
Total	594,200	656,500	718,800	781,100	843,400	905,700	969,100	1,035,300	1,088,300	1,175,700	.....
Column Section	1 Pl. $12\times\frac{3}{4}$ 4 Ls $6\times4\times\frac{3}{4}$ 2 Pl. $14\times\frac{1}{2}$		1 Pl. $12\times\frac{3}{4}$ 4 Ls $6\times4\times\frac{3}{4}$ 2 Pl. $14\times\frac{3}{4}$		1 Pl. $12\times\frac{3}{4}$ 4 Ls $6\times4\times\frac{3}{4}$ 4 Pl. $14\times\frac{9}{16}$		1 Pl. $12\times\frac{3}{4}$ 4 Ls $6\times4\times\frac{3}{4}$ 2 Pl. $14\times\frac{13}{16}$ 2 Pl. $14\times\frac{3}{4}$		1 Pl. $12\times\frac{3}{4}$ 4 Ls $6\times4\times\frac{3}{4}$ 6 Pl. $14\times\frac{5}{8}$		

Fig. 203 (Continued)

Penthouse. The penthouse, Plates G and S, between columns *29-31-38-36* contains a number of special items. Beams are required at the roof level to support the penthouse walls. At the roof level near columns *29-36*, two 18-inch I-beams are provided for the purpose of carrying two water tanks and the concrete platform on which they rest.

The machine platform, Plate S, at an elevation of about 18 feet above the attic floor, supports the freight elevator and its machinery. The arrangement of the sheave beams and the machinery, and the loads are given in Fig. 198. As previously directed, these loads must be doubled for the beams and their connections. It is not worth while to figure closely on the elevator supports. Only a small amount of material is involved, so all the computations should be on the safe side.

With the liberal treatment of the elevator loads suggested above there remains nothing complicated in the designing of the penthouse framings, but the work is tedious on account of the variety of loads and the irregular spacings.

PROBLEM

Check the framing in the penthouse between columns *29-31-38-36*, Plates G and S.

Sidewalk Construction. The sidewalk framing is shown on the first floor plan of the building, Plate C. A strip of prismatic lights extends along the building line, Plate K.

PROBLEM

Check the sizes of beams used in the sidewalk.

Design of Columns. Columns *8* and *9* are selected as typical exterior and interior columns for illustrating the computation of loads and the design.

Loads on Column 8. Fig. 203 gives the schedule of loads on column *8*. The floor area tributary to the column is 19′-5″×9′-10″, or 191 square feet; for convenience use 190 square feet. This area applies at all floors and the roof.

The dead loads are

Roof, per sq. ft.	90 lb.
3rd to attic floors, per sq. ft.	153 lb.
2nd floor, per sq. ft.	170 lb.
1st floor, per sq. ft.	158 lb.

The live loads per square foot for the successive floors after making the reductions described on p. 298 are,

Roof	25 lb.	9th floor	25 lb.
Attic floor	42½ lb.	8th floor	25 lb.
16th floor	40 lb.	7th floor	25 lb.
15th floor	37½ lb.	6th floor	25 lb.
14th floor	35 lb.	5th floor	25 lb.
13th floor	32½ lb.	4th floor	25 lb.
12th floor	30 lb.	3rd floor	25 lb.
11th floor	27½ lb.	2nd floor	25 lb.
10th floor	25 lb.	1st floor	50 lb.

Column *8* supports one-half of the wall between columns *1* and *8*, and one-half between columns *8* and *15*. As these panels of wall are not the same thickness, they are estimated separately. Their respective weights have been estimated to be 166 pounds and 121 pounds per square foot for wall surface.

The wall area estimated is the net area between columns, the width of column for this purpose being taken at 22 inches, out to out, of concrete. The brick facing for this width is estimated with the weight of the column. Between columns *1* and *8* in the typical story, the total wall area is 11′×17′-8″ or 194 square feet. From this is deducted the window area, Plate V, 2×3′-6″×6′-4″ or 44 square feet, leaving a net area of 150 square feet. One-half of this, 75 square feet, is carried by column *8*. At other stories the area differs because of different story heights and different windows. At the roof in this panel are a terra cotta balustrade and a cornice, Plate T; and at the 3rd and 4th floors are belt courses of terra cotta projecting beyond the wall line. These are irregular in shape but their dimensions can be scaled and their approximate weights computed at the rate of 120 pounds per cubic foot.

Between columns *8* and *15* in the typical story, the area supported by column *8* is 11′×17′-6″ or 192 square feet. From this is deducted the window area 2×4′×6′-4″ or 51 square feet, leaving a net area of 141 square feet. One-half of this, 70 square feet, is carried by column *8*. Note that the small window is neglected.

At the roof there is a parapet wall the dimensions of which can be scaled from the drawings.

The basement and first story walls are not supported by the steel framework.

For the weight of the column and covering an average amount per foot of length is computed and used for the whole length thus:

Steel	150 lb.
Concrete (22×22 less 40) say	450 lb.
Brick facing $4'' \times 22''$ say	90 lb.
	690 lb.

This amount is too large at the top and too small at the bottom.

From the foregoing data the loads on column *8* are computed and entered in the schedule in Fig. 203. For the column section in any given story the loads entered are the weight of the column in that story, the weight of the floor above, and the weight of the walls in the story above.

As the loads are entered, the eccentricity, if any, is noted as indicated by the letter *e*. At all floors from the second to the roof, one-half of the floor load comes to the column through the girder *8-9* and one-half hrough *8-15*. These connect on opposite sides and balance each other. At the first floor the entire floor load connects to one flange and, therefore, is eccentric. The wall loads are eccentric throughout, but at the second floor the wall load *1-8* is only slightly so and is on the opposite side of the axis from the wall load *8-15*. In the schedule, on the line marked "eccentric effect", are given the concentric equivalents of the eccentric loads computed from the formula

$$W'_e = W'\frac{ec}{r^2}$$

No serious error is committed if, for the shape of column here used, the value of r is taken at eight-tenths of c. The result can be checked back and the error corrected, if necessary, after the section has been selected. The values in the schedule are computed on this basis but the amount entered is three-fourths of the computed amount. Thus for the attic story column the computations are

$$W'_e = 22{,}300 \times \frac{7 \times 6\frac{1}{2}}{5 \times 5} = 40{,}600, \text{ say } 40{,}000\#$$

Three-fourths of this is 30,000, which amount is used. At all the typical floors, the result is so close to this amount that it may be used from the second story to the roof.

	Pent House	Attic	16th Sty.	15th Sty.	14th Sty.	13th Sty.	12th Sty.	11th Sty.	10th tSy.
Floor Live Load.	....	8,000	13,600	12,800	12,000	11,200	10,400	9,600	8,800
Floor Dead Load.	....	28,800	48,000	48,000	48,000	48,000	48,000	48,000	48,000
Column and Covering.	....	5,600	5,600	5,600	5,600	5,600	5,600	5,600	5,600
Total for Story.	....	42,400	67,200	66,400	65,600	64,800	64,000	63,200	62,400
Accumulated Total.	....	42,400	109,600	176,000	241,600	306,400	370,400	433,600	496,000
Eccentric Effect.	....	9,000	9,000	9,000	9,000	9,000	9,000	9,000	9,000
Total.	....	51,400	118,600	185,000	250,600	315,400	379,400	442,600	505,000
Column Section.	....	1 Pl. $12 \times \frac{3}{8}$ 4 Ls $5 \times 3\frac{1}{2} \times \frac{3}{8}$		1 Pl. $12 \times \frac{3}{8}$ 4 Ls $6 \times 4 \times \frac{7}{16}$		1 Pl. $12 \times \frac{1}{2}$ 4 Ls $6 \times 4 \times \frac{5}{8}$		1 Pl. $12 \times \frac{1}{2}$ 4 Ls $6 \times 4 \times \frac{5}{8}$ 2 Pl. $14 \times \frac{3}{8}$	

Fig. 204. Schedule of Loads for Column 9. Floor Area 320 square feet

	9th Sty.	8th Sty.	7th Sty.	6th Sty.	5th Sty.	4th Sty.	3rd Sty.	2nd Sty.	1st Sty.	Basement	Sub B'sm't
Floor Live Load.	8,000	8,000	8,000	8,000	8,000	8,000	8,000	8,000	8,000	16,000	......
Floor Dead Load.	48,000	48,000	48,000	48,000	48,000	48,000	48,000	48,000	53,400	50,600	......
Column and Covering	5,600	5,600	5,600	5,600	5,600	5,600	5,600	6,900	8,000	6,100	......
Total for Story.	61,600	61,600	61,600	61,600	61,600	61,600	61,600	62,900	69,400	72,700	......
Accumulated Total.	557,600	629,200	680,800	742,400	804,000	865,600	927,200	990,100	1,059,500	1,132,200	......
Eccentric Effect.	9,000	9,000	9,000	9,000	9,000	9,000	9,000	9,000	9,000	20,000	......
Total.	666,600	628,200	689,800	751,400	813,000	874,600	936,200	999,100	1,068,500	1,152,200	......
Column Section.	1 Pl. $12 \times \frac{3}{4}$ 4 Ls $6 \times 4 \times \frac{3}{4}$ 2 Pl. $14 \times \frac{7}{16}$		1 Pl. $12 \times \frac{3}{4}$ 4 Ls $6 \times 4 \times \frac{3}{4}$ 2 Pl. $14 \times \frac{3}{4}$		1 Pl. $12 \times \frac{3}{4}$ 4 Ls $6 \times 4 \times \frac{3}{4}$ 2 Pl. $14 \times \frac{5}{8}$ 2 Pl. $14 \times \frac{7}{16}$		1 Pl. $12 \times \frac{3}{4}$ 4 Ls $6 \times 4 \times \frac{3}{4}$ 4 Pl. $14 \times \frac{3}{4}$		1 Pl. $12 \times \frac{3}{4}$ 4 Ls $6 \times 4 \times \frac{3}{4}$ 4 Pl. $14 \times \frac{5}{8}$ 2 Pl. $14 \times \frac{9}{16}$		

Fig. 204 (Continued)

The eccentric effect at the first floor (on basement column) is

$$W'_e = 39{,}500 \times \frac{7\frac{1}{2} \times 7\frac{1}{2}}{6 \times 6} = 62{,}000\# \text{ (approx.)}$$

three-fourths of this is 46,500 pounds.

Note that the eccentric effect is not cumulative.

Loads on Column 9. The loads on column *9* are much simpler, consisting only of the weight of the column and the floor loads. The floor area is 19′-5″×16′-6″ or 320 square feet. On the floors, second to attic, the part of this area in the panel *9-10-17-16* is lighter than the rest of it. This is taken into account in the following dead loads:

Roof, per sq. ft.	90 lb.
3rd to attic floors, per sq. ft.	150 lb.
2nd floor, per sq. ft.	167 lb.
1st floor, per sq. ft.	158 lb.

The weight of the column per lineal foot is

Steel	150 lb.
Concrete (20×20 less 40)	360 lb.
Total	510 lb.

At each floor there is eccentricity due to the unequal loads from the girders *8-9* and *9-10*. On all floors from second to roof 160 square feet of the total area are applied to the column through girder *9-16* which connects to the web and is not eccentric; 96 square feet are applied through girder *8-9*; and 64 square feet through girder *9-10*. The difference between the last two amounts, 32 square feet, is the unbalanced area producing eccentricity.

The loads in the schedule for column *9*, Fig. 204, are computed from the foregoing data.

The eccentric effect is small and to save tedious calculations can be computed for average conditions at a typical floor and the result applied to all floors except the first. Thus, at the fourteenth floor the total of dead and live loads is 60,000 pounds; one-tenth of this, or 6,000 pounds, is unbalanced and, hence, is eccentric. The values of e and c are equal and may be assumed 7 inches; r may be assumed 5 inches.

$$W'_e = \frac{6{,}000 \times 7 \times 7}{5 \times 5} = 12{,}000\# \text{ (approx.)}$$

According to the rule adopted three-fourths of this amount is used, that is, 9,000 pounds. This is applied at all floors except the first, where the load conditions are different. After the column section is selected, the eccentric effect may be checked, using the actual values of e, c, and r.

Column Section. Type. The column section adopted for this building is the H-section built of plates and angles. It is selected because of its ease of manufacture, ease of making connections both to web and to flanges, and for commercial reasons.

Location. The position of the column as to the direction of greatest stiffness has been discussed, and in both of the examples the column is placed so that the stronger way resists the eccentric moment of the load.

Size. It is desirable, though not of great importance, that the general size of the column be maintained throughout the height. For this reason a 12-inch web plate is used, although this might be made 10 inches in the upper stories and 14 inches in the lower stories. If column *8* were made 10 inches in the upper stories, the eccentric effect would be so increased that the section required would probably be greater than for the 12-inch column. The use of the 14-inch web plate in the lower stories would decrease the weight of the columns but would make the finished columns larger and thus reduce valuable floor space.

Length. The columns are made in two-story lengths, the splices in this case being made at the even numbered floors, that is, at 2, 4, 6, etc. The columns which extend through the sub-basement are made in three-story lengths to bring the splice at the second floor so as to be at the same level as the others. The cross section of any length of column is governed by the stress in the lower of the two stories comprising that length.

Summary. Having the loads computed as given in the schedules and having established the foregoing general conditions, it only remains to select from the tables in the handbooks the sections required for the several lengths of column and enter them in the schedule. (See also Plate H).

In designing these columns* the maximum thickness of metal used is $\frac{3}{4}$ inch, because any metal thicker than this would require reaming or drilling and thus add to the cost. When the total thick-

*The tables referred to on pp. 189 and 194 were not used in making this design; so the identical section may not be found therein.

ness of cover plates on one flange is more than $\frac{3}{4}$ inch, two or more plates are used, each being $\frac{3}{4}$ inch or less in thickness. No cover plates are used unless the stress is beyond the capacity of a section having $\frac{3}{4}$-inch metal in the web plate and angles.

PROBLEMS

1. Compute the loads and make the design for column *16*. (Note that this column extends through the sub-basement.) Make schedule as in Figs. 203 and 204.

2. Compute the loads and make the design for column *17*. (Note that the court walls do not occur below the third floor.)

3. Make a diagram showing the floor areas supported by column *17* at the first, second, third, and typical floors, Plates C, D, E, and F.

4. Give detailed computations of the wall load supported by column *17* in a typical story, Plates F, R, and W.

Column Pedestals. The piers under the columns are round and, therefore, in order to distribute the load as evenly as possible, round cast-iron pedestals of the type shown in Fig. 152 and Plate H are used. The bearing allowed on the masonry in this case is 800 pounds per square inch. The load for column *8* is 1,129,000 pounds and for column *9* is 1,132,000 pounds. (The eccentric effect is not included.) The area required is 1415 square inches, which corresponds to a circle 42 inches in diameter. But for the sake of using few patterns, the diameter is made 44 inches.

Height. While the height of the pedestal is taken at 24 inches, there is no very definite way of determining the height. However, a number of trial designs indicate that pedestals of the type here used should be proportioned as follows:

For a bearing of 800 lb. per sq. in., height 53% of diameter
For a bearing of 600 lb. per sq. in., height 43% of diameter
For a bearing of 400 lb. per sq. in., height 35% of diameter

Top. The size of the top of the pedestal is controlled by the detail of the base of the column. It must extend far enough beyond the hub to provide holes for connecting to the column; $2\frac{1}{2}$ inches at the narrow place is usually enough and this is available to resist the bending moment. The thickness is assumed arbitrarily at $1\frac{1}{2}$ inches.

Ribs. The number of ribs assumed is eight. Their thickness is not less than one-twentieth the height, that is, $1\frac{1}{4}$ inches.

Diameter of Hub. The diameter of the hub is made such that the greater part of the column section is directly over it. In this case 11 inches inside diameter is suitable. The thickness of the hub must be such that its area together with that of the ribs under the top plate will support the column load at 10,000 pounds per square inch. Thus the total area required is 113 square inches. The area of the ribs to be counted is $8\times2\frac{1}{2}''\times1\frac{1}{4}''$ or 25 square inches, thus leaving 88 square inches to be provided in the hub. This requires $2\frac{1}{4}$ inches thickness of metal, which makes the outside diameter $15\frac{1}{2}$ inches. The area of the 11-inch circle is 95 square inches, and of the $15\frac{1}{2}$-inch circle 188 square inches; the difference, 93 square inches, is slightly more than required.

The thickness of the bottom plate must be assumed for trial; use $2\frac{3}{4}$ inches.

The dimensions of the rim are fixed arbitrarily $1\frac{1}{2}$ inches thick and 5 inches high.

Test for Resistance to Bending. Having determined or assumed the thickness of metal in the various parts of the pedestal, it is now necessary to test the cross section for its resistance to bending. The procedure is the same as that given on p. 220.

Center of Gravity. To locate the center of gravity and the neutral axis, take the following:

Bottom plate	area $41\times2\frac{3}{4}$	$=112.75$	$M\,112.75\times\ 1.375=$	155.05
Hub	area $2\times19\frac{3}{4}\times2\frac{1}{4}=$	88.90	$M\ 88.90\times12.625=$	1122.36
Top plate	area $2\times\ 1\frac{1}{2}\times2\frac{1}{2}=$	7.50	$M\ 7.50\times23.25\ =$	174.37
Rim	area $2\times\ 5\ \times1\frac{1}{2}=$	15.00	$M\ 15\ \times\ 5.25\ =$	78.75
		224.15		1530.53

The distance of the neutral axis from the bottom of the plate is $\frac{1530.53}{224.15}$ or 6.85 inches.

Moment of Inertia. The moment of inertia of the section about the neutral axis is

$$\text{For bottom plate}\quad I=\begin{cases}\frac{1}{12}\times41\times(2.75)^3 & =\ 71\\ 112.75\quad\times(5.48)^2 & =\ 3386\end{cases}$$

$$\text{For hub}\quad I=\begin{cases}\frac{1}{12}\times2.25\times(19.75)^3\times2 & =\ 2880\\ 88.9\times\quad(5.78)^2 & =\ 2969\end{cases}$$

$$\text{For top plate}\quad I=\begin{cases}\frac{1}{12}\times2.50\times(1.5)^3\times2 & =\ 1\\ 7.5\ \times(16.4)^2 & =\ 2018\end{cases}$$

$$\text{For rim}\quad I=\begin{cases}\frac{1}{12}\times1\frac{1}{2}\times(5)^3\times2 & =\ 31\\ 15.0\times(1.6)^2 & =\ 38\end{cases}$$

$$\text{Total moment of inertia}=11{,}394$$

Resisting Moment. The resisting moment of the section is

$$R\,M = \frac{3000 \times 11{,}394}{6.85} = 4{,}990{,}000 \text{ in.-lb.}$$

The bending moment of the load is

$$M = 1{,}132{,}000 \times 44 \times \frac{1}{10} = 4{,}980{,}000 \text{ in.-lb.}$$

Hence the assumed plate has the required resistance to bending.

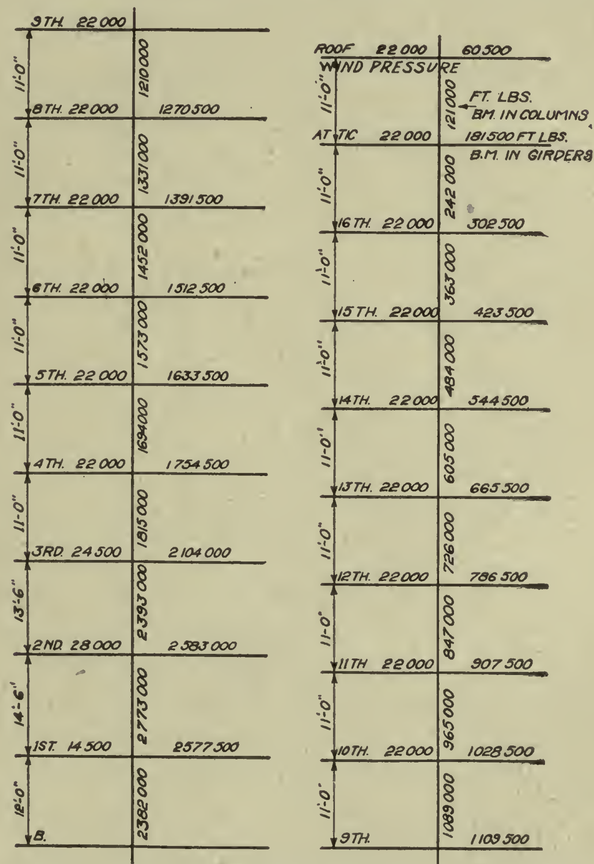


Fig. 205. Diagram Showing Bending Moments Due to Wind Pressure

At columns *36, 37, 38, 40, 41,* and *42,* the piers are built centrally on the lot line to support two sets of columns. The bases cannot be extended beyond the lot line, so are made rectangular. Three

I-beams are used for this purpose, as illustrated in Plate H. The method of designing has been explained under bearings for beams.

WIND BRACING

Wind Loads for Entire Building. The wind load is assumed to be 20 pounds per square foot, all of which is to be resisted by the steel frame. Fig. 205 is a diagram on which are marked the wind loads for the successive stories and the resulting bending moments in the columns and the girders. The values given are for the entire building and, as the building happens to be practically square in plan, the diagram applies for both directions.

At each of the upper floor levels, the load applied to the framework is $100 \times 11 \times 20$ or 22,000 pounds. The first, second, and third floors support different areas, hence different loads.

Bending Moments. *In Columns.* The bending moments in the columns are computed as follows:

Attic	$22{,}000 \times 5\frac{1}{2} =$	121,000 ft.-lb.
16th story	$44{,}000 \times 5\frac{1}{2} =$	242,000 ft.-lb.
15th story	$66{,}000 \times 5\frac{1}{2} =$	363,000 ft.-lb.
etc., etc.		
1st story	$382{,}500 \times 7\frac{1}{4} =$	2,773,000 ft.-lb.
Basement	$397{,}000 \times 6 =$	2,382,000 ft.-lb.

In Girders. The bending moments in the girders, according to the rule previously established, are the means between the bending moments in the columns. The values are

Roof $\quad \dfrac{121{,}000 + 000{,}000}{2} = 60{,}500$ ft.-lb.

Attic floor $\quad \dfrac{121{,}000 + 242{,}000}{2} = 181{,}500$ ft.-lb.

16th floor $\quad \dfrac{242{,}000 + 363{,}000}{2} = 302{,}500$ ft.-lb.

etc., etc.

2nd floor $\quad \dfrac{2{,}393{,}000 + 2{,}773{,}000}{2} = 2{,}583{,}000$ ft.-lb.

1st floor $\quad \dfrac{2{,}773{,}000 + 2{,}382{,}000}{2} = 2{,}577{,}500$ ft.-lb.

Note that the bending moments in the girders given above are for one side only of the column; an equal amount occurs at the other side, making the total amount to be resisted at each floor twice that given.

Resistance of Spandrel Girders. Consider now the wind from the North or from the South. At all floors resistance is offered by the spandrel girders between columns *1-36* and *7-42* (except at first floor *1-36*), and by interior and court wall girders in a north and south direction.

In the upper part of the building, the girder sections which are required by the gravity loads are sufficient for resisting the wind stresses. The first step is to determine the resistance that can be developed by these girders and then find at which floor it is necessary to use special construction. The connections of the spandrel girders are shown in Plate F and Fig. 190. The horizontal shearing value of the (field) rivets in one flange at 50 per cent excess values is $4 \times .44 \times 15{,}000$ or 26,400 pounds. Then the resisting moments for one end of each beam of various depths are as follows:

12-inch beam	$1 \times 26{,}400 = 26{,}400$ ft.-lb.
15-inch beam	$1\frac{1}{4} \times 26{,}400 = 33{,}000$ ft.-lb.
18-inch beam	$1\frac{1}{2} \times 26{,}400 = 39{,}600$ ft.-lb.
20-inch beam	$1\frac{2}{3} \times 26{,}400 = 44{,}000$ ft.-lb.
21-inch beam	$1\frac{3}{4} \times 26{,}400 = 46{,}200$ ft.-lb.
24-inch beam	$2 \times 26{,}400 = 52{,}800$ ft.-lb.

This applies to the court spandrels as well as to the outside spandrels.

Resistance of Interior Girders. The connections of the interior girders are shown in Plate I and Fig. 191. In the case where each flange is connected by six $\frac{3}{4}$-inch rivets, the horizontal shear resistance is $6 \times .44 \times 15{,}000$ or 39,600 pounds. Then the resisting moments for one end of beams of various depths are as follows:

12-inch beam	$1 \times 39{,}600 = 39{,}600$ ft.-lb.
15-inch beam	$1\frac{1}{4} \times 39{,}600 = 49{,}500$ ft.-lb.
18-inch beam	$1\frac{1}{2} \times 39{,}600 = 59{,}400$ ft.-lb.
20-inch beam	$1\frac{2}{3} \times 39{,}600 = 66{,}000$ ft.-lb.
21-inch beam	$1\frac{3}{4} \times 39{,}600 = 69{,}300$ ft.-lb.
24-inch beam	$2 \times 39{,}600 = 79{,}200$ ft.-lb.

On a typical floor the number of connections and their values are:

4 spandrel beams	15 inch at 33,000 = 132,000 ft.-lb.	
12 spandrel beams	18 inch at 39,600 = 475,200 ft.-lb.	
18 spandrel beams	21 inch at 46,200 = 831,600 ft.-lb.	1,438,800
4 interior beams	18 inch at 59,400 = 237,600 ft.-lb.	
14 interior beams	21 inch at 69,300 = 970,200 ft.-lb.	1,207,800
		2,646,600

This resistance is sufficient at the eighth floor and upward. Above the tenth floor the interior connections may be reduced, as indicated on Plate I.

The interior connections cannot be increased without the use

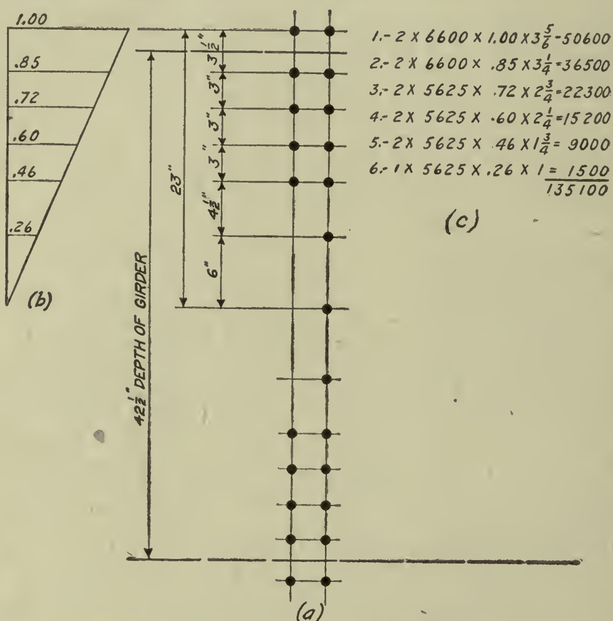


Fig. 206. Diagram Showing Method of Computing Resisting Moment of Girder Connection

of brackets which would project through the fireproofing. The spandrels are not so limited and brackets can be used to increase their resistance. In this manner the resistance to wind stresses can be provided down to and including the fourth floor, Fig. 192.

Girder Resistance for Third Floor. At the third floor, the wall construction is such as to make desirable deep spandrel girders between columns *1-8* and *7-42*. These girders with their connections are shown in Plate E. The total amount to be resisted at the third floor is $2 \times 2{,}104{,}000$ or 4,208,000 foot-pounds. Of this about 1,500,000 foot-pounds are resisted by the interior beams, leaving 2,700,000 foot-pounds to be resisted by the spandrel beams. Consider this divided equally between the two sides, there being 10 connections on each side, so that each connection in the spandrels *1-36* and *7-42* must resist 135,000 foot-pounds. This requires brackets of the type shown in Fig. 192 for the I-beams *8-36* and the connections shown for the plate girders, Plate E.

The computations of the connection of the plate girders are shown in Fig. 206; *a* is the rivet spacing; *b* is a graphical diagram giving the proportions of the full rivet stress for the rivets at various distances from the center, and *c* is the computations. Thus item 1 is 2 field rivets, $\frac{3}{4}$-inch diameter, in single shear, at full unit stress, with a moment arm of $3\frac{5}{6}$ feet; item 3 is 2 field rivets, $\frac{3}{4}$-inch diameter, in single shear, at 0.72 of the full unit stress, with a moment arm of $2\frac{3}{4}$ feet. The total resistance is somewhat larger than required.

The girder section is excessive, the depth being fixed by the spandrel construction, and plates and angles being the minimum sizes suitable for this situation.

Girder Resistance for Second Floor. At the second floor the interior girders are arranged differently, so their resistance must be computed. The methods just given, applied here give 172,000 foot-pounds as the bending moment at each spandrel connection. The connections to the columns are designed in the manner previously illustrated, Plate D.

Girder Resistance for First Floor. At the first floor there are no spandrel girders between columns *1-36*. The columns in this row are bedded in the basement wall. The wall is assumed to resist one-half of the wind stress at this floor. The other half of the stress is resisted by the interior girders *20-41* and the spandrel girders *7-42*, Plate C.

The mistake is sometimes made of neglecting the wind bracing at the first floor. *This is the most important place where it should be given attention.* It cannot be expected that the pressure will be transmitted to the earth at a higher level than the basement floor.

Proof of Column Sections. It remains to be determined whether the column sections are overstressed by adding the wind stress to the gravity stresses. One case serves to illustrate the method.

At the second floor, the bending moments in column *8*, corresponding to those in the connecting spandrel girders, are 160,000 foot-pounds and 184,000 foot-pounds above and below the floor, respectively. Consider the first-story column. This bending moment is based on a moment arm of $7\frac{1}{4}$ feet. The critical section is at the base of the bracket which is 3 feet below the center of the girder. At this point the bending moment is $184{,}000 \times \frac{4\frac{1}{4}}{7\frac{1}{4}}$ or 108,000 foot-pounds, or 1,296,000 inch-pounds.

The column section in the first story is

1 web plate	$12'' \times \frac{3}{4}''$
4 Ls	$6'' \times 4'' \times \frac{3}{4}''$
6 cover plates	$14'' \times \frac{5}{8}''$

The bending is about the axis which is parallel to the web, so the values of c and r must be taken in reference to this axis. c is 7 inches, one-half of the width of cover plate, and r taken from the tables for this column is 3.5. Then the concentric equivalent load is

$$W_e' = \frac{1{,}296{,}000 \times 7}{3.5 \times 3.5} = 740{,}000\#$$

The gravity load on this column is 1,088,000 pounds, making the total for which it must be designed 1,828,000 pounds. The length may be taken at 11 feet on account of the depth of bracket. According to the column formula, this section is good for 1,196,000 pounds. For the combined stress this is increased 50 per cent and equals 1,794,000 pounds. As this is within 2 per cent of the required capacity, it is accepted.

The designer is warranted in making liberal assumptions as to the lengths of columns and the allowance of excess stress when they are built into substantial masonry walls.

This case illustrates the desirability of carrying as much of the wind load as practicable on the interior columns and girders, otherwise the exterior columns may need to be increased above the requirements of the gravity loads in order to take the heavy wind stresses.

In cases like that above, it may be best to turn the columns in the other direction. It is simply a question whether the effect of the wind stress is more important than the effect of the eccentric gravity loads.

Other Wind Stresses. Now, consider the wind from the East or from the West. It happens that the south wall of the building is solid, so that diagonal bracing can be used, as shown in Plate I, and such bracing is designed to take one-half of the wind stress in this direction. At the ninth floor a strut extends across the court so that the two sets of bracing co-operate below that level. The other half of the wind stress is carried by the interior east and west girders and the spandrel girders *1-7*. The problems involved do not differ from those that have been described.

MISCELLANEOUS FEATURES

Chimney and Its Supports. The chimney, Plates H and I, is located near column *31*. It extends from the sub-basement floor to the top of the penthouse. It is made of steel plates. The thickness of plates is arbitrary, the chief consideration being durability. The chimney is lined inside with an insulating material which is supported by shelf angles spaced 3 feet apart. The chimney is designed to be built in sections corresponding to the two-story column lengths. The sections are joined together by means of flange angles and bolts.

The entire weight of the chimney must be carried from one support, as its length varies with changes in temperature. So far as the finished structure is concerned, it could rest on the sub-basement floor, but for convenience in erection it is supported at the first floor. Thus it can be erected along with the structural steel, the basement and sub-basement sections being placed at any convenient time afterward. Usually the sub-basement work is not done until after the steel framework is erected and it would then be difficult to get the chimney into place.

The details of the breeching connection are given to control both the structural steel fabricator and the builder of the breeching.

Masonry Supports. Along the two facades at the first floor are some granite bases which require supports. These supports, detailed in Plate C, are made independent of the sidewalk construction so that the granite can be set in advance of building the sidewalk and also so it will not be affected by any possible settlement of the sidewalk.

At all floor levels or other convenient points, provision must be made for supporting the masonry across the face of the columns. This can be done on this building in most cases by extending a part of the spandrel sections across the column. But in many buildings special shelves must be built.

Lintels. Most of the spandrel girders are so located that they serve as lintels over the windows. Plates are riveted on the bottom flange over these openings to support the outer course of bricks or the terra cotta lintel. The edge of the plate is placed 2 inches back from the outer face of the brickwork. Some designers prefer to extend these plates the entire length of the girder to support the face brick, Plates L and T. When the windows are not high enough for the above lintel detail, detached angle lintels are used.

Spandrel Sections. On buildings having elaborate facades, many special details must be designed for supporting the masonry. The spandrel sections on this building, Plates L and T, are comparatively simple.

At the second floor a projecting plate is used along the bottom flange of the girder. At the third floor a similar plate is used and, at the top of the girder, brackets project out for supporting a belt course of terra cotta.

Ornamental metal balconies at the seventh, ninth, eleventh, and thirteenth floors are supported by light angle brackets riveted to the girders.

A terra cotta balcony at the fifteenth floor requires the special framing shown for it.

In general, wherever terra cotta is used, anchor holes are required in the structural steel. It is the duty of the designer to secure the necessary data and put it on the drawings. These holes usually are spaced about six inches apart horizontally. Only the vertical dimensions need be supplied.

The cornice support is quite similar to that of the terra cotta course at the third floor. For wide cornices, brackets project from the columns, and these brackets carry beams for the support of the terra cotta. Every case requires its special design.

Flag Pole Support. Near column *7* on the roof plan, Plate G, is shown a pair of channels for supporting a flag pole. A similar pair of channels occurs at the attic floor. On some buildings the flag pole can be connected directly to a column. This is the simplest and most desirable scheme. In some cases it may be set in sockets on the roof and braced with angle or other struts.

No data are known to the writer regarding the load on a flag pole. A load of 20 pounds per square foot applied to the area of the flag seems sufficient to cover the actual wind pressure and vibration.

Mullions. Where the space between windows is not enough to permit a substantial masonry pier, the mullion should be reinforced. I-beams, tees, or angles may be used, depending on the conditions. In this case two rods are built into the brickwork, Plate L.

Anchors. The anchor rods shown extending through the spandrel girders and into the concrete slab hold the spandrel girders laterally and make a rigid connection between the framework and the floor construction, Plate L.

DIMENSIONING DRAWINGS

Base Lines. The base lines for horizontal dimensions are the building lines of the structure. They are shown on the first-floor plan, Plate C. The building lines nominally represent the outside lines of the building walls. In reality they are often imaginary reference lines, for, on account of the offsets, parts of the wall may extend beyond these lines and other parts be inside of them. For the class of buildings under consideration, the building lines usually coincide with the lot lines. If they do not, then the lot lines should be shown and dimensioned from the building lines. If the corners of the building are not exactly right angles, the angles must be marked on the first-floor plan. The cardinal points of the compass should be marked with approximate accuracy on the first-floor plan. One of these points is used as a reference in marking one side of columns and one end of girders for convenience in erecting; thus *E* on the east face of a column, or *N* on the north end of a girder.

Column Centers. Having established the building lines, the next step is to dimension the column centers. The simplest situation is had when the building is rectangular and the columns are in rows in both directions. Then two lines of dimensions will suffice to fix the location of all columns, Plate D. Any irregularity of spacing in any row requires a special line of dimensions in that row.

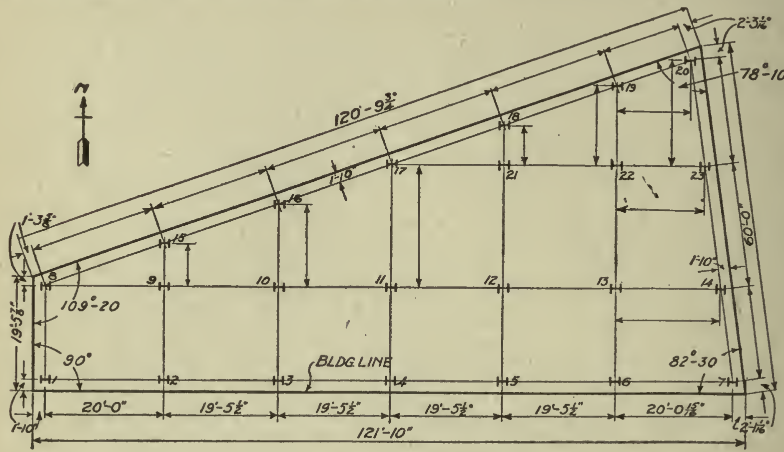


Fig. 207. Diagram Showing Method of Dimensioning Column Centers in an Irregular Building

With an irregularly shaped building, the dimensioning becomes more complicated. One building line should be adopted as a reference line, taking the one to which the greatest number of column lines are perpendicular and parallel. Then all columns should be located by dimension lines perpendicular and parallel to this reference line, that is, by rectangular co-ordinates. The only diagonal dimensions needed are those along which, or parallel to which, steel members are placed.

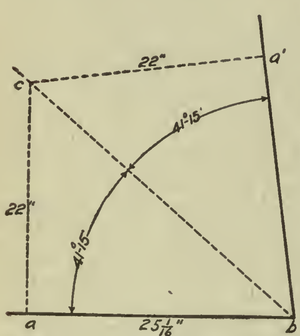


Fig. 208. Construction Diagram for Details of Figure 207

In Fig. 207, the reference line used is the south building line. The building lines in this case are probably lot lines. Their lengths and the angles are determined by a survey. The distance from the lot lines to the column centers is established at 1′-10″ on all sides. The spacing of columns *1* to *7* and the arrangement of the other columns are fixed by architectural conditions.

From the foregoing data all the required dimensions can be computed by trigonometry. First, compute the distances from column *7* to the corner of the building. From Fig. 208 it is apparent that these distances $a\ b$ and $a'b$ are equal to each other and equal to $c\ a \times \cot 41° 15'$; then

$$a\ b = a'b = 22'' \times 1.140 = 25\tfrac{1}{16}''$$

The distance between columns *9* and *15* is

$$20'\text{-}0'' \times \tan 19° 20'' = 20'\text{-}0'' \times .3508 = 7'\text{-}0\tfrac{3}{16}''$$

In this manner all the dimensions can be computed.

PROBLEM

Compute the distances between columns which are lacking in Fig. 207.

The column center dimensions should be repeated on all the floor plans. If the floor framing plan is crowded, a separate diagram at small scale may be placed on the drawing to display the column center distances.

Girders and Joists. Girders and joists are dimensioned from the column centers. The dimension lines required are illustrated in Figs. 201 and 202. Note in Fig. 201-b that there is no joist at column *23*, so the space is divided and the adjacent joists tied in the column. No dimensions are required for the lengths of joists and girders other than those locating the centers of the columns and beams to which they connect. The shop detailer computes the actual lengths of beams required. But if one end of a beam rests on a wall, one face of the wall and its thickness must be given.

Such details as struts, mullions, plates for supporting brickwork, etc., are also located from column centers, as illustrated on the floor plans.

Vertical Dimensions. The vertical dimensions from floor to floor are given in a separate diagram or in connection with the column schedule, Plate H. At the first floor a reference is made to established sidewalk grade in terms of its elevation above datum, Plate C. The elevations of beams are given in reference to the finished floor elevations, respectively. Usually the elevation of joists and girders can be covered by a note, Plate F. Special cases can be given by figures alongside the beams indicating the distance from the floor level to the top flange of the beam; thus $-5\frac{1}{2}''$ means that the top flange is $5\frac{1}{2}$ inches below the floor line.

Elevations of Spandrel Beams. The elevations of spandrel beams can be shown best on the sections, where both the elevation and the horizontal position can be given in relation to the other materials of construction thereabout, Plate L.

Summary. The use of unnecessary dimensions and needless repetitions may be a source of much inconvenience. It increases the probability of errors and causes extra work in checking.

While structural steel drawings should be made reasonably accurate to scale, scaled dimensions must not be used in executing the work.

The scales used in making drawings of structural steel should be as follows: for framing plans, $\frac{1}{8}$ inch or $\frac{1}{4}$ inch; for spandrel sections, $\frac{1}{2}$ inch or $\frac{3}{4}$ inch; and for details showing all dimensions and rivet spacing, 1 inch or $1\frac{1}{2}$ inches. In each case the scale first given is preferred. The use of a number of different scales in the same set of drawings is objectionable.

WOOLWORTH BUILDING IN PROCESS OF CONSTRUCTION
Cass Gilbert, Architect
Courtesy of Thompson-Starrett Company, New York City

STEEL CONSTRUCTION

PART V

PROTECTION OF STEEL

PROTECTION FROM RUST

Rust. Although steel is the strongest of building materials, under unfavorable conditions it may be one of the least durable. Its great enemy is rust. The corrosion or rusting of iron and steel is familiar to every one. It is a chemical change in which the metallic iron unites with oxygen and forms oxide of iron or rust.

RUST FORMATION

Theory. While rust is largely or wholly oxide of iron, it is not produced directly by the contact of the iron with the oxygen of the air. The presence of moisture seems essential to its formation. Much study has been given to the process of rust formation, but the reactions have not yet been determined positively. It is quite generally believed that electrolytic action occurs. This theory is well described by Houston Lowe in "Paints for Steel Structures" as follows:*

"The electrolytic theory, which no doubt has the strongest support, is based upon the recognized tendency of metals to go into solution, even in pure water. The act is accompanied by the release of hydrogen positively charged with electricity, leaving on the metal a corresponding charge of negative electricity. If oxygen is at hand to combine with the hydrogen, the electrical tension is relieved in an infinitely small current and new portions of the metal pass into solution; otherwise the action is arrested by the non-conducting quality of the thin film of hydrogen.

"The presence of minute particles of suitable impurities in or on the iron, whose solution tension differs from the iron, or the presence of acids in the water, facilitates the discharge of the electric tension and, hence, the continuous removal of particles of iron. On the other hand, the presence of alkalies, and a few other substances that decrease hydrogen ion concentration, will diminish or even stop iron solution and rusting altogether.

"This, in brief, is the substance of the electrolytic theory of rusting, the

*John Wiley & Sons, Publishers, New York.

more complete explanation of which would involve the details and language of the ionic theory of chemical action. Corrosion of iron, in the sense in which that term has been used in this section, has nothing whatever to do with electrolysis by stray electrical currents from outside sources. The currents involved in rusting under the theory of electrolytic action are almost infinitely short and minute, and originate in or on the metal itself.

"The theory is valuable to the extent that it suggests reasonable and practical remedy of the defects either of the metal or its proposed covering, or both. As in the treatment of diseased animal and plant tissues, so in this case, intelligent diagnosis must precede the application of preventives of rust. Experimental work following the lines of the electrolytic theory in seeking, first, to prevent, or 'inhibit' corrosion by a priming coat and, secondly, to diminish the penetration of water by suitable overcoats, is promising good results, and a final solution of the problem is confidently looked for.

"The tendency of rust to grow and spread out from a center has an adequate explanation in the electrolytic theory. This phenomenon is especially pernicious, as it results in pitting or, under a paint coat, in a growth which finally flakes off the paint and exposes large areas of the iron."

Degrees of Exposure. A piece of steel exposed to the air will ultimately change entirely to oxide of iron (except as to the contents other than pure iron) i. e., it will be entirely destroyed by rusting. The rapidity of the change varies with the conditions of exposure. The rusting will proceed very slowly if the steel is kept in dry air; less slowly if subjected occasionally to moist air; rapidly if exposed to moisture frequently; and very rapidly if exposed to moisture in the presence of sulphur or other acid fumes.

The first condition prevails when steel is enclosed in other materials of construction, as columns and beams enclosed by plaster in partitions, and in floor construction, so that the moisture conditions change only slightly. The second condition applies when the steel is within the building, but not encased in other materials, thus being exposed to varying degrees of moisture, as unprotected columns and beams in storerooms. The third degree of exposure fairly represents unprotected beams in basements, vaults under sidewalks, and steel work out of doors. And the worst possible exposure, that is, to moisture in the presence of acid fumes, is had in smelters, and in structures where the steel is subjected to the smoke from railroad locomotives.

Rate of Rusting. Some studies have been made of the rate of corrosion under different conditions. It is very evident that the rate varies greatly with the conditions of exposure. Experiments along this line have not gone far enough to give conclusive results,

i. e., definite figures as to the thickness of metal that will change to rust in a given time. But it is a matter of common knowledge that there is enough rusting even under the most favorable conditions to make it important that steel be protected.

Effect of Composition of Metal. The composition of the metal has some effect on the rate of corrosion. Structural steel probably rusts more rapidly than any other form or alloy of iron. Cast iron rusts slowly, probably due to the presence of graphite, which protects the iron. Wrought iron rusts more rapidly than cast iron and much less rapidly than steel. It is believed that the slag in wrought iron protects the fibers of iron from exposure to the air and moisture. The presence of manganese is supposed to accelerate corrosion, while copper and other alloys retard it.

Efforts have been made to produce rust-resisting metals by two methods; by making iron nearly pure, and by using an alloy of copper. The resulting metals are not rustproof but show much slower rates of corrosion than ordinary steel. Both have been commercially successful as applied to sheet steel, but are not yet used for structural steel. Pure iron is not suitable for structural purposes because of its lack of strength. It is quite possible that an alloy of copper or other metal will be developed for structural steel that will be nearly rustproof.

PAINT

Purpose. The usual means employed to prevent corrosion is to exclude all air and moisture from contact with the metal by a covering of paint. It is desirable that the paint material be such as will inhibit the formation of rust, thus counteracting any imperfections of the paint in excluding moisture.

Qualities. The following qualities are desirable:

(1) Adhesive, so that it will hold fast to the steel.
(2) Non-porous, so that it will exclude air and moisture.
(3) Elastic, so that it will not crack with changes in temperature, or with the deflection of the steel.
(4) Hard at all ordinary temperatures.
(5) Non-volatile, so that the oils may not evaporate and leave the inert materials of the paint without a binder.
(6) Not soluble in water.
(7) Not soluble in oil, so that it will not soften when additional coats are applied.

(8) Inhibitive, that is, of such material as will prevent the chemical or electrolytic action of rusting.
(9) Color may be important.

Many of these qualities obviously are much more important on out-of-door work than on ordinary building work. No paint has all of these desirable qualities, but by using different paints for the several coats, the ideal conditions may be approximated. Thus the first coat should be inhibitive and adhesive; and the second (or last coat, if more than two are used) should be non-porous and should provide the required wearing properties.

Composition. A paint is made of a liquid and a solid, called, respectively, the "vehicle" and the "pigment".

Vehicle. The best vehicle for paint is linseed oil. It may be had as raw oil or boiled oil. The latter is used when quick drying is desired but the raw oil is believed to give better results under most circumstances, and especially with red lead. The drying of paint is accelerated by the use of driers in the oil. A drier may be a volatile oil, as turpentine, which effects its purpose by rapidly evaporating after the paint is applied; or it may be a japan, which hastens the hardening of the oil and pigment. Turpentine being cheaper, it is more used than japans. The drier should not exceed 8 per cent of the vehicle.

Linseed oil varies greatly in quality even when pure, and is subject to adulterations which are difficult to detect. Some paint makers claim, and probably justly so, that they improve the vehicle by adding other oils to the linseed oil; but in general any additions other than the drier must be considered adulterations.

Pigments. Pigments commonly used for structural steel paints are red lead, iron oxide, graphite, and lampblack.

Red lead is the red oxide of lead, $Pb_3\ 0_4$, but the red lead of commerce contains a certain amount of litharge and metallic lead. These elements cannot be entirely eliminated on a commercial basis, but it is practicable to obtain a red lead which is 95 per cent pure and it should be so specified.

When mixed with linseed oil, red lead hardens, much as cement when mixed with water, and forms a strong tenacious coating. It can be made into a heavy paint, almost a paste, thus giving a heavy coat on the steel, or it can be thinned to give a light coat. On account of its weight, red lead is difficult to mix with oil. This is especially true when a large proportion of lead is used. The maximum proportion is 33 pounds of red lead to one gallon of raw linseed oil. While this heavy mixture is desirable, it is expensive as to labor and materials. A more practicable proportion is 25 pounds of red lead to one gallon of oil; a still smaller weight of lead is often used and will invariably be used unless the proportions required are definitely specified, for there is no standard practice to govern it. Red lead paint with a small proportion of red lead can be mixed by hand, but if the amount of lead is as much as 25 pounds, the mixing should be done in a churn, or ground into the oil at the paint factory.

On account of its weight and its settling qualities, it has not been practicable, heretofore, to keep red lead paint for any length of time, as the lead settles to the bottom and hardens. The hardening quality seems to be due largely to the litharge. Now that the litharge can be eliminated from the red lead, it is practicable to keep the ready-mixed paint for a much longer period. It can now be obtained from the paint manufacturers ground into the oil, forming a thick paste, which can be thinned to the proper consistency by the addition of oil when it is to be used. The thinning can be gaged by the weight of the finished paint on the following basis:

A weight of 24.43 pounds for the finished paint corresponds to 25 pounds of lead to one gallon of oil.

A weight of 25.92 pounds corresponds to 28 pounds of lead to one gallon of oil.

A weight of 26.76 pounds corresponds to 30 pounds of lead to one gallon of oil.

A weight of 27.10 pounds corresponds to 33 pounds of lead to one gallon of oil. (These values are taken from a circular issued by the National Lead Company.).

A ready-mixed red lead paint can be made by substituting for a part of the red lead some other pigment of inert material which will retard the settling, and harden. Lampblack, asbestine, and mica are sometimes used for this purpose. Such paints usually contain less than 15 pounds of red lead per gallon of oil, and are much less satisfactory than the red lead paste.

Iron oxide, commercially available, varies greatly in weight and physical characteristics. Some is taken direct from mines but most of it is manufactured. It does not have any cementing properties when mixed with linseed oil so must be held in place by the oil. The paint will last only as long as the oil binder remains intact. The iron oxide does not inhibit corrosion but under some circumstances accelerates it, thus leading to the formation of patches of rust under the paint. Under favorable conditions it makes a good protective coating. Iron oxide is mixed with boiled linseed oil, using about 8 pounds of the pigment to one gallon of oil.

The carbon paints, which include lampblack and graphite, have no cementing properties when mixed with oil. The amount of pigment used is small compared with that used in red lead paint. It, therefore, has much greater spreading power and consequently makes a much thinner film. As it does not inhibit corrosion, its protective power depends entirely on the oil, making it necessary to use several coats in order to get satisfactory results. It makes a satisfactory second coat over red lead. The carbon pigments, particularly graphite, are subject to many adulterations. There are no standard proportions. Carbon paints can be made at the factory and will keep for an indefinite period.

Prepared Paints. Many proprietary paints are offered for structural steel. Some have much merit, others none. They should not be used unless there are authentic records of successful use.

Painting Required. Structural steel in buildings is protected from moisture by being enclosed by other materials. On the other hand, in most cases it cannot be repainted, so the original painting is of great importance. The writer recommends painting it two coats, first red lead, second graphite or lampblack. If the steel is to be encased in concrete, the second coat may be omitted, the concrete furnishing as much protection as the second coat of paint.

Cleaning. The paint can have no mechanical bond to the steel so must depend on adhesion to hold it in place. This makes it necessary that the surfaces be cleaned before painting, removing all rust, dirt, grease, and mill scale. The cleaning is of utmost importance, for if not done thoroughly, the paint will not adhere; and, if rusting has already started, it may continue under the paint. It is not uncommon to find large patches of rust over which the paint remains unbroken. This is apt to occur when the surface is not properly cleaned before repainting.

The most effective way of cleaning steel is by means of the sand blast. This method is expensive and is not much used for steel work for buildings. It is used chiefly for cleaning old steel work, especially bridges, for repainting. The usual means of cleaning is by the use of the scraper, chisel, and wire brush. This work can be well done with these tools, if enough labor is expended on it.

Applying the Paint. The paint is best applied with heavy round brushes. It must be spread evenly and cover the entire surface and be worked into all corners and joints. The metal surfaces should be warm and free from moisture. In cold weather the paint should be warmed.

Surfaces in Contact. It is customary to specify that surfaces which will be in contact after assembling shall be painted before assembling. The desirability of this has been questioned on the basis that the paint is probably destroyed by the heat from the rivets. Nevertheless, there is no evidence that such painting does any harm and it is best to do it in accordance with usual practice. Box sections, such as channel columns, should have two coats on the inner surfaces before assembling.

Cement as a Rust Preventive. Portland cement mortar and concrete are inhibitors of rust and, if dense and in actual contact with the metal, provide the necessary protection against moisture. If applied to clean steel surfaces, no other protection is required. But the steel, if not painted at the shop, usually will become badly rusted before it is enclosed in the building, making it desirable that the shop coat of paint be used. Then the concrete casing will make it unnecessary to apply the second coat of paint.

PROTECTION FROM FIRE

Effects of Heat on Steel. *Expansion.* Heat applied to steel causes it to expand. Its coefficient of expansion is 0.0000067 for one degree Fahrenheit, that is, for each increase of one degree in temperature a unit of length increases by the amount of the coefficient. Thus for an increase of 100 degrees in temperature, the increase for each unit of length is $100 \times 0.0000067 = 0.00067$; for a length of 18 feet, the total increase in length is $0.00067 \times 18 = 0.01206$ feet, or .14472 inches.

There is a corresponding change in the opposite direction, if the temperature decreases. From this it is clear that expansion and contraction due to changes in temperature occur in appreciable amount. The longer the member, or series of members, the greater the change in length. Within buildings, the change in temperature ordinarily is not enough to cause trouble, but if the steel is exposed to fire, it might expand enough to push a wall out of place even though not heated enough to affect its strength. Cases have occurred where walls have been seriously displaced by ordinary changes of temperature, because the expansion of the steel pushed the wall outward, whereas the succeeding contraction did not pull it back; then the next expansion pushed it farther out, and thus by successive movements the wall was pushed farther and farther out of place.

Loss of Strength. Experiments indicate that steel can be heated to a temperature of about 600 degrees Fahrenheit before it begins to lose strength. At higher temperatures, it loses strength rapidly and will fail of its own weight at a temperature of about 1500 degrees. Steel melts at 2500 degrees (approx.).

Intensity of Heat in a Fire. The intensity of heat developed in a fire varies greatly according to conditions. Many cases are recorded showing steel bent into a tangled mass from the burning of a building, indicating temperatures of 1500 degrees or more. Such temperatures can be produced by burning the wood framework of an ordinary building, or even the contents of a fireproof building.

Protective Methods. Unprotected steel yields very quickly in a fire, much more quickly than wood beams of the same strength. It is dangerous and inexcusable to use structural steel in a building without providing for its safety. Steel is protected from fire by encasing it in a fireproof material. Almost any material encasing steel will protect it to some extent. Even a tight casing of wood will protect it for a little while in a fire. Ordinary plaster on wood lath will protect it only until the fire gets through the plaster, after which the burning of the lath aids in the destruction of the steel. Cement plaster on metal lath is efficient only to a limited degree, and while it is an incombustible material, it is not fireproof within the meaning of that term as used in building construction.

Misuse of the Term Fireproof. Many buildings are called fireproof when the protection of the steel is nothing more than described above. Instances can be cited of hotels advertised as fireproof with steel beams placed among wood joists with no protection whatever.

Amount of Protection Depends on Conditions. A building may be made entirely of incombustible material and still not be fireproof, if the steel is not encased to protect it from the contents of the

BRICK ARCH CONSTRUCTION

Fig. 209. Brick and Concrete Arch Construction Showing Partial Protection for I-Beams

building. Fig. 209 illustrates a form of construction of this sort which was much used a number of years ago. The brick arches and the concrete filling protect the beam except on the bottom flange, which is left exposed to fire from the burning of the contents of the room below. Fig. 210 is a similar form of construction in which a corrugated-steel arch replaces the brick arch. This partial protection is of some value, but it is so easy under present methods to get complete protection that these forms are no longer used.

On the other hand, a building having no combustible material

CORRUGATED IRON ARCH CONSTRUCTION

Fig. 210. Corrugated Iron and Concrete Arch Construction Showing Insufficient Protection for I-Beams

in its construction or contents, and having no external hazard, need not have its steel framework fireproofed. A foundry building or a machine shop may be such a case.

Standard Specifications. Steel to be really fireproofed must be entirely encased in a fireproof material. The material must be such that it will conduct heat very slowly and that it will maintain its integrity when subjected to a fire of the greatest instensity and longest duration likely to occur, and when subjected to a stream of water from a fire hose while at its maximum heat.

The "Standard Test for Fireproof Floor Construction" adopted by the American Society for Testing Materials* requires:

*American Society for Testing Materials, Edgar Marburg, Secretary, University of Pennsylvania, Philadelphia.

"No plastering shall be applied to the underside of the floor construction under test.

"The floor shall be subjected for four hours to the continuous heat of a fire of an average temperature of not less than 1700° F.; the fuel used being either wood or gas, so introduced as to cause an even distribution of heat throughout the test structure.

"The heat obtained shall be measured by means of standard pyrometers, under the direction of an experienced person. The type of pyrometer is immaterial so long as its accuracy is secured by proper standardization. The heat should be measured at not less than two points when the main floor span is not more than 10 feet and one additional point when it exceeds 10 feet. Temperature readings at each point are to be taken every three minutes. The heat determination shall be made at points directly beneath the floor so as to secure a fair average.

"At the end of the heat test a stream of water shall be directed against the underside of the floor, discharged through a $1\frac{1}{8}$-inch nozzle, being held at more than 3 feet from the firing door during the application of the water."

Material which will withstand this test is suitable for fireproofing steel in any part of a building.

Fireproof Materials. *Cinder Concrete.* Cinder concrete has been used extensively for fireproofing but it is not altogether satisfactory. It is difficult to get cinders free from unburned coal, ashes, and refuse. Sulphur in the cinders causes rusting of the steel. Its use is not warranted on first-class work.

Portland Cement Concrete. Portland cement concrete, made of crushed stone or gravel, is an excellent fireproofing material. It has the necessary resistance to fire and water, prevents rusting of the steel, and in many situations adds to the strength of the steel member. If its surface is left rough or is roughened after the forms are removed, plaster will stick to it.

When subjected to a fire, the concrete is damaged. The depth of the injury may be as much as $1\frac{1}{2}$ inches, depending on the quality of the concrete and the kind of stone used in it. The better the concrete, the less it is injured by the heat. Heat calcines limestone and disintegrates granite, so that these stones are not as suitable for fireproofing purposes as hard sandstones, trap, and other stones not so easily affected by heat. An excellent concrete for fireproofing can be made from crushed tile and brick. On buildings where tile is used for floor arches and partitions, the broken pieces can be crushed and used for fireproofing the columns and any other members not protected by the tile floor arches.

Concrete which has been damaged by fire does not lose its property of non-conductivity, consequently it is efficient as fireproofing so long as it remains in place; although it has lost its strength, it usually will remain in place until removed by some mechanical means, as the application of a stream of water. After a fire, the damaged concrete must be removed and replaced.

Concrete is placed around steel by building forms around the members and pouring concrete into them, Fig. 211. Wire mesh or expanded metal should be attached to the bottom flanges of beams and wrapped around columns to provide a mechanical bond for the concrete so that it will not fall off during or after a fire.

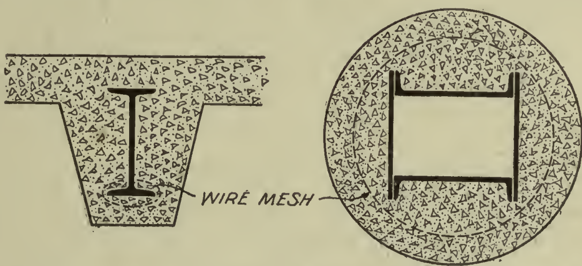


Fig. 211. I-Beam and Column Sections Showing Concrete Fireproofing.

Hollow Tile. Hollow tile is molded from clay and baked at a high temperature. The clay used must be such that it will not warp, or fuse in the kiln. It is desirable that the tile be porous and tough rather than dense and brittle. The tile is made porous by mixing sawdust with the clay. This burns out during the baking, leaving voids and producing the desired porosity. Dense tile and tile which is glazed is likely to shatter, if exposed to a stream of water when hot, thus making it useless for fire protection; furthermore, plaster does not adhere to it as well as to porous tile.

The tile is made hollow to save weight, and to provide air spaces which are insulators against both heat and moisture.

This material is molded into a great variety of shapes to suit the various requirements of the steel members to be protected. Certain shapes are practically standard; special shapes can be had only when required in large quantity.

Figs. 212, 213, 214, and 215 show a number of illustrations of tile fireproofing of joists, girders, spandrels, and columns. The joists are usually fireproofed by the skewbacks of the floor arches. On other members the tile serves only for fireproofing and for furnish-

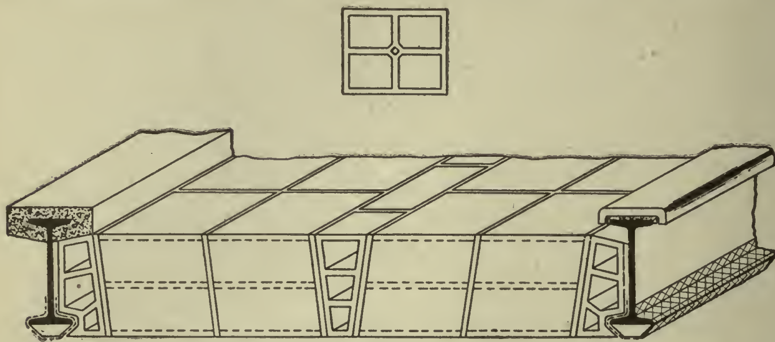


Fig. 212. Method of Fireproofiing Joists in Connection with Flat Tile Floor Arch

ing a surface for plastering. It can be used for fireproofing steel members in almost any situation.

Tile is set in mortar in the same manner as bricks are laid. Any space between the tile and the steel should be filled with Port-

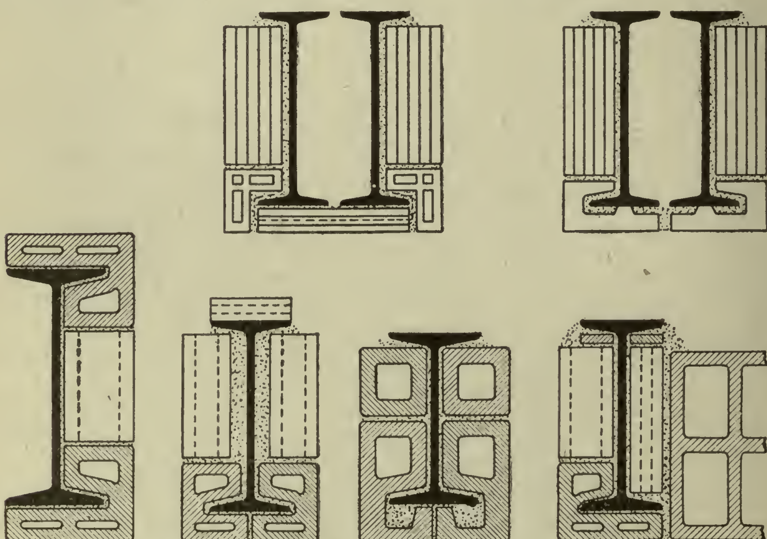


Fig. 213. Sections Showing Method of Fireproofing Beams

land cement mortar. A heavy layer of mortar should be plastered on the webs of beams before setting skewbacks or other tiles against them.

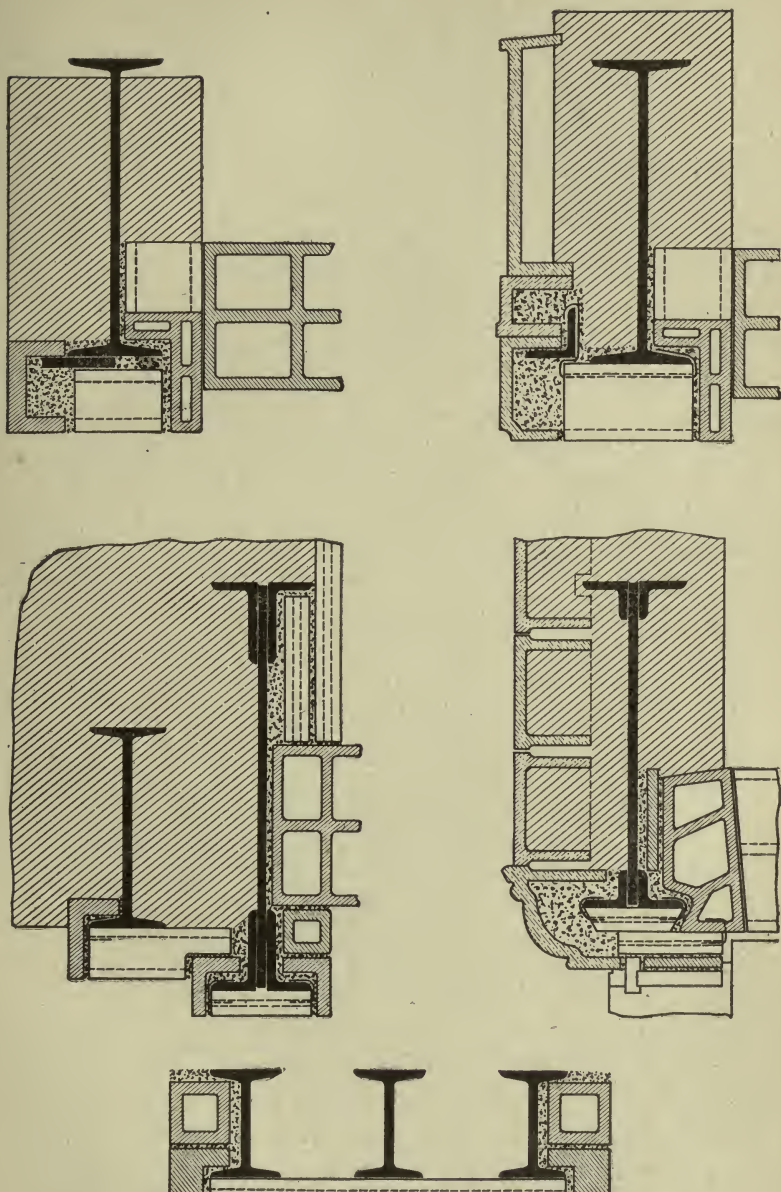


Fig. 214. Sections Showing Method of Fireproofing Spandrels

Fireproofing tile must be designed to be securely supported by the steel. Steel clips or wire must be used in some situations. Thus the column casing should be held in place by copper wire bands unless it is securely held by interlocking of the tile. Soffit tile on joists and girders require metal clips or woven wire fabric to hold them in place even though they appear to have support from shoe tile or other adjacent members.

Tile has considerable strength in compression and may be so used, but should not be subjected to other stresses.

Brick. Brick masonry is an excellent fireproofing material so far as its resistance to heat is concerned. However, it is not easily

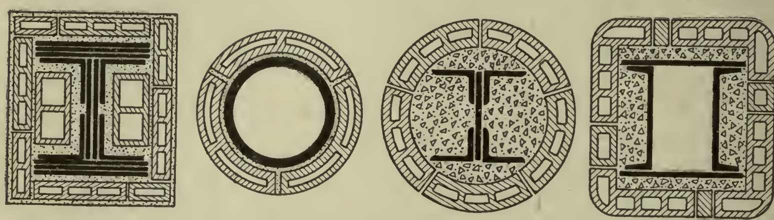


Fig. 215. Sections Showing Method of Fireproofing Columns with Tile and Concrete

supported and, therefore, is not generally available for this purpose on beams, but in some cases it can be used to good advantage for encasing columns.

Selection of Fireproofing. Portland cement concrete and hollow tile are the materials best suited for fireproofing. Both are efficient for this purpose. The choice between them is usually governed by other considerations, chief of which is the type of floor construction, which in turn may be determined by cost or some other consideration. If the floor is to be of reinforced concrete, concrete will be used for fireproofing the steel framework. If the floor is to be of tile arch construction, that material will be used for fireproofing; but even in this case concrete can be used advantageously for the columns.

Thickness of Fireproofing. The thickness of the covering required to furnish the desired protection varies with the situation and the importance of the member. Columns being vital to the support of the building are given the most protection. Lintels and spandrel girders are subject to severe exposure and are given about the same protection as columns. Joists and girders are local members and not so heavily fireproofed. The top flanges of beams and girders do not need as much protection as the bottom flanges.

The requirements in Chicago are:*

Columns—Exterior. (a) All iron or steel used as vertical supporting members of the external construction of any building exceeding fifty feet in height shall be protected against the effects of external change of temperature and of fire by a covering of fireproof material consisting of at least four inches of brick, hollow terra cotta, concrete, burnt clay tiles, or of a combination of any two of these materials, provided that their combined thickness is not less than four inches. The distance of the extreme projection of the metal, where such metal projects beyond the face of the column, shall be not less than two inches from the face of the fireproofing; provided, that the inner side of exterior columns shall be fireproofed as hereafter required for interior columns.

(b) Where stone or other incombustible material not of the type defined in this ordinance as fireproof material is used for the exterior facing of a building, the distance between the back of the facing and the extreme projection of the metal of the column proper shall be at least two inches, and the intervening space shall be filled with one of the fireproof materials.

(c) In all cases, the brick, burnt clay, tile, or terra cotta, if used as a fireproof covering, shall be bedded in cement mortar close up to the iron or steel members, and all joints shall be made full and solid.

Columns—Interior. (a) Covering of interior columns shall consist of one or more of the fireproof materials herein described.

(b) If such covering is of brick it shall be not less than four inches thick; if of concrete, not less than three inches thick; if of burnt clay tile, such covering shall be in two consecutive layers, each not less than two inches thick, each having one air space of not less than one-half inch, and in no such burnt clay tile shall the burnt clay be less than five-eighths of an inch thick; or if of porous clay solid tiles, it shall consist of at least two consecutive layers, each not less than two inches thick; or if constituted of a combination of any two of these materials, one-half of the total thickness required for each of the materials shall be applied, provided that if concrete is used for such layer it shall not be less than two inches thick.

(c) In the case of columns having an "H" shaped cross section or of columns having any other cross section with channels or chases open from base plates to cap plates on one or more sides of the columns, then the thickness of the fireproof covering may be reduced to two and one-half inches, measuring in the direction in which the flange or flanges project, and provided that the thin edge in the projecting flange or arms of the cross sections does not exceed three-quarters of an inch in thickness. The thickness of the fireproof covering on all surfaces measuring more than three-quarters of an inch wide and measuring in a direction perpendicular to such surfaces shall be not less than that specified for interior columns in the beginning of this section, and all spaces, including channels or chases between the fireproof covering and the metal of the columns, shall be filled solid with fireproof material. Lattice or other open columns shall be completely filled with approved cement concrete.

*Revised Building Ordinances of the City of Chicago as amended Feb. 20, 1911.

Columns—Wiring Clay Tile On. (a) Burnt clay tile column covering shall be secured by winding wire around the columns after the tile has all been set around such columns. The wire shall be securely wound around tile in such manner that every tile is crossed at least once by a wire. If iron or steel wire is used it shall be galvanized and no wire used shall be less than number twelve gage.

* * * * * * * *

Pipes Enclosed by Covering. (a) Pipes shall not be enclosed in the fireproofing of columns or in the fireproofing of other structural members of any fireproof building; provided, however, gas or electric light conduits not exceeding one inch diameter may be inserted in the outer three-fourths inch of the fireproofing of such structural member, where such fireproofing is entirely composed of concrete.

(b) Pipes or conduits may rest upon the tops of the steel floor beams or girders, provided they are imbedded in cinder concrete to which slaked lime equal to five per cent of the volume of concrete has been added before mixing or their being imbedded in stone concrete.

* * * * * * * *

Spandrel Beams, Girders, Lintels. The metal of the exterior side of the spandrel beams or spandrel girders of exterior walls, or lintels of exterior walls, which support a part of exterior walls, shall be covered in the same manner, and with the same material as specified for the exterior columns in this chapter; provided, however, that shelf angles connected to girders by brackets or projections of girder flanges not figured as part of the flange section may come within two inches of the face of the brick or other covering of such spandrel beams, girders, or lintels. The covering thickness shall be measured from the extreme projection of the metal in every case.

Beams, Girders and Trusses—Coverings of. (a) The metal beams, girders, and trusses of the interior structural parts of a building shall be covered by one of the fireproof materials hereinbefore specified, so applied as to be supported entirely by the beam or girder protected, and shall be held in place by the support of the flanges of such beams or girders and by the cement mortar used in setting.

(b) If the covering is of brick, it shall be not less than four inches thick; if of hollow tiles or if of solid porous tiles or if of terra cotta, such tiles shall be not less than two inches thick, applied to the metal in a bed of cement mortar; hollow tiles shall be constructed in such a manner that there shall be one air space of at least three-fourths of an inch by the width of the metal surface to be covered within such clay coverings; the minimum thickness of concrete on the bottom and sides of metal shall be two inches.

(c) The top of all beams, girders, and trusses shall be protected with not less than two inches of concrete or one inch of burnt clay bedded solid on the metal in cement mortar.

(d) In all cases of beams, girders, or trusses, in roofs or floors, the protection of the bottom flanges of the beams and girders and as much of the web of the same as is not covered by the arches shall be made as hereinbefore specified for the covering of beams and girders. In every case the thickness of the covering shall be measured from the extreme projection of the metal, and the entire space or spaces between the covering and the metal shall be filled solid with one of the fireproof materials, excepting the air spaces in hollow tile.

(e) Provided, however, that all girders or trusses when supporting loads from more than one story shall be fireproofed with two thicknesses of fireproof material or a combination of two fireproof materials as required for exterior columns, and each covering of fireproof material shall be bedded solid in cement mortar.

Fireproofing of Exterior Sides of Mullions. In buildings required by this chapter to be of fireproof construction on exposures where metal frames, doors, sash, and wire glass are not required, all vertical door or window mullions over eight inches wide shall be faced with incombustible material, and horizontal transom bars over six inches wide shall be faced with a fireproof or with an incombustible material.

* * * * * * * * *

Iron or Steel Plates for Support of Wall. Where iron or steel plates or angles are used in each story for the support of the facings of the walls of such story, such plates or angles shall be of sufficient strength to carry the weight within the limits of fiber stress for iron and steel elsewhere specified in this chapter of the enveloping material for such story, and such plates or angles may extend to within two inches of the exterior of such covering.

SPECIFICATIONS

Purpose. The purpose of specifications is to give a detailed description of such features of the work as can thus be given more clearly or be more easily defined than on drawings. They must co-operate with and supplement the drawings, but should not repeat the data given on the drawings, for every repetition is an added opportunity for conflict or error.

In addition to the technical requirements referred to above, the specifications usually include certain items more related to the business transaction between the purchaser and the contractor.

The specifications prepared by the designer are to be used for the guidance of the contractor in estimating the value of the work, of the mill in rolling the steel, of the engineer in preparing working drawings, and of the fabricating shop in manufacturing the material. These purposes should be kept in mind in writing specifications.

The relation of the specifications to the contract should be clearly understood. In all cases the specifications should be made a part of the contract and they are then just as binding as if written into the contract. This indicates the importance of having them correctly written. As far as practicable, items in the specifications should not be repeated in the contract and, on the other hand, items which belong in the contract should not be in the specifications, for such repetitions lead to conflicting or ambiguous provisions.

GENERAL CHARACTERISTICS

A number of proposed standard specifications for structural steel have been published. Usually their purpose is more for the guidance of the designer than of the contractor. Some of them cover both purposes quite fully. Such a one is "Revised General Specifications for Structural Work for Buildings" by C. C. Schneider, M. Am. Soc., C. E., published in the *Transactions of the American Society of Civil Engineers*, Vol. LIV, page 490. This is referred to as Schneider's Specifications. It can be used in whole or in part in making up specifications for a particular work. It is published and for sale by the Engineering News Publishing Company, so that copies are readily available. Consequently, in using the specifications, the parts desired need not be copied but can be referred to by subject and paragraph number. Considerable portions are quoted in the specifications given later.

When such general specifications are used, they must be supplemented to provide for the special requirements of the work and for the business features before mentioned.

Outline for Specifications. Complete specifications should include the following subjects:

Instructions to Bidders	Quality of Materials
General Conditions	Details of Construction
Scope of Work	Workmanship
Loads	Painting
Unit Stresses	Inspection

Erection

Instructions to Bidders. This is entirely a business feature and may be made a separate document from the specifications. But if so, it should accompany the specifications which are sent to bidders. As the instructions may contain items which might later affect the interpretation of the contract, it is best that they be included in the specifications and thus, automatically, become a part of the contract.

The instructions give the time and place for submitting bids, the price basis, and any other directions pertinent to the case in hand. Bidders may be required to state the length of time required by them, if this will be a consideration in letting the contract.

General Conditions. The general conditions have no very direct relation to the technical requirements but are more clearly business features. They cover such items as bonds, liability insurance, watchman service, etc.

Scope of Work. This section of the specifications is devoted to the particular work under consideration and should be most carefully stated, for it governs the amounts of material and service to be furnished. The paragraphs should cover the following items:

(a) Describe definitely the work included. If separate drawings are made for the structural steel and show completely all the material to be furnished, the work may be so described. But if the structural steel is shown on drawings with other materials, particularly ornamental or miscellaneous iron, then the description must be given in sufficient detail to make it perfectly clear. It must be understood that the term "structural steel" is not definite enough to be used without such a description as required above, for structural shapes may be used in stair construction, for furring, for window frames, and in other situations, when it is desirable that such items be furnished by other contractors. Cast-iron pedestals for steel columns and cast-iron columns, if used, are usually included in the contract with the structural steel.

(b) Identify the drawings involved by numbers and dates.

(c) State the place of delivery if erection is not included, and specify by whom transportation charges are to be paid.

(d) Give requirements as to working drawings.

Loads. It is desirable that the loads used in making the design be given in the specifications or marked on the drawings. The latter method is preferable for special loads, such as machinery, tanks, storage space, etc. This information is needed in detailing connections, stiffeners, etc. It is not sufficient to say that connections shall develop the full strength of the member, for there may be situations when a concentrated load near the end of a beam may produce a stress at the connection greater than would be produced by a uniformly distributed load.

Unit Stresses. The unit stresses concern the design of the structural steel more than they do the manufacture and construction of it. However, they are needed in making the working drawings and should be included in the specifications. Those given in Schneider's Specifications should be used unless local building ordinances require other values.

Quality of Material. The quality of material to be used is discussed at length on p. 42. The specifications of the American Society for Testing Materials are recommended for general use. They need not be written into the specifications, it being sufficient to state that the steel shall comply with the "Standard Specifications for Structural Steel for Buildings", adopted by the American Society for Testing Materials. Similarly, the quality of cast iron may be specified.

In this section the kind and quality of paint should be given.

Details of Construction. This section of the specifications is concerned with such items as connections, rivet spacing, etc. It is chiefly to guide the engineers and draftsmen in making working drawings. Design drawings should be consistent with its provisions.

Schneider's Specifications are recommended for this portion of the specifications. They may be used by reference, saying that the details of construction should conform to Schneider's Specifications in so far as they apply to this work; or the specific paragraphs which do apply may be referred to by number.

Workmanship. The specifications for workmanship govern the operations in the shop. Schneider's Specifications are recommended and may be used the same as for construction details.

Painting. This is well covered by Schneider's Specifications, which may be used without modification unless some special provision is to be inserted.

Inspection and Tests. Schneider's Specifications are used for this part of the work without change.

Erection. The specifications for erection must deal with the specific job. However, some of its provisions are general.

The conditions at the site, the relations to the other parts of the structure, order of procedure, storage available, etc., etc., must be written to suit each case. If the contract for erection is separate from the contract for furnishing the steel, the division between them must be clearly defined. This division is usually best made at the place where the material is delivered on board cars.

Quality of workmanship of erection applies generally to all structures.

EXAMPLE OF SPECIFICATIONS

The following specifications accord with the preceding discussions and may be used as a guide in writing specifications for a particular structure.

SPECIFICATIONS

for the

Structural Steel and Iron

for a

(Kind of Building)

for

(Owner)

Instructions to Bidders. Bids will be received for the structural steel and iron work required for (kind of building) located at .... Street, in the City of ....for the ....(owner)............ in accordance with the following specifications and the plans described therein.

Bids must be filed at the office of.. Architect, on or before noon,.................................. 19.....

Bidders shall state a lump sum which shall include furnishing, delivering, and erecting the structural steel and iron work and shall also include the cost of the bond, insurance, and watch service as required under general conditions.

General Conditions.

Ownership. The building is known as the......................... Building and is owned by the....................................[a partnership (or corporation) existing under the laws of the State of.........................]

Location. It is located at.. Street in the City of................................on lots.......................(give legal description).

Bond. The contractor shall furnish surety bond in the penal sum of one-half the contract price, guaranteeing the fulfillment of the terms of the contract. Said bond shall be in terms and with surety satisfactory to the Architect.

Liability Insurance. The contractor shall protect the owner against loss due to any damage to property or injury to persons which may result from his operations. He shall provide adequate liability insurance in a company approved by the Architect.

Patented Articles. The contractor shall protect the owner against any claim arising out of the use of any patented article, appliance, or method.

Protection. The contractor shall provide such barricades, scaffolding, staging, and other means of protection as may be required to comply with the state and municipal laws and to adequately safeguard property and persons.

Watchmen. The contractor shall keep competent watchmen on the building day and night.

Scope. [Give a general description similar to the following: The building is designed for office purposes with stores on the first and second floors. It is twenty stories high above street level with a basement and sub-basement below street level. The ground area occupied is approximately 100 feet by 162 feet.]

Work Covered. The work to be done under the specifications is the furnishing and the erecting of the structural steel and iron work. The contractor shall make the working drawings, furnish and fabricate the material, pay all transportation charges, assemble the material in place in the building, rivet the connections, and furnish the materials and labor for shop and field painting.

Materials Included. The structural steel and iron work consists of the following items: (To be changed to suit the case).

Grillage Beams and Girders
Cast-Iron Pedestals
I-Beam Reinforcement in Retaining Walls
Structural Steel Framework
Cast-Iron Columns
Detached Lintels
Cornice Brackets
Roofing Tees
Steel Chimney
All minor parts belonging to the above items

It includes all the material of the above character shown in the structural plans of the building and, in addition, it includes the detached lintels over exterior windows which are shown on the architectural plans.

Materials Not Included. The structural steel and iron work (to be changed to suit the case) does not include the angles, channels, and hangers of the suspended ceiling over the top story, the elevator sheave beams, the beams and channels for the stairs other than those shown on the framing plans, the marquise framing, the steel column guards and door guards in the shipping room, and other like items shown on the architectural plans. It does not include the rods for reinforced concrete work shown on the structural plans except certain items which are definitely marked on the drawings to be furnished with the structural steel.

Plans. The structural plans consist of drawings prepared by .... Structural Engineer for ....Architect, as follows:

(Give list of drawings)

....

....

The architectural plans prepared by.................................... Architect, which show structural steel and iron work not given on the structural plans are drawings No.............................

While making the working drawings, the contractor shall consult all architectural drawings which may be supplied to him, for the purpose of discovering discrepancies, making necessary allowances for clearance, providing connections and supports for other materials, etc.

When provision must be made for attaching other materials to the structural steel work, the contractor shall furnish the holes required. If the necessary data are not given on the structural or architectural drawings, he shall apply to the Architect for the data before completing the working drawings. This applies particularly to stone, terra cotta, concrete, miscellaneous iron, ornamental iron, furring (wood and steel), pipes, and conduits.

Working Drawings. The contractor is required to prepare working drawings to supplement the design drawings prepared by the Engineer and the Architect. Two copies of such drawings shall be submitted to the Architect for approval. After approval, three copies shall be furnished to the Architect for his files, and as many copies as may be required shall be furnished to the inspector and to other trades.

Copies or prints of drawings issued before approval shall be marked "Not Approved" and those issued after approval shall be marked "Approved Drawing." During the preparation of the working drawings, the contractor shall examine the design drawings carefully for omissions and errors, and when such omissions and errors are discovered, he shall submit them to the Architect for correction. Figured dimensions only shall be used.

If the contractor does not have a force of engineers competent to prepare working drawings to the satisfaction of the Architect, he shall employ a consulting engineer for that purpose.

Working drawings shall be accompanied by erection diagrams and a complete index giving marking numbers of the material and page or sheet numbers of the drawings.

Approval of Working Drawings. If the working drawings are found to be consistent with the design drawings and these specifications, and if the details shown on them are satisfactory, they will be approved. One copy so marked will be returned to the contractor. If not consistent and satisfactory as above, one copy will be marked to indicate the required changes and returned to the contractor, who shall then make the required changes, and if so ordered, shall submit copies of revised drawings for final approval.

The Architect's approval will cover the arrangement of the principal members and auxiliary members, and the strength of connections. At the same time an effort will be made to discover any errors in sizes of material, in general dimensions, and in detail dimensions; but the responsibility for these items shall remain with the contractor.

The manufacturing of any material or the performing of any work before approval of working drawings will be entirely at the risk of the contractor.

Transportation. The contractor shall pay all costs of transportation of material from his shop to the building site and shall assume all risk of loss and damage in transit.

Loads. The structural steel and iron work is designed to support the estimated dead loads and the assumed live loads. In making the working drawings, the contractor shall design all connections to carry the same loads.

The dead loads are the actual weights of all materials of construction in the positions which they occupy, except that the effect of movable partitions may be assumed to be equivalent to a uniformly distributed load of 25 pounds per square foot of floor on all office floors. On other floors and along corridors, the partitions shall be provided for where they occur.

The live loads for which this structure is designed are:

(Subject to change)

Roof	50 lb. per sq. ft.
Office floor	50 lb. per sq. ft.
Second floor	100 lb. per sq. ft.
First floor	125 lb. per sq. ft.
Sidewalk	150 lb. per sq. ft.
Wagon space and shipping room	250 lb. per sq. ft.

The special loads from elevators, tanks, etc., are marked on the drawings.

The framework is designed for a wind pressure of 20 pounds per square foot applied horizontally to the vertical projection of the building in any direction.

Where stresses are marked on the drawings, they may be used as the full effect of the loads.

Beams and girders shall have their connections made strong enough to develop the full capacity of the members when they are uniformly loaded, even when the live and dead loads are less than this capacity.

Unit Stresses. The design is based on the unit stresses given in Schneider's Specifications, * paragraphs 19 to 34 inclusive. These unit stresses shall be used in proportioning the details.

Steel

19. *Permissible Strains.* All parts of the structure shall be proportioned so that the sum of the dead and live loads, together with the impact, if any, shall not cause the strains to exceed those given in the following table:

*"Revised Specifications for Structural Work for Buildings" by C. C. Schneider, M. Am. Soc. C. E., *Trans. Am. Soc. C. E.*, Vol. LIV, Page 494.

	Pounds per square inch
Tension, net section	16,000
Direct compression	16,000
Shear, on rivets and pins	12,000
Shear, on bolts and field rivets	9,000
Shear, on plate-girder web (gross section)	10,000
Bearing pressure, on pins and rivets	24,000
Bearing pressure, on bolts and field rivets	18,000
Fiber strain, on pins	24,000

20. Permissible Compression Strains. For compression members, the permissible strain of 16,000 lb. per sq. in. shall be reduced by the following formula:

$$p = 16{,}000 - 70\frac{l}{r}$$

Where p = permissible working strain per square inch in compression;
l = length of piece, in inches, from center to center of connections;
r = least radius of gyration of the section, in inches.

21. For wind bracing, and the combined strains due to wind and the other loading, the permissible working strains may be increased 25%, or to 20,000 lb. for direct compression or tension.

22. Provision for Eccentric Loading. In proportioning columns, provision must be made for eccentric loading.

23. Expansion Rollers. The pressure per linear inch on expansion rollers shall not exceed 600 d, where d = diameter of rollers, in inches.

24. Combined Strains. Members subject to the action of both axial and bending strains shall be proportioned so that the greatest fiber strain will not exceed the allowed limits for the axial tension or compression in that member.

25. Reversal of Strains. Members subject to reversal of strains shall be proportioned for the strain giving the largest section, but their connections shall be proportioned for the sum of the strains.

26. Net Sections. Net sections must be used in calculating tension members, and in deducting the rivet holes; they must be taken $\frac{1}{8}$ in. larger than the nominal size of the rivets.

27. Pin-connected riveted tension members shall have a net section through the pin holes 25% in excess of the net section of the body of the member. The net section back of the pin hole shall be at least 0.75 of the net section through the pin hole.

28. Compression Members Limiting Length. No compression member shall have a length exceeding 125 times its least radius of gyration, except those for wind and lateral bracing, which may have a length not exceeding 150 times the least radius of gyration.

29. Plate Girders. Plate girders shall be proportioned on the assumption that one-eighth of the gross area of the web is available as flange area. The compression flange shall have at least the same sectional area as the tension flange, but the unsupported length of the flange shall not exceed 16 times its width.

30. In plate girders used as crane runways, if the unsupported length of the compression flange exceeds 12 times its width, the flange shall be figured as a column between the points of support.

31. Web Stiffeners. The web shall have stiffeners at the ends and inner edges of bearing plates, and at all points of concentrated loads, and also at intermediate points, when the thickness of the web is less than one-sixtieth of the unsupported distance between flange angles, generally not farther apart than the depth of the full web plate, with a minimum limit of 5 feet.

32. Rolled Beams. I-beams, and channels used as beams or girders, shall be proportioned by their moments of inertia.

33. Limiting Depth of Beams and Girders. The depth of rolled beams in floors shall be not less than one-twentieth of the span and, if used as roof purlins, not less than one-thirtieth of the span.

In case of floors subject to shocks and vibrations, the depth of beams and girders shall be limited to one-fifteenth of the span. If shallower beams are used, the sectional area shall be increased until the maximum deflection is not greater than that of a beam having a depth of one-fifteenth of the span, but the depth of such beams shall in no case be less than one-twentieth of the span.

Cast Iron

34. Permissible Strains. Compression.............12,000 lb. per sq. in.
Tension................ 2,500 " " " "
Shear................... 1,500 " " " "

Quality of Materials. *Steel.* The structural steel shapes, plates, and rivets shall conform to the Standard Specifications for Structural Steel for Buildings adopted by the American Society for Testing Materials*, as follows:

SPECIFICATIONS FOR STRUCTURAL STEEL FOR BUILDINGS

Structural steel may be made by either the open-hearth or Bessemer process.

Rivet steel and plate or angle material over $\frac{3}{4}$ inch thick, which is to be punched, shall be made by the open-hearth process.

The chemical and physical properties shall conform to the limits shown in the tabular matter on the following page.

For the purposes of these specifications, the yield point shall be determined by the careful observation of the drop of the beam or halt in the gage of the testing machine.

In order to determine if the material conforms to the chemical limitations prescribed * * * * * * * analysis shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt or blow of steel, and a correct copy of such analysis furnished to the engineer or his inspector.

Specimens for tensile and bending tests shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig. 1 (see Fig. 46); or with both edges parallel; or they may be turned to a diameter of $\frac{3}{4}$ inch for a length of at least 9 inches, with enlarged ends.

(*a*) For material more than $\frac{3}{4}$ inch thick the bending test specimen may be 1 inch by $\frac{1}{2}$ inch in section.

(*b*) Rivet rounds and small rolled bars shall be tested as rolled.

*American Society for Testing Materials, Edgar Marburg, Secretary, University of Pennsylvania, Philadelphia

Properties of Structural Steel

Properties Considered	Structural Steel	Rivet Steel, Open Hearth
Phosphorus, max., Bessemer..........	0.10 per cent	
Phosphorus, max., open hearth	0.06 per cent	0.06 per cent
Ult. tensile strength, pounds per sq. in. .	55,000–65,000	48,000–58,000
Yield point........................	$\frac{1}{2}$ Ult. tens. str.	$\frac{1}{2}$ Ult. tens. str.
Elongation, min. per cent in 8 in........	$\frac{1,400,000}{\text{Ult. tens. str.}}$	$\frac{1,400,000}{\text{Ult. tens. str.}}$
Character of fracture.................	Silky	Silky
Cold bend without fracture............	180° to diameter of 1 thickness	180° flat

Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimens for tensile tests, representing such material, shall be cut from properly annealed or similarly treated short lengths of the full section of the bar.

At least one tensile and one bending test shall be made from each melt or blow of steel as rolled. In case steel differing $\frac{3}{8}$ inch and more in thickness is rolled from one melt or blow, a test shall be made from the thickest and thinnest material rolled. Should either of these test specimens develop flaws, or should the tensile test specimen break outside of the middle third of its gaged length, it may be discarded and another test specimen substituted therefor. If tensile test specimen does not meet the specification, additional tests may be made.

(*c*) The bending test may be made by pressure or by blows.

For material less than $\frac{5}{16}$ inch and more than $\frac{3}{4}$ inch in thickness, the following modifications shall be made in the requirements for elongation.

(*d*) For each increase of $\frac{1}{8}$ inch in thickness above $\frac{3}{4}$ inch, a deduction of 1 shall be made from the specified percentage of elongation.

(*e*) For each decrease of $\frac{1}{16}$ inch in thickness below $\frac{5}{16}$ inch, a deduction of $2\frac{1}{2}$ shall be made from the specified percentage of elongation.

(*f*) For pins, the required percentage of elongation shall be 5 less than that specified * * * * * as determined on a test specimen, the center of which shall be 1 inch from the surface.

Finished material must be free from injurious seams, flaws, or cracks, and have a workmanlike finish.

Test specimens and every finished piece of steel shall be stamped with melt or blow number, except that small pieces may be shipped in bundles securely wired together, with the melt or blow number on a metal tag attached.

A variation in cross section or weight of each piece of steel of more than $2\frac{1}{2}$ per cent from that specified will be sufficient cause for rejection, except in case of sheared plates, which will be covered by the following permissible variations, which are to apply to single plates.

When Ordered to Weight

Plates $12\frac{1}{2}$ pounds per square foot or heavier:

(*g*) Up to 100 inches wide, $2\frac{1}{2}$ per cent above or below the prescribed weight.

(*h*) 100 inches wide and over, 5 per cent above or below.

Plates under $12\frac{1}{2}$ pounds per square foot:

(*i*) Up to 75 inches wide, $2\frac{1}{2}$ per cent above or below. 75 inches and up to 100 inches wide, 5 per cent above or 3 per cent below.

(*j*) 100 inches wide and over, 10 per cent above or 3 per cent below.

When Ordered to Gage

Plates will be accepted if they measure not more than 0.01 inch below the ordered thickness.

An excess over the nominal weight corresponding to the dimensions on the order will be allowed for each plate, if not more than that shown in the following tables, one cubic inch of rolled steel being assumed to weigh 0.2833 pound.

Plates $\frac{1}{4}$ inch and over in thickness

Thickness Ordered. Inches	Nominal Weights Lb. per sq. ft.	Width of Plate			
		Up to 75 in.	75 in. and up to 100 in.	100 in. and up to 115 in.	Over 115 in.
1-4	10.20	10 per cent	14 per cent	18 per cent	
5-16	12.75	8 per cent	12 per cent	16 per cent	
3-8	15.30	7 per cent	10 per cent	13 per cent	17 per cent
7-16	17.85	6 per cent	8 per cent	10 per cent	13 per cent
1-2	20.40	5 per cent	7 per cent	9 per cent	12 per cent
9-16	22.95	$4\frac{1}{2}$ per cent	$6\frac{1}{2}$ per cent	$8\frac{1}{2}$ per cent	11 per cent
5-8	25.50	4 per cent	6 per cent	8 per cent	10 per cent
Over 5-8	.....	$3\frac{1}{2}$ per cent	5 per cent	$6\frac{1}{2}$ per cent	9 per cent

Plates under $\frac{1}{4}$ inch in thickness

Thickness Ordered Inches	Nominal Weights Lb. per sq. ft.	Width of Plate		
		Up to 50 in.	50 in. and up to 70 in.	Over 70 in.
1-8 up to 5-32	5.10 to 6.37	10 per cent	15 per cent	20 per cent
5-32 up to 3-16	6.37 to 7.65	$8\frac{1}{2}$ per cent	$12\frac{1}{2}$ per cent	17 per cent
3-16 up to 1-4	7.65 to 10.20	7 per cent	10 per cent	15 per cent

The inspector representing the purchaser shall have all reasonable facilities afforded to him by the manufacturer to satisfy him that the finished material is furnished in accordance with these specifications.

All tests and inspections shall be made at the place of manufacture, prior to shipment.

Cast Iron. The cast iron shall conform to the Standard Specifications for Gray Iron Castings adopted by the American Society for Testing Materials*, as follows:

*American Society for Testing Materials, Edgar Marburg, Secretary, University of Pennsylvania, Philadelphia.

SPECIFICATIONS FOR GRAY IRON CASTINGS

Unless furnace iron is specified, all gray castings are understood to be made by the cupola process.

The sulphur contents to be as follows:

Light castings	not over 0.08 per cent
Medium castings	not over 0.10 per cent
Heavy casting	not over 0.12 per cent

In dividing castings into light, medium, and heavy classes, the following standards have been adopted:

Castings having any section less than ½-inch thick shall be known as *light castings*.

Castings in which no section is less than 2 inches thick shall be known as *heavy castings*.

Medium castings are those not included in the above classification.

Transverse Test. The minimum breaking strength of the "Arbitration Bar" under transverse load shall not be under:

Light castings	2,500 lb.
Medium castings	2,900 lb.
Heavy castings	3,300 lb.

In no case shall the deflection be under 0.10 inch.

Tensile Test. Where specified, this shall not run less than:

Light castings	18,000 lb. per sq. in.
Medium castings	21,000 lb. per sq. in.
Heavy castings	24,000 lb. per sq. in.

The quality of the iron going into castings under specification shall be determined by means of the "Arbitration Bar". This is a bar 1¼ inches in diameter and 15 inches long. It shall be prepared as stated further on and tested transversely. The tensile test is not recommended, but in case it is called for, the bar as shown in Fig. 1, (figure not given) and turned up from any of the broken pieces of the transverse test shall be used. The expense of the tensile test shall fall on the purchaser.

Two sets of two bars shall be cast from each heat, one set from the first and the other set from the last iron going into the castings. Where the heat exceeds twenty tons, an additional set of two bars shall be cast for each twenty tons or fraction thereof above this amount. In case of a change of mixture during the heat, one set of two bars shall also be cast for every mixture other than the regular one. Each set of two bars is to go into a single mold. The bars shall not be rumbled or otherwise treated, being simply brushed off before testing.

The transverse test shall be made on all the bars cast, with supports 12 inches apart, load applied at the middle, and the deflection at rupture noted. One bar of every two of each set made must fulfil the requirements to permit acceptance of the castings represented.

The mold for the bars is shown in Fig. 2 (figure not given.). The bottom of the bar is $\frac{1}{16}$ inch smaller in diameter than the top, to allow for draft and for the strain of pouring. The pattern shall not be rapped before withdrawing. The flask is to be rammed up with green molding sand, a little damper than usual, well mixed and put through a No. 8 sieve, with a mixture of one to twelve bituminous facing. The mold shall be rammed evenly and fairly hard, thoroughly dried, and not cast until it is cold. The test bar shall not be removed from the mold until cold enough to be handled.

The rate of application of the load shall be from 20 to 40 seconds for a deflection of 0.10 inch.

Borings from the broken pieces of the "Arbitration Bar" shall be used for the sulphur determinations. One determination for each mold made shall be required. In case of dispute, the standards of the American Foundrymen's Association shall be used for comparison.

Castings shall be true to pattern, free from cracks, flaws, and excessive shrinkage. In other respects they shall conform to whatever points may be specially agreed upon.

The inspector shall have reasonable facilities afforded him by the manufacturer to satisfy him that the finished material is furnished in accordance with these specifications. All tests and inspections shall, as far as possible, be made at the place of manufacture prior to shipment.

Paint. The paints used shall be red lead paint for the shop coat and graphite paint for the field coat.

The red lead paint shall be made of red lead containing not less than 95 per cent Pb_3O_4, for the pigment and pure raw linseed oil with not more than 8 per cent of turpentine or Japan drier for the vehicle.

The red lead paint shall be mixed on the premises where it is used, and each batch shall be used within twenty-four hours after being mixed. The mixing shall be done in a churn or other mechanical mixer. The material shall be used in the proportion of twenty-five pounds of red lead to one gallon of oil.

The contractor shall furnish samples of the lead and oil for testing, and if required to do so shall furnish the name of the manufacturer of the oil and of the dealers who have handled it.

The graphite shall be the... brand manufactured by the...Company, or any other graphite paint of equal quality, if it is approved by the Architect.

The contractor shall furnish samples of the graphite paint for analysis and test. He shall guarantee that the paint will fulfill all the published claims made for it by its manufacturer.

Details of Construction. The details of construction shall conform to paragraphs 37 to 81, inclusive, of Schneider's Specifications, in so far as their provisions are applicable to this work.

37. Minimum Thickness of Material. No steel of less than $\frac{1}{4}$ in. thickness shall be used, except for lining or filling vacant spaces.

38. Adjustable Members. Adjustable members in any part of structures shall preferably be avoided.

39. Symmetrical Sections. Sections shall preferably be made symmetrical.

40. Connections. The strength of connections shall be sufficient to develop the full strength of the member.

41. No connection, except lattice bars, shall have less than two rivets.

42. Floor Beams. Floor beams shall generally be rolled steel beams.

43. For fireproof floors, they shall generally be tied with tie-rods at intervals not exceeding eight times the depth of the beams. This spacing may be increased for floors which are not of the arch type of construction. Holes for tie-rods, where the construction of the floor permits, shall be spaced about 3 in. above the bottom of the beam.

44. Beam Girder. When more than one rolled beam is used to form a girder, they shall be connected by bolts and separators at intervals of not more than 5 ft. All beams having a depth of 12 in. and more shall have at least two bolts to each separator.

45. Wall Ends of Beams and Girders. Wall ends of a sufficient number of joists and girders shall be anchored securely to impart rigidity to the structure.

46. Wall Plates and Column Bases. Wall plates and column bases shall be constructed so that the load will be well distributed over the entire bearing. If they do not get the full bearing on the masonry, the deficiency shall be made good with Portland cement mortar.

47. Floor Girders. The floor girders may be rolled beams or plate girders; they shall preferably be riveted or bolted to columns by means of connection angles. Shelf angles or other support may be provided for convenience during erection.

48. Flange Plates. The flange plates of all girders shall be limited in width, so as not to extend, beyond the outer line of rivets connecting them to the angles, more than 6 in., or more than eight times the thickness of the thinnest plate.

49. Web Stiffeners. Web stiffeners shall be in pairs, and shall have a close bearing against the flange angles. Those over the end bearing, or forming the connection between girder and column, shall be on fillers. Intermediate stiffeners may be on fillers or crimped over the flange angles. The rivet pitch in stiffeners shall not be more than 5 in.

50. Web Splices. Web plates of girders must be spliced at all points by a plate on each side of the web, capable of transmitting the full strain through splice rivets.

51. Columns. Columns shall be designed so as to provide for effective connections of floor beams, girders, or brackets.

They shall preferably be continuous over several stories.

52. Column Splices. The splices shall be strong enough to resist the bending strain and make the columns practically continuous for their whole length.

53. Trusses. Trusses shall preferably be riveted structures. Heavy trusses of long span, where the riveted field connections would become unwieldy, or for other good reasons, may be designed as pin-connected structures.

54. Intersecting Members. Main members of trusses shall be designed so that the neutral axes of intersecting members shall meet in a common point.

55. *Roof Trusses.* Roof trusses shall be braced in pairs in the plane of the chords.

Purlins shall be made of shapes, or riveted-up plate, or lattice girders.

Trussed purlins will not be allowed.

56. *Eyebars.* The eyebars in pin-connected trusses composing a member shall be as nearly parallel to the axis of the truss as possible.

57. *Spacing of Rivets.* The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for $\frac{7}{8}$-in. rivets, $2\frac{1}{2}$ in. for $\frac{3}{4}$-in. rivets, $2\frac{1}{4}$ in. for $\frac{5}{8}$-in. rivets, and $1\frac{3}{4}$ in. for $\frac{1}{2}$-in. rivets.

58. For angles with two gage lines, with rivets staggered, the maximum in each line shall be twice as great as given in Paragraph 57, and, where two or more plates are used in contact, rivets not more than 12 in. apart in any direction shall be used to hold the plates together.

59. The pitch of the rivet, in the direction of the strain, shall not exceed 6 in., nor 16 times the thinnest outside plate connected, and not more than 50 times that thickness at right angles to the strain.

60. *Edge Distance.* The minimum distance from the center of any rivet hole to a sheared edge shall be $1\frac{1}{2}$ in. for $\frac{7}{8}$-in. rivets, $1\frac{1}{4}$ in. for $\frac{3}{4}$-in. rivets, $1\frac{1}{8}$ in. for $\frac{5}{8}$-in. rivets, and 1 in. for $\frac{1}{2}$-in. rivets; and to a rolled edge, $1\frac{1}{4}$, $1\frac{1}{8}$, 1, and $\frac{7}{8}$ in., respectively.

61. The maximum distance from any edge shall be eight times the thickness of the plate.

62. *Maximum Diameter.* The diameter of the rivets in any angle carrying calculated strains shall not exceed one-quarter of the width of the leg in which they are driven. In minor parts, rivets may be $\frac{1}{8}$ in. greater in diameter.

63. *Pitch at Ends.* The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets for a length equal to one and one-half times the maximum width of the member.

64. *Tie Plates.* The open sides of compression members shall be provided with lattice having tie plates at each end at intermediate points where the lattice is interrupted. The tie plates shall be as near the ends as practicable. In main members, carrying calculated strains, the end tie plates shall have a length not less than the distance between the lines of rivets connecting them to the flanges, and intermediate ones not less than half this distance.

Their thickness shall be not less than one-fiftieth of the same distance.

65. *Lattice.* The thickness of lattice bars shall be not less than one-fortieth for single lattice and one-sixtieth for double lattice, of the distance between end rivets; their minimum width shall be as follows:

For 15-in. channels, or built sections with $3\frac{1}{2}$ and 4-in. angles.......	$2\frac{1}{2}$ in. ($\frac{7}{8}$-in. rivets)
For 12-, 10- and 9-in. channels, or built sections with 3-in. angles.	$2\frac{1}{4}$ in. ($\frac{3}{4}$-in. rivets)
For 8- and 7-in. channels, or built sections with $2\frac{1}{2}$-in. angles.	2 in. ($\frac{5}{8}$-in. rivets)
For 6- and 5-in. channels, or built sections with 2-in. angles.	$1\frac{3}{4}$ in. ($\frac{1}{2}$-in. rivets)

66. Lattice bars with two rivets shall generally be used in flanges more than 5 in. wide.

67. Angle of Lattice. The inclination of lattice bars with the axis of the member, generally, shall be not less than 45°, and when the distance between the rivet lines in the flange is more than 15 in., if a single rivet bar is used, the lattice shall be double and riveted at the intersection.

68. Spacing of Lattice. The pitch of lattice connections, along the flange divided by the least radius of gyration of the member between connections, shall be less than the corresponding ratio of the member as a whole.

69. Faced Joints. Abutting joints in compression members when faced for bearing shall be spliced sufficiently to hold the connecting members accurately in place.

70. All other joints in riveted work, whether in tension or compression, shall be fully spliced.

71. Pin Plates. Pin holes shall be reinforced by plates where necessary; and at least one plate shall be as wide as the flange will allow; where angles are used, this plate shall be on the same side as the angles. The plates shall contain sufficient rivets to distribute their portion of the pin pressure to the full cross section of the member.

72. Pins. Pins shall be long enough to insure a full bearing of all parts connected upon the turned-down body of the pin.

73. Members packed on pins shall be held against lateral movement.

74. Bolts. Where members are connected by bolts, the body of these bolts shall be long enough to extend through the metal. A washer at least $\frac{3}{16}$ in. thick shall be used under the nut.

75. Fillers. Fillers between parts carrying strain shall have a sufficient number of independent rivets to transmit the strain to the member to which the filler is attached.

76. Temperature. Provision shall be made for expansion and contraction, corresponding to a variation of temperature of 150° Fahr., where necessary.

77. Rollers. Expansion rollers shall be not less than 4 in. in diameter.

78. Stone Bolts. Stone bolts shall extend not less than 4 in. into granite pedestals and 8 in. into other material.

79. Anchorage. Columns which are strained in tension at their base shall be anchored to the foundations.

80. Anchor bolts shall be long enough to engage a mass of masonry. the weight of which shall be one and one-half times the tension in the anchor.

81. Bracing. Lateral, longitudinal, and transverse bracing in all structures shall preferably be composed of rigid members.

Adjacent ends of column sections, which do not have full bearing, shall have bearing plates not less than $\frac{3}{8}$ inch thick.

Rivets generally shall be $\frac{3}{4}$ inch in diameter, but the diameter of the rivet shall not be less than one-fourth of its grip; $\frac{7}{8}$-inch rivets shall be used when the pieces connected are $\frac{3}{4}$ inch or more in thickness.

No beam connections shall be less than the standards of the American Bridge Company.

The clearance from the ends of beams to columns or to girders shall not exceed $\frac{1}{2}$ inch.

Tie-rods between floor beams shall be threaded at both ends for a length of at least 3 inches.

The number of rivets furnished for field connections shall be 10 per cent in excess of the nominal number required.

Chimney. The connections of the cast-iron or steel chimney to the framework shall be such as to permit expansion and contraction, due to changes in temperature.

Provide flanges with holes for breeching connection.

Cast-iron chimneys may have either flanged joints or hub and spigot joints. The bearing surfaces shall have contact on the entire perimeter and shall be exactly at right angles to the axis of the pipe, being turned or planed, if necessary to make them so. The calking space in hub and spigot joints shall be filled with iron fillings and sal ammoniac and calked solid. Connections for anchors shall be cast on.

Steel chimneys shall have lap joints for all shop connections. They may have either lap or flange joints for the field connections, except that the lap joints generally will be required for self-supporting chimneys exposed to wind pressure. All joints shall be practically air-tight and, if not so made by the riveting, shall be calked.

Cast Iron. The ends of cast-iron columns and the tops of cast-iron base plates and pedestals shall be planed.

Bolt holes in cast iron shall be drilled. Holes for grout may be cored.

In each cast-iron pedestal a grout hole shall be provided which shall be not less than $2\frac{1}{2}$ inches in diameter and placed as near the center of the base as practicable. Additional holes shall be provided in bases larger than 4 feet in diameter.

The joints in cast-iron columns shall be made by means of flanges cast on the columns. Each joint shall be bolted with not less than four $\frac{3}{4}$-inch bolts. The metal in the flanges shall be not less than 1 inch thick.

Unless otherwise designed, each beam connection shall consist of a bracket and a lug. The bracket shall sustain the entire reaction from the beam. It shall project not less than 4 inches from the column and shall slope $\frac{1}{8}$ inch. The lug shall provide for two or more bolts connecting to the web of the beam.

Workmanship. The workmanship in the fabrication of the structural steel shall conform to paragraphs 23 to 51 of Schneider's Specifications, in so far as they concern this work.

23. General. All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge work.

24. Straightening Material. Material shall be thoroughly straightened in the shop, by methods which will not injure it, before being laid off or worked in any way.

25. Finish. Shearing shall be done neatly and accurately, and all portions of the work exposed to view shall be neatly finished.

26. Rivets. The size of rivets called for on the plans shall be understood to mean the actual size of the cold rivet before heating.

27. Rivet Holes. The diameter of the punch for material not more than $\frac{5}{8}$ in. thick shall be not more than $\frac{1}{16}$ in., nor that of the die more than $\frac{1}{8}$ in. larger than the diameter of the rivet. Material more than $\frac{5}{8}$ in. thick, excepting in minor details, shall be sub-punched and reamed or drilled from the solid.

28. Punching. Punching shall be done accurately. Slight inaccuracy in the matching of holes may be corrected with reamers. Drifting to enlarge unfair holes will not be allowed. Poor matching of holes will be cause for rejection, at the option of the inspector.

29. Assembling. Riveted members shall have all parts well pinned up and firmly drawn together with bolts before riveting is commenced. Contact surfaces shall be painted. (See Paragraph 52.)

30. Lattice Bars. Lattice bars shall have neatly-rounded ends, unless otherwise called for.

31. Web Stiffeners. Stiffeners shall fit neatly between the flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

32. Splice Plates and Fillers. Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

33. Connection Angles. Connection angles for floor girders shall be flush with each other and correct as to position and length of girder.

34. Riveting. Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

35. Rivets. Rivets shall look neat and finished, with heads of approved shape, full, and of equal size. They shall be central on the shank and shall grip the assembled pieces firmly. Re-cupping and calking will not be allowed. Loose, burned, or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjoining metal. If necessary, they shall be drilled out.

36. Field Bolts. Wherever bolts are used in place of rivets which transmit shear, such bolts must have a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under the nut.

37. Members to be Straight. The several pieces forming one built member shall be straight and shall fit closely together, and finished members shall be free from twists, bends, or open joints.

38. Finish of Joints. Abutting joints shall be cut or dressed true and straight and fitted closely together, especially where open to view. In compression joints depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

39. Eyebars. Eyebars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling, or forging. Welding will not be allowed. The form of the heads will be determined by the dies in use at the works where the eyebars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body with a silky fracture, when tested to rupture. The thickness of the head and neck shall not vary more than $\frac{1}{16}$ in. from the thickness of the bar.

40. Boring Eyebars. Before boring, each eyebar shall be perfectly annealed and carefully straightened. Pin holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin holes can be passed through the holes at both ends of the bars at the same time.

41. Pin Holes. Pin holes shall be bored true to gages, smooth and straight; at right angles to the axis of the member, and parallel to each other, unless otherwise called for. Wherever possible, the boring shall be done after the member is riveted up.

42. Variation in Pin Holes. The distance from center to center of pin holes shall be correct within $\frac{1}{32}$ in., and the diameter of the hole not more than $\frac{1}{50}$ in. larger than that of the pin, for pins up to 5 in. diameter, and $\frac{1}{32}$ in. for larger pins.

43. Pins and Rollers. Pins and rollers shall be turned accurately to gages, and shall be straight, smooth, and entirely free from flaws.

44. Pilot Nuts. At least one pilot and driving nut shall be furnished for each size of pin for each structure.

45. Screw Threads. Screw threads shall make tight fits in the nuts, and shall be United States standard, except for diameters greater than $1\frac{3}{8}$ in., when they shall be made with six threads per inch.

46. Annealing. Steel, except in minor details, which has been partially heated shall be properly annealed.

47. Steel Castings. All steel castings shall be annealed.

48. Welds. Welds in steel will not be allowed.

49. Bed Plates. Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed at top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

50. Shipping Details. Pins, nuts, bolts, rivets, and other small details shall be boxed or crated.

51. Weight. The weight of every piece and box shall be marked on it in plain figures.

Curved framing, hoppers, bins, and other complicated work shall be assembled and fitted in the shop.

Cast Iron. The ends of cast-iron columns and the tops of base plates and pedestals must be finished exactly at right angles to the vertical axis of the column.

The thickness of metal in cast-iron columns shall be not less at any point than that marked on the design drawings. The inside must be concentric with the outside. Shifting of the core more than $\frac{1}{8}$ inch will cause rejection. At least three holes shall be drilled in each column to test the thickness of metal.

Fins, chaplets, and other irregularities shall be removed by chipping, leaving neatly-finished surfaces. No holes shall be filled with cement or other substance without permission from the Architect.

The best practice shall be followed in reference to the quality of sand, molding, and the stripping of molds from castings.

Painting. The material shall be painted one coat of red lead paint at the shop and one coat of graphite paint after erection. The painting shall be done in accordance with paragraphs 52 to 58 of Schneider's Specifications.

52. Shop Painting. Steelwork, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

53. In riveted work, the surfaces coming in contact shall be painted before being riveted together.

54. Pieces and parts which are not accessible for painting after erection shall have two coats of paint before leaving the shop.

55. Steelwork to be entirely embedded in concrete shall not be painted.

56. Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

57. Machine-finished surfaces shall be coated with white lead and tallow before shipment, or before being put out into the open air.

58. Field Painting. After the structure is erected, the metal work shall be painted thoroughly and evenly with an additional coat of paint, mixed with pure linseed oil, of such quality and color as may be selected. The field paint shall be of different color from the shop paint.

Inspection and Testing. The inspection and testing will be done by the Architect or his representative. The contractor shall furnish the facilities for inspecting and testing and be governed by all of the provisions contained in paragraphs 59 to 64 of Schneider's Specifications.

59. The manufacturer shall furnish all facilities for inspecting and testing the weight, quality of material, and workmanship. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

60. When an inspector is furnished by the purchaser, he shall have full access at all times to all parts of the works where material under his inspection is manufactured.

61. The purchaser shall be furnished with complete copies of mill orders, and no material shall be rolled and no work done before he has been notified as to where the orders have been placed, so that he may arrange for the inspection.

62. The purchaser shall also be furnished with complete shop plans, and must be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect the material and workmanship.

63. Complete copies of shipping invoices shall be furnished to the purchaser with each shipment.

64. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

Erection. *Conditions at the Site.* (To be changed to suit the case). The site of the building cannot be given over to the contractor for his exclusive use. He must conduct his work as directed by the Architect, and in harmony with the other contractors working on the building at the same time.

There is no storage space on or adjacent to the building site sc the contractor must deliver the material as needed for erection, except arrangements may be made from time to time for the temporary storage of small quantities of material. He shall provide elsewhere such storage space as he may need.

Construction Equipment. The contractor shall furnish all equipment required for his operations. The equipment shall be adequate for its purpose, and must have ample capacity to carry on the work quickly and safely. The Architect shall have authority to order changes in equipment if, in his judgment, it is not adequate or safe.

Storing. Stored materials must be placed on skids and not on the ground. They must be piled and blocked up so that they will not become bent or otherwise injured.

Unpainted material shall not be so stored in the open. The materials shall be handled with cranes or derricks as far as practicable. They must not be dumped off of cars or wagons nor in any other way treated in a manner likely to cause injury.

Erecting Steel and Iron Work. The structural steel and iron work shall be erected as rapidly as the progress of the other work (particularly foundations and walls) will permit.

Setting Plates and Grouting. Base plates, bearing plates, and ends of girders which require to be grouted, shall be supported exactly at proper level by means of steel wedges. The grout will be furnished and poured by the mason contractor.

Plumbing, Leveling, Bracing. The structural steel and iron work shall be set accurately to the lines and levels established for the building, as shown on the drawings. Particular care shall be taken to have the work plumb and level before riveting.

Necessary bracing shall be provided for this purpose, and for resisting stresses due to derricks and other erection equipment and erection operations.

Elevator shafts shall be plumbed from top to bottom with piano wire and must be left perfectly plumb.

Temporary Bolts. The members shall be connected temporarily with sufficient bolts to insure the safety of the structure until it is riveted. Not less than one-third the holes shall be bolted.

Riveting. All field connections shall be riveted unless otherwise ordered. The riveting shall follow as closely as practicable after erection. The connecting members shall be drawn up tight with bolts before riveting. Rivets generally shall be driven with pneumatic hammers.

The rivets must be of proper length to form full heads. Rivets must be tight, with full concentric heads. Defective rivets must be cut out and re-driven. No re-cupping or calking will be allowed.

Permanent Bolts. When bolts are used for permanent connections, washers shall be placed under the nuts, the nuts drawn tight, and the threads checked. In such cases, bolts must be used which are provided for that purpose, and not ordinary machine bolts.

Connections to cast iron shall be bolted.

Removal of Equipment and Rubbish. The contractor shall remove the construction equipment as rapidly as its service is completed and shall remove all rubbish from day to day.

Immediately after final acceptance of the work, the contractor shall remove all his equipment and property and shall remove all rubbish resulting from his operations.

REVIEW QUESTIONS

REVIEW QUESTIONS

ON THE SUBJECT OF

STEEL CONSTRUCTION

PART I

---

1. State the three fundamental relations of equilibrium.

2. Discuss the expression "factor of safety" and show why its use should be discouraged in steel construction work.

3. Give short descriptions of the Bessemer and the open-hearth processes.

4. Name the important sections of structural steel members.

5. In what two ways are plates designated?

6. How is the center of gravity for angles with equal legs determined? Make sketch.

7. Compute the moments of inertia for a plate 6 inches wide and $\frac{3}{4}$ inch thick.

8. How is the radius of gyration derived from the moment of inertia?

9. What is the section modulus and how is it derived?

10. Compute the section modulus and radius of gyration for an angle $3'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$, taking as an axis the line parallel to the longer leg.

11. On what bases are structural steel orders handled? Which is the cheaper one?

12. Which sets of specifications may be used in regard to the quality of structural steel?

13. Which of the above sets refers to railway bridges?

14. How do these specifications compare in regard to rivet steel strength?

15. What are the usual limits of carbon for structural shapes?

16. Define "yield point".

17. What is the maximum allowable bearing on shop-driven rivets and pins in pounds per square inch.

18. State the sizes of rivets and bolts commonly used.

REVIEW QUESTIONS

ON THE SUBJECT OF

STEEL CONSTRUCTION

PART II

---

1. A joist has a span of 18 feet. It supports a floor area 6 feet in width. The floor construction weighs 115 pounds per square foot. Live load is 50 pounds per square foot. Compute the shear and bending moment.

2. Figure 76. Distance between supports is 18 feet. Overhang on the right end 6 feet, on the left 8 feet. Load 800 pounds per linear foot. What is the maximum shear? Bending moments at R_1 and R_2? Maximum positive bending moment?

3. Given a span of 20 feet, what is the load per lineal foot for a bending moment of 82,000 foot-pounds?

4. Compute shear and bending moments for two loads of 35,000 pounds placed at third points of an 18-foot span; at the quarter points.

5. What is the bending moment on an I-beam $18'' \times 55'' \times 40$ feet long due to its own weight and to a load of 4500 pounds concentrated at mid-span?

6. A crane girder has a span of 30 feet. The wheel load is 25,000 pounds, the wheel base is 8 feet. What is the position of loads for maximum bending moment? What is the amount of maximum bending moment?

7. Two angles are required to support a load of 5200 pounds uniformly distributed on a span of eight feet. Determine the section by means of the section modulus.

8. What I-beam is required to support a uniformly distributed load of 3500 pounds on a 30-foot span, the permissible deflection being $\frac{3}{4}$ inch?

9. Assuming the same loads per square foot as in Fig. 87 but using 18′ for the 16′ span and 24′ for the 21′ -6 span, (a) determine sizes of joist and girders, scheme *b;* (b) in scheme *g* it is desired to make the joists and girders the same depth, use two I-beams for the girder; what sections are required?

REVIEW QUESTIONS

ON THE SUBJECT OF

STEEL CONSTRUCTION

PART III

1. Define a column.

2. What is a tier?

3. Calculate the capacity of a column, the allowable unit stress of which is 12,000 pounds per square inch and its area 15 square inches.

4. It is desired to find (a) the average stress and (b) the maximum fiber stress resulting from the bending moment (taking $\frac{3}{4}$ of the computed moment), and (c) the total maximum fiber stress in the column, if total load equals 20,000 pounds, area of section equals 24.7 square inches, bending moment equals 600,000 inch-pounds, section modulus equals 89.3 inches. Show calculation.

5. State the A.R.E. formula for unit stress in columns. Which formula is used by the Bethlehem Steel Company?

6. In Fig. 152 make height of pedestal 1′-6″, load 800,000 pounds. Compute area of base on the masonry using 500 pounds per square inch, for other dimensions use those in the figure.

7. Compute proper cast-iron column, length 140 inches, concentric load 200,000 pounds, eccentric load 60,000 pounds, eccentricity 9″; assume 14″ for outside diameter.

8. In Fig. 174, assume four panels; let H equal 18 feet, L_1, L_2, L_3, and L_4, equal 22 feet, and W equal 40,000. Compute stresses in the diagonals.

9. In Fig. 176, assume H_2, H_3, and H_4 equal 12 feet, H_1, 14 feet, H_5, 16 feet, L_1, L_2, L_3, 18 feet. Compute the stresses in the diagonals. Assume $W = 200$ pounds per lineal foot.

10. Referring to Fig. 179, assume $W = 18{,}000$ pounds, H equals 14 feet, L equals 18 feet. Compute (a) axial stresses in the three members of the frame, (b) bending momemt at d, (c) Construct moment diagram.

11. Design the joint as shown in Fig. 187 using $\frac{7}{8}$-inch rivets spaced 3 inches.

REVIEW QUESTIONS

ON THE SUBJECT OF

STEEL CONSTRUCTION

PART IV

---

1. Estimate the weights of steel in the panels shown in Fig. 201, scheme *c*. Diminish each span by 1 foot.

2. Design girder *10-11*, typical floor, girder *31-33*.

3. Design spandrel girder *2-3*, typical.

4. Compute loads and make the design for column *9*. Make schedule as in Figs. 203 and 204.

5. Make a diagram showing floor areas supported by Column *33* at first, second, third, and typical floors. Plates C, D, E, and F.

6. In which direction do dead loads act?

7. Define live loads in buildings.

8. Draw sketch of section of flat tile arch floor.

9. State some of the items which affect the selection of floor type.

10. By what is the distance of columns from the building line governed?

11. How shall the framing around stair wells be designed?

12. Which scales should be used in making structural steel drawings?

REVIEW QUESTIONS

ON THE SUBJECT OF

STEEL CONSTRUCTION

PART V

1. Give briefly a theory on rust formation.

2. State the different degrees of exposure.

3. What qualities are desirable in a rust preventing paint?

4. What pigments are commonly used for structural steel paints?

5. Why should all surfaces of structural steel members be thoroughly cleaned before painting?

6. What numerical value has the coefficient of expansion of steel?

7. Discuss the influence of heat on structural steel.

8. What fireproof materials are employed for protection?

9. State the purpose of specifications.

10. What subjects shall complete specifications contain?

11. What considerations govern the thickness of fireproofing?

12. Which materials are best suited for fireproofing?

13. State the Chicago requirements in regard to "pipes enclosed by covering".

INDEX

The page numbers of this volume will be found at the bottom of the pages; the numbers at the top refer only to the section.

Page

A

Acid open-hearth process 24
Anchors 144, 343
Angles 39

B

Basic open-hearth process 26
Beam construction, details of 130
  anchors 144
  beams to beams, connection of 130
    angle 130
    special 134
  beams to columns, connection of 134
    combination 136
    seat 134
    web 136
  bearings 140
    bearing plates, dimensions of 140
    bearing plates, graphical diagram for design of 141
    bearing plates, standard 142
    cast-iron plates 143
    penetration into wall 142
  details, miscellaneous 144
  tie rods 139
Beam design 86
  bending moment, maximum 86
  deflection 87, 90
  flexure 87
  shear 87, 89
Beam specifications in fireproof structure 320
  beam, design of 323
  beam elevations 321
  columns, arrangement of 322
  girders, arrangement of 320
  joists, arrangement of 321

Page

Beams 85
  classification 85
    restrained 85
    simple 85
  definitions 85
  details of construction 130
  load effects 90
  practical applications 123
  resistance 107
  riveted girders 144
  specifications 320
  theory of design 86
Bessemer process 23
Bolts 81
  length 83
  machine 82
  tension, in 82
  turned 82
Built-up beams 85

C

Cantilevers 128
Cast-iron columns 237
  advantages 237
  column sections 238
  construction, details of 244
  design, method of 238
  disadvantages 237
  tables 242
Channels 38
Chemical composition of steel 55
  alloys 55
  manganese 55
  phosphorus 55
  silicon 55
  sulphur 55
Column bases 230
  cast-iron pedestals 232

Note.—For page numbers see foot of pages.

Page

Column bases (continued)
  cast-iron plates 232
  flat plates 231
  steel grillage 236
Column pedestals in fireproof structure 333
  dimensions 333
Column sections 193, 195, 238
  area 193
  distance from neutral axis to extreme fiber 193
  moment of inertia 194
  proof of 340
  radius of gyration 194
  unsupported length 194
Columns 185
  arrangement 322
  bases 230
  bending moment 336
  cast-iron 237
  details of construction 228
    brackets 230
    connections 230
    riveting 229
    splices 228
  loads 185
    computation 185
    concentric 186
    eccentric 186, 190
    typical cases 187
  pedestals 333
  sections, properties of 193
  specifications 327
  steel 185
  strength of 191, 201
    unit stress, formula for 191
  tables
    200, 202–205, 206, 208–221, 222–227, 241
  wind bracing 251
Concentric loads on columns 186
Connections
  beams to beams 130
  beams to columns 134
  girders to columns 175
  separators 137
  tie rods 139

Page

E

Eccentric loads on columns 186

F

Fire 355
  effects on steel 355
  intensity of heat in 356
  protective methods 356
Fireproof materials 358
  brick 362
  cinder concrete 358
  hollow tile 359
  Portland cement concrete 358
  selection of 362
Fireproofing
  materials used in 358
  requirements, Chicago building ordinances 363
  selection of 362
  thickness of 362
Fireproofing specifications 308, 357
  concrete, choice of 308
  exterior columns, effect on position of 308
  thickness required 308
Floor construction, fireproof 315
  tile and concrete, combination 318
  tile arch 315

G

Girder 85, 323

J

Joist 85, 323

L

Lintel 85, 126, 342
Load effects, calculation of 90
  combined 99
    bending moments for 275
    full length distributed load and concentrated load 99
    two distributed loads and concentrated loads 101
    two uniformly distributed loads not overlapping 100

Note.—For page numbers see foot of pages.

	Page
Load effects, calculation of (continued)	
concentrated	95
cantilever	96
simple	95
simple beams on two supports and projecting at both ends	96
typical loads	103
moving	106
simple	103
tabular data	103
two or more	106
uniformly distributed	90
cantilever	91
joists	91
simple and cantilever, combination	92
Loads on fireproof building	309
dead	309
live	311
special	312
specifications for	372

M

Manufacture of steel	19
iron ore to pig iron	19
pig iron	20
smelting	19
pig iron to steel	21
acid open-hearth process	24
basic open-hearth process	26
Bessemer process	23
rolling the ingots	28
blooming	29
plate rolls	32
rolls, roughing and finishing	29
Modulus of elasticity	90
Moment of inertia	46, 194

P

	Page
Paint	351, 379
applying	355
composition	352
prepared	354
purpose	351
qualities	351
Painting	354, 386
cleaning	354
surfaces in contact	355
Physical properties of steel	56
area, reduction of	58
breaking load	57
elastic limit	57
railway bridge grade	56
yield point	57, 58
yield point and actor of safety	58
Pig iron	20
Plates	42
Practical design of sixteen-story fireproof hotel	283
beam specifications	320
column pedestals	333
column specifications	327
description	283
dimensioning drawings	343
base lines	343
column centers	343
elevations of spandrel beams	345
girders and joists	345
vertical dimensions	345
fireproofing specifications	308
floor construction, type of	315
loads	309
miscellaneous features	341
anchors	343
chimney and its supports	341
flag pole support	343
lintels	342
masonry supports	341
mullions	343
spandrel sections	342
weights of materials of construction	310
wind bracing	336
Protection of steel	349
fire, from	355
rust, from	349

R

Radius of gyration	48
Resistance, calculation of	107
deflection	119

Note.—For page numbers see foot of pages.

Page

Resistance, calculation of (continued)
- lateral support 122
- moment, resisting 107
  - application of tables to concentrated loads 109
  - section modulus 108
  - tabular values 108
- shearing 118

Resisting moment 107

Restrained beam 85
- moment diagram for 274

Rivet heads 66

Rivets 62
- bearing 74
- driving 68
  - hand 72
  - pneumatic hammer 71
  - riveting machines in shop 70
- friction 77
- function 73
- heads 66
- investigation 77
- length 83
- punching and reaming 72
- rivet heads 66
  - button 67
  - flattened and countersunk 67
  - manufacture 66
- shear 75
- size 62
- spacing 63
  - clearance 65
  - edge distance 65
  - gage 64
  - pitch 64
- tables 77
- tension 77

Riveted girder 144
- beam box 169
- construction, details of 174
  - connections to columns 175
  - end bearings 174
  - lateral support 180
  - splices 175
- I-beams with flange plates 168
- plate girder 147
- practical applications 170

Page

Riveted girder (continued)
- practical applications
  - girders, crane 173
  - girder supporting a column 170
  - plate girder lintel 172
  - roof girder 172
- theory of design 145
- unsymmetrical sections 169

Riveted girder design 145
- depth 148
- flange section 151
  - width of flange plates 153
  - with flange plates 152
  - without flange plates 151
- flange plates, length of 154
  - graphical solution for concentrated loads 155
  - graphical solution for uniormly distributed loads 155
- load effects, calculation of 147
- moment of inertia required 150
- resisting moment, determination of 145
  - chord stress method 145
  - moment of inertia method 145
- rivets connecting flange angles to web 159
  - cover plates, or 161
  - number 159
  - rivet spacing computed from web bearing 162
  - rivet spacing in flanges 160
  - rivet spacing when load is transmitted through flange rivets into web 162
- stiffeners, web 156
  - intermediate 158
  - loaded points, at 156
- tables and diagrams 164
- web, thickness of 149

Riveted girder construction, details of 174
- connections to columns 175
- end bearings 174
- lateral support 180
- splices 175

Riveting in girders 159

Note.—For page numbers see foot of pages.

	Page
Rolling ingots	28
Rust	
cement as a preventive for	355
composition of metal, effect of	351
degrees of exposure, effect of	350
formation, theory of	349
paint as a preventive for	351
Rusting, rate of	350
S	
Section modulus	49, 108
Separators	137
Simple beam	85
Span	85
Spandrel beam	85
Spandrel girders	324
Specifications	365
example	369
details of construction	368, 379
erection	368, 387
general conditions	367, 369
inspection and tests	368, 386
instructions to bidders	366, 369
loads	367, 372
painting	368, 386
quality of material	368, 375
scope of work	367, 370
unit stresses	367, 373
workmanship	368, 384
general characteristics	366
outline	366
purpose	365
Splices	228
Standard specifications	
beam	320
structural steel	52
bending requirements	54
chemical analysis	53
elongation and fracture	54
process of manufacture	53
range of application	53
rivet steel strength	54
tensile strength	53
Steel construction	
beams	85
books, reference	15
columns	185

	Page
Steel construction (continued)	
furnishing	18
practical design	283
presentation, method of	14
protection	349
safety, factor of	17
scope	11
structural steel	19
tables	16
tension members	245
Steel sections	
adaptability and use	33
angles	39
channels	38
I-beams	35
plates	42
tees	41
zees	40
properties	45
center of gravity	45
moment of inertia	46
radius of gyration	48
section modulus	49
Strength of columns	191, 201
Structural steel	19
beams	85
composition, chemical	55
design, practical	283
fire, protection from	355
girders, riveted	144
manufacture	19, 54
members, compression	185
members, tension	245
orders, mill and stock	50
price	50
properties, physical	56
rivets and bolts	62
rust, protection from	349
sections	
adaptability and use	33
properties	45
specifications	365
standard specifications of	52
stresses, unit	59
tests, inspection and	58
weight	51
wind bracing	251

Note.—For page numbers see foot of pages.

Page

T

Tables
- beams, strength of 110–117
- gages for angles 64
- safe loads on Bethlehem columns 208–221
- safe loads on channel columns 222–227
- safe loads for round cast-iron columns 241
- typical loadings, reactions and bending moments for 104, 105
- unit stress in compression 206
- un t stress in compression in columns 202–205

Tank support 128

Tees 41

Tension members 245
- connection details 249
- definition and theory 245
  - axial tension 245
  - tension due to eccentricity 246
- net area 248
- sections 247

Tie rods 139

U

Unit stresses 59
- cast iron 61
- masonry 62
- specifications for 373
- structural steel 61

Page

W

Wind bracing 251, 336
- combined wind and gravity stresses in girders 274
- framework, systems of 255
  - rectangular 258
  - triangular 255
- horizontal pressure 251
- paths of stress 252
- wind-bracing girders, design of 267
  - end connections for I-beam girders 273
  - end connections for riveted girders 267
- wind stresses on columns 278
  - combined direct and bending 278
  - combined stresses, design of column for 279

Wind bracing in fireproof building 336
- bending moments 336
- column sections, proof of 340
- interior girders, resistance of 337
  - first floor, girder resistance for 339
  - second floor, girder resistance for 339
  - third floor, girder resistance for 338
- loads for entire building 336
- other wind stresses 341
- spandrel girders, resistance of 337

Z

Zees 40

Note.—For page numbers see foot of pages.