STEEL CONSTRUCTION

A Text and Reference Book on the Design of Steel Framework for Buildings

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PREFACE

This edition of Steel Construction has been thoroughly revised and enlarged to bring it up to the present in steel-construction practice. The revision includes up-to-date specifications and data, and numerous photographs and drawings. The material has been rearranged into chapter form to make it more convenient for use. Also, additional chapters have been written, such as those on Welding and Industrial Buildings, to cover present steel building design more completely.

Complete sets of structural plans for a power house and for a four-story school building are included in the chapters on Industrial Buildings and Multistory Buildings.

A great part of the satisfaction derived from the practice of engineering comes from seeing the "dreams come true." The engineer is commonly assumed to deal with facts and to be guided by mathematical relations. While this is true, he must at times be a dreamer, a man with an active imagination. Before a line is drawn or a figure placed on paper, the engineer must have some conception of the structure he is to create. The more definite this conception, the more readily it can be committed to paper in the form of drawings. The mention of a building of a certain size or for a certain purpose brings a vision of the skeleton to support it; the architect's perspective or elevation suggests the columns concealed within the piers and the girders behind the spandrels; the floor plans indicate the location of columns, girders, and joists which will be required to support the floors, partitions, and walls. From these mental pictures, the design drawings can be evolved by applying the mathematical relations to determine the sizes of members required.

But the use of the imagination does not stop here; it is needed in perfecting the details. The more fully it is developed, the more quickly can proper sizes and arrangement of material be established. Imagination is a natural talent which can be improved by practice and experience. It is not easily distinguished from the judgment resulting from experience.

This book does not deal with the visions of proposed structures, but with the facts and formulas for transforming these visions into tangible designs. Having realized the dream in definite plans, there follows the growth of the practicable structure. The successful completion of the skeleton, which, silently and unseen, must carry the weight of the building with assurance of perfect safety to the people who occupy it, must give great satisfaction to him whose brain has created it. Then has the dream come true.

This book is intended to give its students the facts and formulas needed in designing the structural steel framework for buildings. Since facts and formulas alone would be of little use, they are accompanied by explanations of the underlying principles, a clear understanding of which is essential to the intelligent use of the formulas. The use of the formulas is shown by illustrations of a practical nature which serve not only to teach the proper application, but to illustrate current practice in this form of construction.

For use as a textbook by students, the important feature is the theory on which are based the formulas and their applications. A student can easily learn to use tables, apply formulas, and copy the work of others. But without a knowledge of the fundamental principles he will not be able to determine the proper limitations of the tables and formulas nor to distinguish the good and bad features of designs made by himself or others.

For use as a reference by designers, the book brings together the necessary data, easily accessible, for the complete design of structural steel work for business buildings, and gives enough illustrations to guide in the solution of the problems usually encountered in practice.

ACKNOWLEDGMENTS

In addition to the acknowledgments given in the text, appreciation is expressed to the American Institute of Steel Construction; American Specification Institute; American Welding Society; American Bridge Co.; Atchison, Topeka and Santa Fe Railway; A. S. Alschuler, architect; Bethlehem Steel Co.; Dardelet Threadlock Corp.; Eastern Structural Clay Tile Association; Graham, Anderson, Probst and White, architects; Hanna Engineering Works; Holabird and Root, architects; Ingersoll-Rand Co.; Lincoln Electric Co.; The Linde Air Products Co.; National Fireproofing Corp.; H. H. Robertson Co.; Steel Joist Institute; Thomas Machine Manufacturing Co.; Truscon Steel Co.; and Whiting Corporation. These organizations have furnished photographs, drawings, and other valuable data.

Appreciation and thanks are also due Mr. Henry Penn, District Engineer, A.I.S.C.; Mr. S. Jonsson, Chief Structural Engineer, Graham, Anderson, Probst and White; Mr. Arthur Burke for excellent work on the drawings, Mr. Kenneth Person for help in checking computations and Mrs. F. Swarts for editorial work on the manuscript.

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41-STORY BUILDING, ROCKEFELLER CENTER
Steel Erected by American Bridge Company

STEEL CONSTRUCTION

CHAPTER 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, mill buildings, residences, etc.

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, wide-flange 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. A short discussion regarding the unit stresses in common use today is given.

After these preliminary considerations comes the study of design. As rivets, bolts, and also welding, 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, complete structures are then investigated and designed. Examples of existing structures are taken for this purpose.

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 equipment for designing any ordinary steel construction.

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.

The design of steel bridges and trusses and the theory underlying statically indeterminate stresses, that is, stresses that cannot be figured without considering the strains and deflections of the structural members, are not treated in this book, since commercial and residential buildings do not usually involve such computations or design.

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 essential theories of strength of materials, structural drafting, statics, and roof trusses, have been mastered.

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.

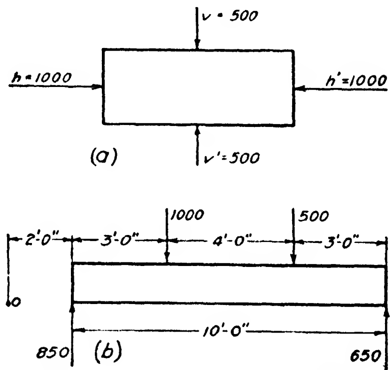


Fig. 1. Diagram Showing Forces in Equilibrium

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

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. There is no tendency to rotate and the third relation is satisfied.

In Fig. 1-b 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 $(5\times1000)+(9\times500)=9500$, and counterclockwise $(2\times850)+(12\times650)=9500$ foot pounds. Hence the summation of moments equals zero, and the forces acting on the body are in equilibrium.

Design Viewpoints. The designing engineer must always keep three things in mind—strength, cost, and practicality. He considers strength or safety when he so designs his steel framework that the steel frame and every part of it safely carries all the loads (dead, live, and wind loads) to the foundations. Every joist, beam, column, and connection must be able to support all the loads that it may be called upon to carry and it must do so safely so that no part of the building is unduly strained or weakened or made unsightly by cracks due to faulty design or construction. The possibility of fire must be considered and the steel frame protected against rust. The designer must not forget that shop mechanics and builders are not necessarily engineers and he must therefore so word the specifications and so thoroughly conduct the supervision of steel erection that the strength of the construction is safeguarded in every detail. Just as a chain is only as strong as its weakest link so is a building only as safe as its weakest part, and the failure of any column, beam, girder, joist, or even of a connection may lead to the collapse of a building.

The cost of a building is a consideration that cannot be forgotten by the designer. Economic design requires planning so that the least weight of metal be used; that standard and stock sizes be specified where possible; that fabrication be reduced to a minimum; and that erection difficulties be considered, foreseen, and minimized. Economic design means careful, accurate, and detailed design; for mistakes and changes are costly.

Designs must be practical. They must work. Connections, columns, and girders should be possible of fabrication and erection. The needs of the architectural plan must be considered. The steel framework must often be hidden in walls, floors, and partitions. It must not interfere with mechanical or other equipment of the building. Minimum clear headroom over passages and stairs must be kept in mind. The artistic effects sought by the architect must not be marred by the engineer's failure to consider details and dimensions in designing his frame.

Sometimes when a structural frame is made part of the visible portion of a hall, the engineer finds it necessary to consider appearance as another factor in design. It is surprising how graceful steel framework may be made to appear by properly proportioning and simplifying the frame and its details and by adding a curved angle, beam, arch, or gusset. Witness the true artistry of the New York Hell Gate Bridge or of the Train Hall of the Union Depot in Chicago. On the other hand, failure by engineers to consider appearance where structural steel is exposed to view has dotted the country in our industrial sections with hideous structures that are a disgrace to our profession.

Appearance must be considered in the design of exposed steel. Simple framing, an occasional curve, long sweeping lines, genuine and economic construction—all these tend to lend beauty to a structural steel design. Beautiful designs usually prove economic.

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. Unless city building codes cover, 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 Carnegie-Illinois Steel Corporation, Pittsburgh, Pa., and Chicago, Illinois; Bethlehem Steel Co., Bethlehem, Pa.; and American Institute of Steel Construction, New York.

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 *pounds* and others in *thousands of pounds* (kips). Of course the heading of the table should show this, but special care should be taken to make sure which is used. 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.*

Handbooks usually have loads expressed in kips unless otherwise indicated. The term "kip" is abbreviated from kilo-pounds and is widely used in technical literature to designate one thousand 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.

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. The ratio of the ultimate strength to the allowable working stress is called the "factor of safety." This term is frequently misunderstood or misinterpreted to give a false sense of security to the designer or purchaser. The error occurs in assuming the ultimate strength of steel in all cases as being that of steel in direct tension. This is erroneous, as the ultimate strength of steel columns is approximately the yield point. Girders and beams also may fail in the compression flange, which failure occurs at approximately the yield point of the material.

Structural steel has an average ultimate strength in tension of about 66,000#/□″, an average yield point of about 38,000#/□″, and a working stress of 20,000#/□″. It can be said that structural steel has a nominal factor of safety of about 3.3 and an actual of about 1.9.

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.

Problem. If a steel has an ultimate strength of 70,000#/□″, a yield point of 35,000#/□″, and an allowable working stress of 20,000#/□″, what is the factor of safety for a steel rod in tension? For a column? (Assume failure at yield point.)

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, welding, 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, weld, 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 engineer cooperating 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. If welding is used, the type, size, and length of all welds are shown.

IRON ORE BEING MINED BY OPEN PIT OR STRIPPING OPERATION

Photo by Bethlehem Steel Company

ORE STORAGE YARD SHOWING ORE BRIDGES OR CRANES

The Different Grades of Ore Are Stored Here and Used as Needed by the Blast Furnace

Photo by Bethlehem Steel Company

CHAPTER II

MANUFACTURE OF STEEL

The procedure in the manufacture of structural steel sections from iron ore consists of the following operations: (1) smelting the

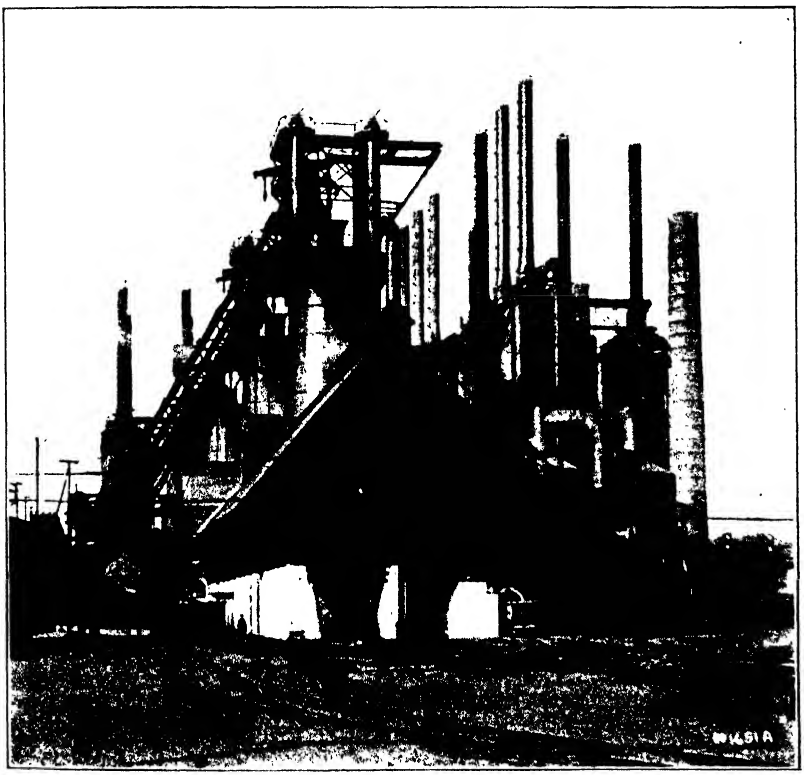


Fig. 2. General View of a Blast Furnace
Photo by Bethlehem Steel Company

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

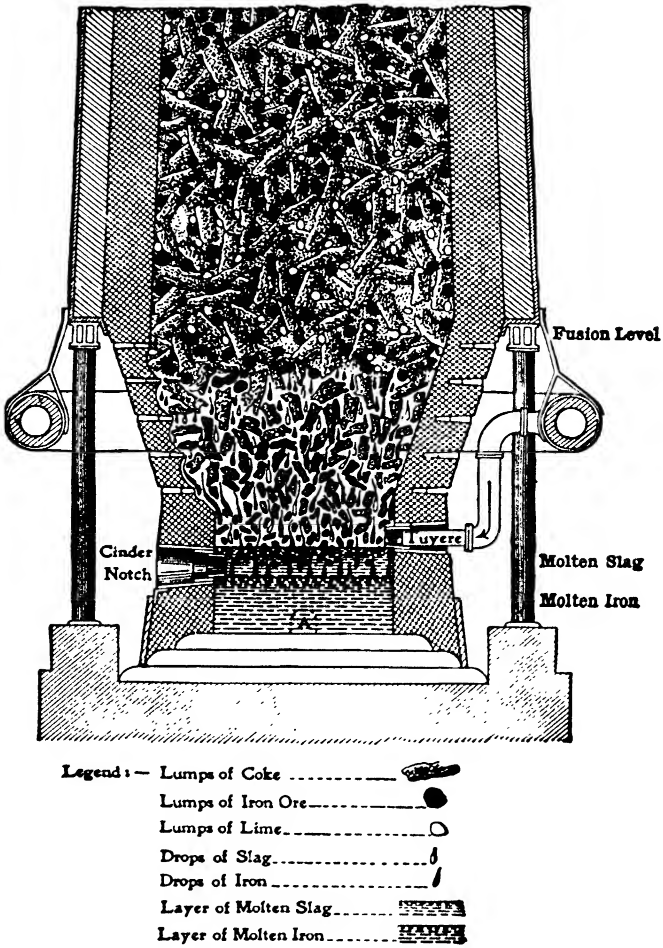


Fig. 3. Section Through Base of Furnace Showing Layers of Molten Iron and Slag with Unmelted Ingredients Above
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy McGraw-Hill Publishing Company

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.

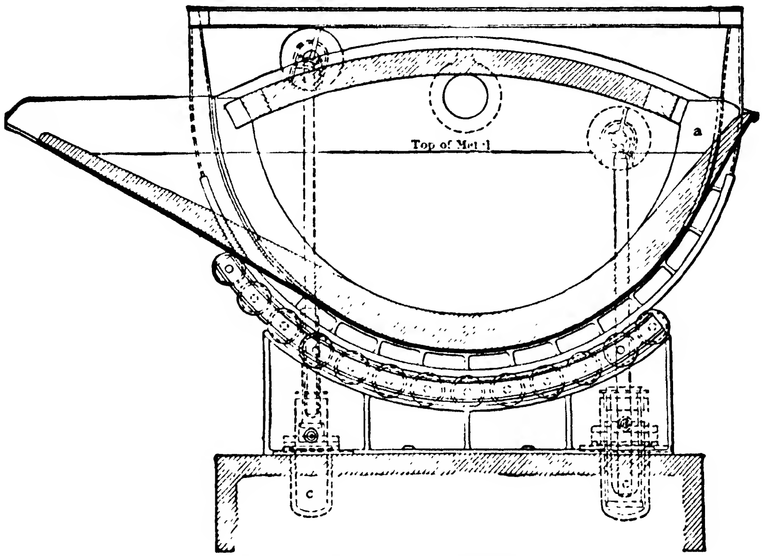


Fig. 4. Section Through a Mixer
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy, McGraw-Hill Publishing Company

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

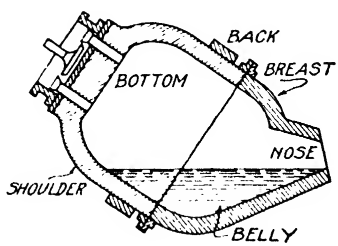


Fig. 5. Parts of Converter
Courtesy McGraw-Hill Publishing Company

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.

Fig. 2 shows a general view of a blast furnace. The charges of iron ore, limestone, and coke are hoisted at intervals up the incline shown at the left of the furnace. The molten pig iron is tapped about every five hours from the bottom into the ladle cars seen on the track at the left.

Fig. 3 shows a section through the bottom part of the furnace,

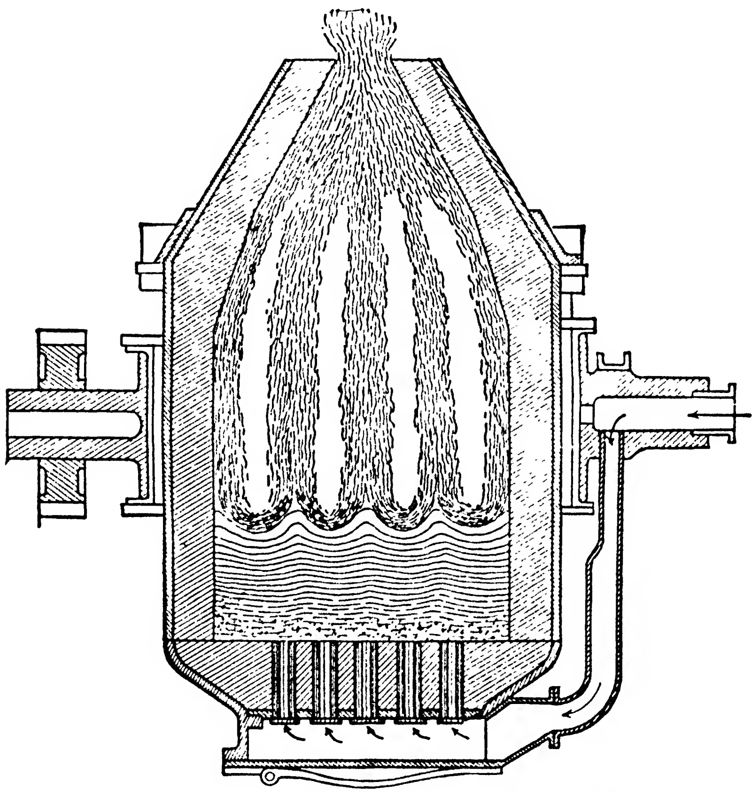


Fig. 6. Section Through Bessemer Converter While Blowing
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy McGraw-Hill Publishing Company

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 is either poured into forms called pigs or is kept in the molten stage for the next operation. 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

Fig. 7. Bessemer Converter in Operation
Photo by Bethlehem Steel Company

absorbed from the coke, about 4 per cent of silicon, and smaller quantities of manganese and phosphorus which remain from the ore.

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 elimination 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, published by 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. 4, 5 and 6, lined with silicious material. Through the molten 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

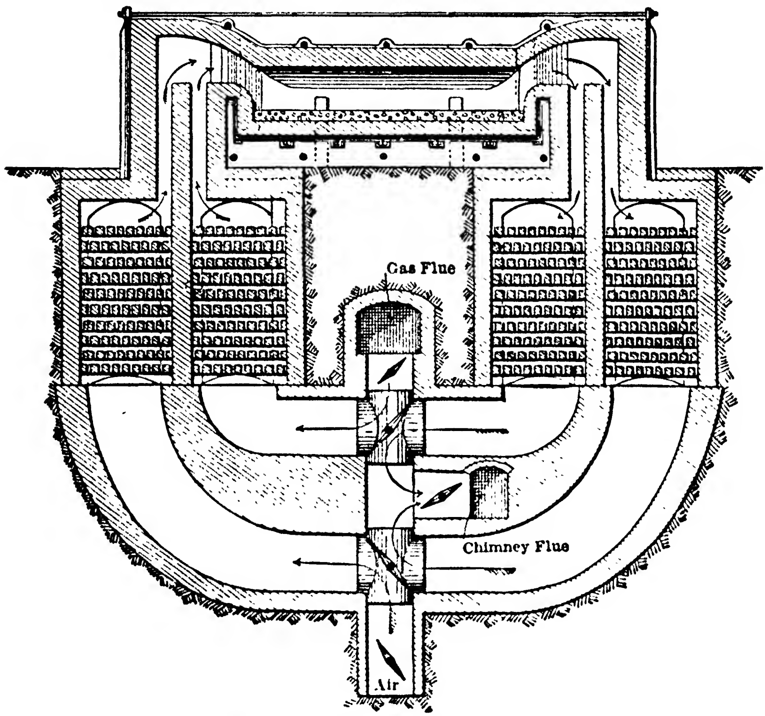


Fig. 8. Section of Regenerative Open-Hearth Furnace
From Stoughton's "Metallurgy of Iron and Steel"
Courtesy McGraw-Hill Publishing Company

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. 7. 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 operation 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 Bes-

Fig. 9. Charging Materials into an Open-Hearth Furnace
Photo by Bethlehem Steel Company

semer process are: (a) great rapidity of purification, say ten minutes per 'heat'; (b) no extraneous fuel is used; and (c) the metal is not melted in the furnace where the purification takes place."

The Bessemer process was the first method used for large-scale production of steel. In recent years, however, it has been nearly entirely replaced by the open-hearth method. It is now used only for making nails, screw stock, and some tin plate. In the Duplex Process of making steel the Bessemer converters are used for supplying decarburized metal to the open-hearth furnaces. In this method the metal from the Bessemer converters serves as pig iron for the open-hearth furnaces. When this metal is charged in the open-hearth furnaces with the higher carbon pig iron, it shortens the time required to make steel. It only takes about seven hours to produce a heat of steel by the Duplex process, whereas, by the regular open-hearth method it takes ten to twelve hours.

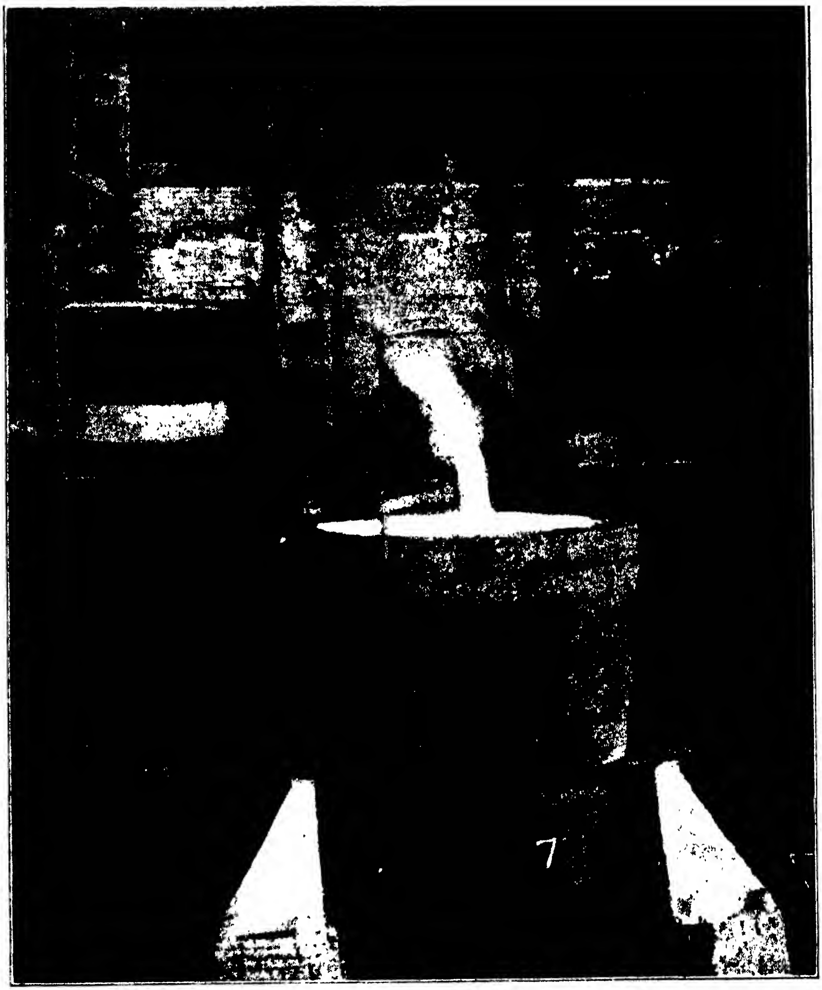


Fig. 10. Tapping Steel from Open-Hearth Furnace
Photo by Bethlehem Steel Company

*"**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

*From Stoughton's *Metallurgy of Iron and Steel*, McGraw-Hill Publishing Company furnace is shown in Fig. 8. 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 neces-

Fig. 11. Ladle Filling Ingot Molds at Pouring Platform
Photo by Bethlehem Steel Company

sary 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 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 12. Removing Ingots from Molds
Photo by Bethlehem Steel Company

Fig. 9 shows the charging of materials into an open-hearth furnace. The arm of the charging machine is seen conveying a load of scrap into the furnace. The box at the end of the arm is turned when in the furnace, thus dumping the load in the box onto the furnace floor.

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

*From Stoughton's *Metallurgy of Iron and Steel*, McGraw-Hill Publishing Company

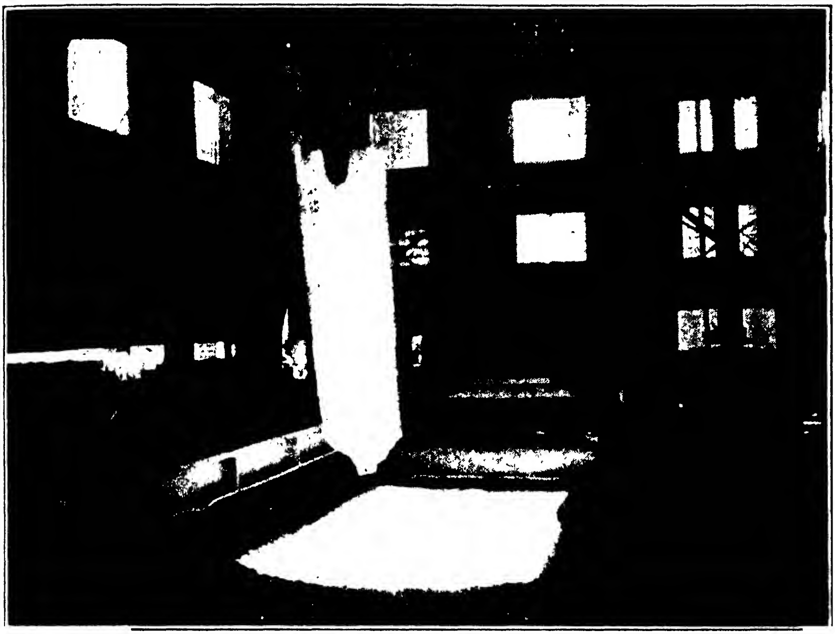


Fig. 13. Removing Ingot Heated to Proper Rolling Temperature from Soaking Pit
Photo by Bethlehem Steel Company

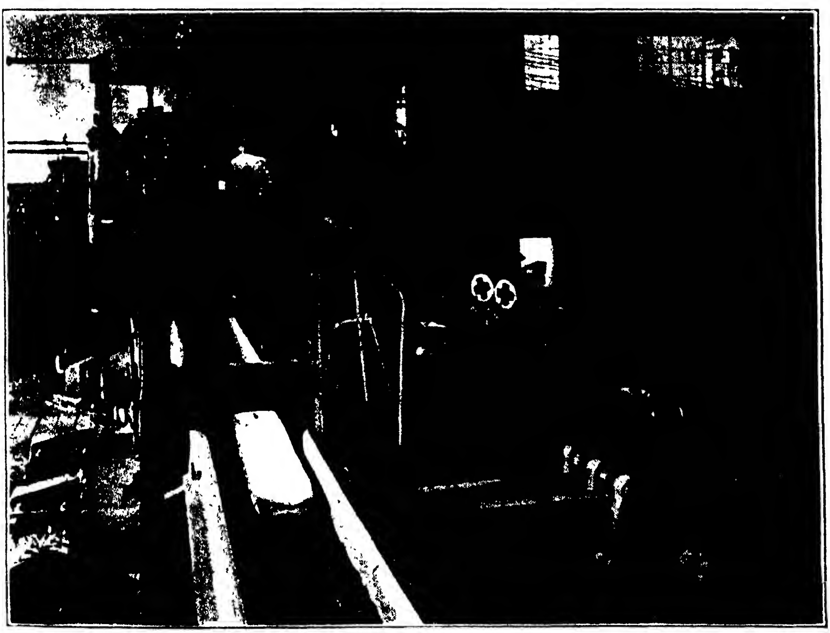


Fig. 14. 35-Inch Blooming Mill
Photo by Bethlehem Steel Company

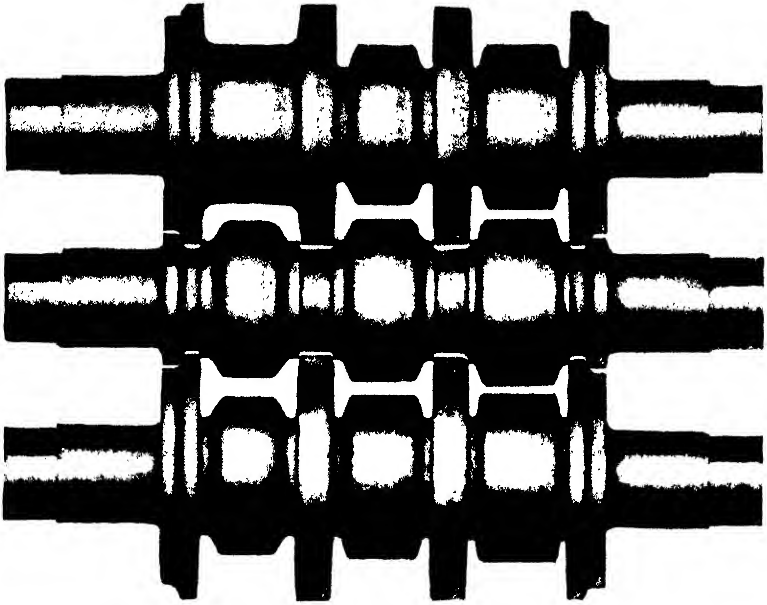


Fig. 15. "Three-High" I Beam Roughing Rolls
Courtesy Seaman, Sleeth Company

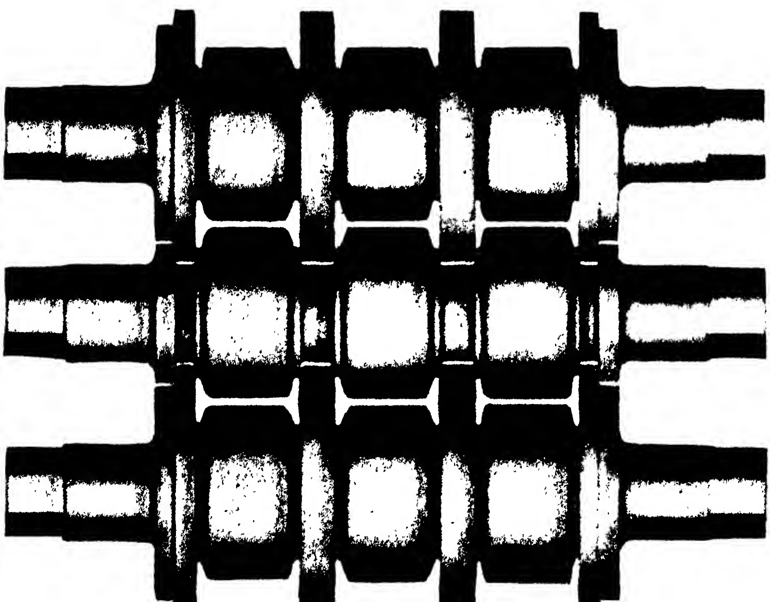


Fig. 16. "Three-High" I Beam Finishing Rolls
Courtesy Seaman, Sleeth Company

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 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."

Fig. 10 shows the tapping of an open-hearth furnace. On the balcony is seen an observer with an optical pyrometer for determining the temperature of the pour.

The ladle full of molten metal is carried by a crane to the desired positions above the ingot molds and there it is teemed out of the bottom of the ladle into the different molds. The steel solidifies here and in this form is known as ingots. Fig. 11 shows the ladle filling ingot molds at the pouring platform.

As soon as the molten metal is poured into the ingot molds, it loses heat and begins to solidify. When the metal in the molds is sufficiently rigid, the train load is transferred to the soaking pits where a specially designed crane removes the ingots from the molds which remain on the cars. Fig. 12 shows the removal of the ingots from the molds. The ingots are then heated in the soaking pits to the correct temperatures for rolling. Fig. 13 shows an ingot that has reached the desired temperature and is being removed for rolling.

Rolling the Ingots. The steel in the ingot is in its final condition as to chemical composition and must now be worked into plates and shapes required for structural uses. This is done by passing the steel back and forth between rolls. The first rolls the ingot is passed through are called blooming rolls. In passing between these rolls, 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 bloom is intended for rolling into structural shapes, it is turned from side to edge in passing through the rolls so that it will be kept approximately square in section until it is reduced to proper size for beginning to form the shape. Fig. 14 shows a 35-inch Blooming Mill.

Roughing and Finishing Rolls. The next step is to pass the steel

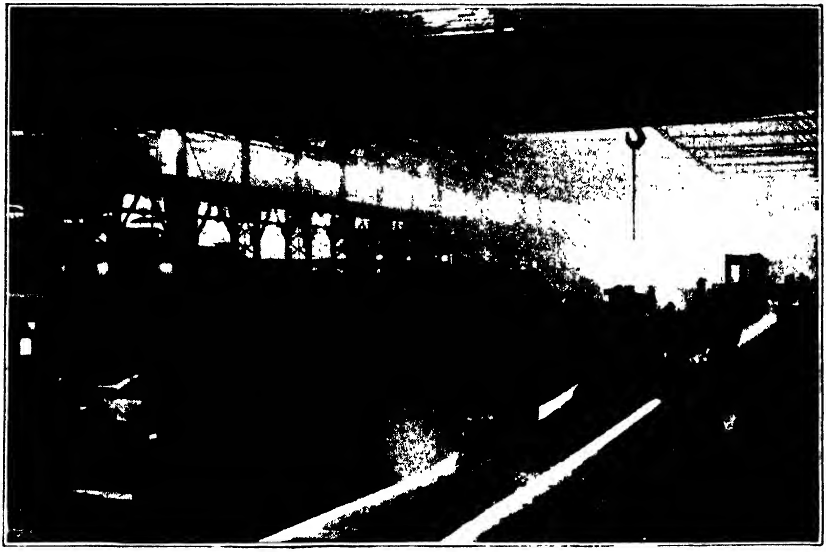


Fig. 17. Fourteen-Inch Mill for Rolling Small Structural Shapes
Photo by Bethlehem Steel Company

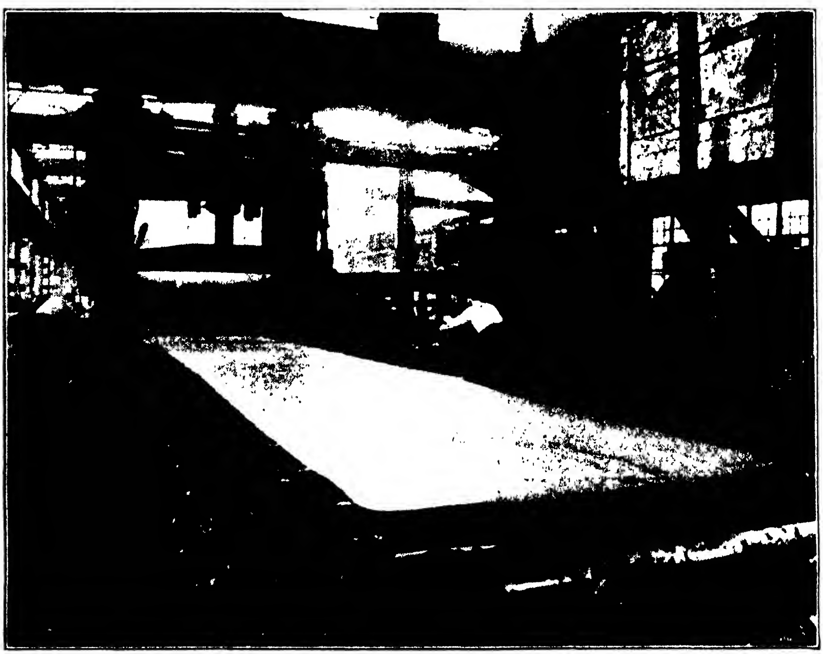
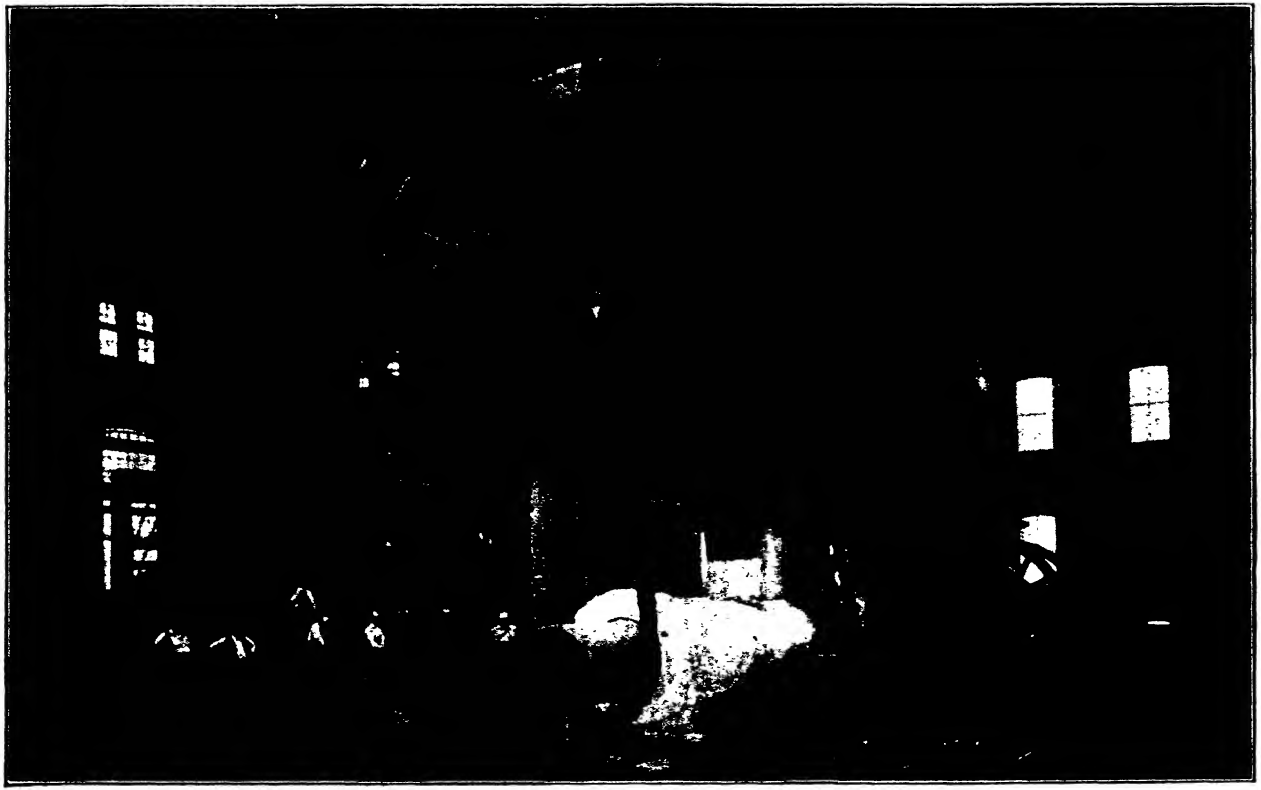


Fig. 18. Plate Mill
Photo by Bethlehem Steel Company

through the roughing rolls. These rolls are grooved in such 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. 15 and 16.

Fig. 17 shows a small structural shape being rolled in a 14-inch mill. In this 14-inch mill are rolled standard angles, channels, beams, flats, and rounds.

Plate Rolls. Plates are rolled as shown in Fig. 18. After being rolled down to the required thickness the plates are then sheared to the specified size. Vertical rolls can be placed in front and on the side of the horizontal rolls to control the width, and such a mill is called a "Universal Mill." The plates produced from such a mill are called "Universal Mill Plates" or edged plates.



FORGING A CYLINDRICAL VESSEL IN A 14,000-TON HYDRAULIC PRESS
Photo by Bethlehem Steel Company

CHAPTER III

SPECIFICATIONS

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 in the furnace, and the product is analyzed and tested, chemically and 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.

Steel Specifications. For many years there has been only one specification for building steel (American Society for Testing Material —Steel for Buildings serial designation A9) and one for bridge steel (A.S.T.M.—A7). However, for the past several years the difference between these two specifications has been only of a minor nature, so that the steel companies have made no distinction or difference in price between the two steels. The main difference between the two specifications is that A.S.T.M.-A9 allows the use of Bessemer steel; however, this is of no real significance any more as there are no mills in the country that roll structural shapes, except small angles and miscellaneous sections, of Bessemer steel. Until recently, there was one mill in the east that had an ore supply that was sufficiently free from phosphorus to make it practical and economical to manufacture Bessemer steel. This mill had continued rolling American Standard sections of Bessemer steel; however, recently this practice has been discontinued.

Chemical Properties and Analysis. The specification limits the amount of phosphorus and requires the chemical analysis to report the percentages of carbon, manganese, phosphorus, and sulphur—also copper when it is specified.

Physical Properties and Tests. The specification allows the ultimate tensile strength to vary between 60,000 to 72,000 pounds per square inch. The yield point is required to be one-half of the ultimate tensile strength, but in no case less than 33,000 pounds per square inch.

The ductility required is secured by specifying a certain allowed elongation, also definite bend-test requirements.

Rivet Steel. As noted in the specification A.S.T.M.-A9, the specification for Structural Rivet Steel should be A.S.T.M.-A141 unless otherwise noted. The required ultimate strength for this steel is 52,000 to 62,000#/□″. The specified yield point is one-half of the ultimate strength, but in no case less than 28,000 pounds per square inch.

DISCUSSION OF IMPORTANT FEATURES

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. Steel that is to be welded should not contain over 0.30 per cent carbon and preferably not over 0.25 per cent.

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 for Bessemer steel and not over 0.06 per cent for open-hearth steel.

Steel containing too much phosphorus is said to be cold short because it yields more easily to shocks and is then unsafe as structural material.

Sulphur. Sulphur is also found as an impurity in the iron ore. Its presence makes steel "red short," that is, it causes steel to crack when rolled hot and makes welding difficult. It usually amounts to less than 0.05 per cent.

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 two alloys used to any extent for structural work are silicon and nickel steels. These steels, of course, are higher in price than carbon steel, and the fabrication cost is also higher. They are economical only in special cases where extra heavy construction is employed.

Physical Properties. The determination of the physical properties most suitable for structural steel has been a gradual development —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 chemical tests are of interest only to the extent that they indicate physical properties.

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, economy of manufacture, and ease of fabrication.

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 5,000 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.

The specification A.S.T.M.-A9 (to be found printed in the Appendix) requires that the yield point be one half of the ultimate strength, but in no case less than 33,000 pounds per square inch. Mill test reports show that, actually, the yield-point strength varies between 36,000 and 40,000 pounds per square inch.

Elastic Limit. The change in the rate of elongation does not

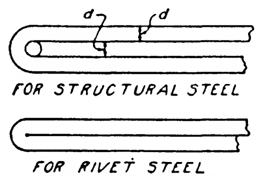


Fig. 19. Bending Tests for Steel

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 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.

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.

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 fin-

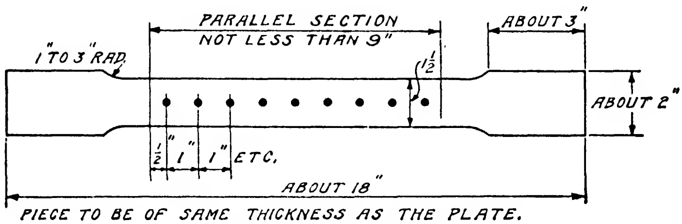


Fig. 20. Tension Test Piece

ished structural steel. The bend test is made by bending the specimen around a pin whose diameter equals the thickness of the specimen, for material $\frac{3}{4}$ inch or less in thickness, Fig. 19. 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.

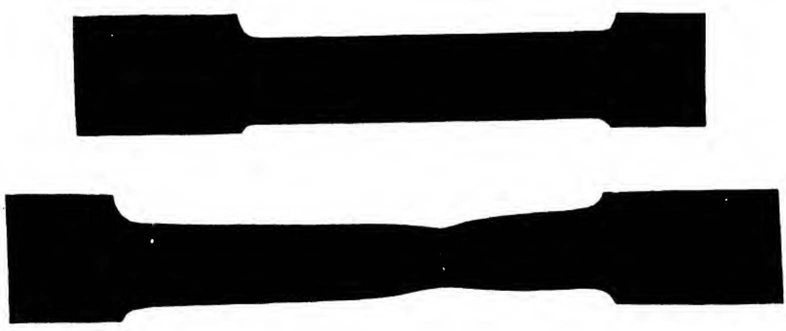


Fig. 21. Test Piece before and after Being Broken by Tension

The tension-test specimen is shaped as shown in Fig. 20. 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. 21.

Records of these tests are furnished to customers if desired. Customers usually have test engineering companies make these tests for them. The inspectors also make a surface inspection to see that the finished sections are straight and free from cracks, blisters, buckles, and slivers. Fig. 22 is a specimen report of tests.

Material Specifications. The American Society for Testing Materials issues, every three years, two books of material specifications. One of these books contains standard specifications for all kinds of metals, the other book contains specifications of all nonmetals. When using alloy steels, cast steel, cast iron, or any other material, it is desirable to refer to the applicable specification of the A.S.T.M.

DESIGN SPECIFICATIONS

Practically all of the larger cities have building codes or specifications covering the design of buildings, therefore in the design of most buildings it will be necessary to use the city code that applies.

Steel. Undoubtedly the most widely used structural steel specification covering building design is that of the American Institute of Steel Construction. A large number of cities throughout the country have adopted this specification. As this specification is printed in whole in the Appendix, it will not be repeated here.

The A.I.S.C. specification allows a basic unit stress in tension of 20,000 pounds per square inch. Another recent improvement is in allowing plate girders to be proportioned by the moment of inertia of the gross section. The student should study this specification so as to understand and become well versed in its requirements.

Cast Steel. The A.I.S.C. Specification allows the same unit stresses for cast steel in compression and bearing as for structural steel. For other unit stresses, 75 per cent of those for structural steel are allowed.

Cast Iron. The quality of cast iron varies somewhat and defects are likely to occur in casting, therefore it is not used to any great extent. Where castings are required for truss shoes, etc., cast steel is now used. Cast iron columns are made for use in small buildings.

Unit stresses for cast iron are as follows:

Axial tension, net section	3,000
Axial compression, gross section	12,000
Tension, extreme fiber in bending, net section	5,000
Compression, extreme fiber in bending, gross section	12,000
Shear	3,000

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REPORT NO. 1
SHEET 1 OF 1 SHEETS

REPORT OF MILL TESTS AND SHIPMENT OF STRUCTURAL MATERIAL

FOR 18th Street Power House, Chicago, Illinois. DATE OF REPORT 9/1/37.
MANUFACTURED BY Carnegie-Illinois Steel Corporation, S. Chicago, Illinois.
ALL TESTS WERE WITNESSED AND RESULTS COMPLY WITH A.R.E.A. 1935. SPECIFICATION
MATERIAL CONSIGNED TO American Bridge Company, Gary, Indiana. CONTRACT G-7027.
SHIPPED IN CARS See below ON Below

QUANTITY	DESCRIPTION OF MATERIAL	SHIPPERS WEIGHT	HEAT NUMBER	YIELD POINT IN LBS. PER SQ. INCH	TENSILE STRENGTH IN LBS. PER SQ. INCH	% ELONGATION IN 8 IN.	% REDUCTION OF AREA	BEND	MANUFACTURERS CHEMICAL ANALYSIS CAR.	MANG.	PHOS.	SUL.	Cu.
	SPECIFIED LIMITS (BASIC OPEN HEARTH)			0.5 TS	60/72000	1,500,000 TS		Cold 180°			Acid .06 Basic .04	Max. .05	
	CAR W&LE-71816 Shipped 8/28/37												
1	Angle 8 x 8 x 56.9 x 22'5"	1276	906524	38970	63510	24.00		OK	.22	.50	.012	.027	.29
	CAR EJ&E-31136 Shipped 8/25/37												
2	Beams CB-142 x 58 x 27'3-1/2"	3166	906524	39100	63970	26.25		OK	.22	.50	.012	.027	.29
	CAR EJ&E-31143 Shipped 8/23/37												
2	Beams CB-145 x 119 x 38'3"	9104	906534	35960	66010	25.00		OK	.20	.63	.014	.028	.29
	CAR EJ&E-31582 Shipped 8/27/37												
1	Beams CB-122 x 45 x 12'6"	563	153524	37690	65290	26.25		OK	.23	.60	.013	.033	.30
1	" do. 12'6"	563	1022578	39100	63250	26.25		OK	.18	.61	.011	.027	.32
3	" do. 13'2-1/2"	1783	ditto										
1	" do. 14'9"	664	do.										
			CLIENT: ATCHISON TOPEKA & SANTA FE R.R. CO.										

TOTAL THIS REPORT 17,119#
PREVIOUSLY SHIPPED
TOTAL TO DATE 17,119#

Robert W. Hunt Co

Fig. 22. Sample Report of Inspection Tests
Courtesy of A. T. & S. F. Railway System

Axial compression, columns, gross section . . . . . $12{,}000-60\frac{l}{r}$
with maximum of 10,000
l = length of member in inches.
r = the least radius of gyration of the member in inches.

Concrete. Concrete is used to some extent in practically every building. It is used for foundations, floors, walls, fireproofing, etc. A widely used concrete specification is that of the American Concrete Institute.

In the design of reinforced concrete the symbol $f'c$ is used to denote the ultimate compressive strength at 28 days. The term fc equals the compressive unit stress in the extreme fiber and is usually taken as $.35 f'c$ or $.40 f'c$. The A.C.I. Building Code requires an $f'c$ value of not less than 3,000 pounds per square inch for exposed concrete in severe climates. This specification also requires that for average materials the water-content per sack of cement shall not exceed the following values.

Assumed Strength of Concrete Mixtures

Water-content per 94-lb. sack of cement............U. S. gallons	$7\frac{1}{2}$	$6\frac{3}{4}$	6	5
Assumed compressive strength at 28 days.....pounds per square inch	2,000	2,500	3,000	3,750

The most important factor in determining the strength of concrete is the water-cement ratio. Concrete should be made as dry as consistent with workability.

The A.I.S.C. Specification requires a unit bearing stress on concrete of 600 pounds unless otherwise specified. Some specifications, notably the American Railway Engineering Association's Masonry Specification, allow a unit bearing pressure based on the 28 day ($f'c$) strength. The A.R.E.A. Specification allows a bearing pressure under steel base slabs of columns of $.35 f'c$. This would amount to a unit pressure of 700 pounds per square inch if 2,000-pound concrete is used, and 1,050 pounds per square inch for 3,000-pound concrete.

Masonry. Structural steel is frequently supported by walls, piers, footings, or foundations of masonry. The unit stresses in the accompanying table are taken from the *Recommended Minimum Requirements for Masonry Wall Construction* published by the Bureau of Standards in 1926.

Portland cement mortar is defined as a mortar composed of one volume of Portland cement to not more than three volumes of sand.

Cement lime mortar is defined as a mortar composed of one volume Portland cement, one volume hydrated lime, and not more than

Masonry Stresses

The maximum allowable compressive stresses in masonry due to combined live and dead loads shall not exceed the following:

Unit	Maximum Unit Working Stresses (Pounds per Square Inch Gross Area of Section)		
	Portland Cement Mortar	Natural Cement or Cement Lime Mortar	Lime Mortar
Clay brick (medium grade)	170	130	90
Sand lime brick	170	130	90
Concrete brick	170	130	70
Hollow tile, Concrete block, Concrete tile, Hollow brick walls	80	70 {Natural cement not to be used}	...
Ashlar masonry			
Granite	800	640	400
Limestone	500	400	250
Marble	500	400	250
Sandstone	400	320	160
Rubble masonry	140	100	70

six volumes of sand, except for hollow walls when not more than four volumes of sand shall be mixed with one volume of cement and one of hydrated lime.

Natural cement mortar is defined as a mortar composed of one volume natural cement to not more than three volumes of sand. *Not used for hollow walls.*

Lime mortar is composed of one volume slaked lime (lime putty) or hydrated lime to not more than four volumes of sand.

Timber. Structural timber is now graded and sold on the basis of unit working stresses. The various stress grades are identified by designations 1,800f, 1,600f, 1,200c, 1,100c, etc., and represent the safe unit working stresses applicable to material when used in locations continuously dry or covered. f = fiber stress in pounds per square inch for joists, planks, beams, and stringers. c = compression stress in pounds per square inch for short columns, posts, and timbers. The two tables presented give the standard stress grades and working stresses for timbers in bending and compression.

Beams and Stringers—Standard Stress-Grades and Working Stresses

Grades and Species	Fiber Stress in Bending or Tension	Maximum Horizontal Shear	Compression Perpendicular to Grain	Modulus of Elasticity
Softwoods				
1800 f. Dense Douglas fir (coast and inland)	1800	120	380	1,600,000
1800 f. Dense longleaf and shortleaf southern pine	1800	120	380	1,600,000
1600 f. Close-grained Douglas fir (coast)	1600	100	345	1,600,000
1600 f. Dense longleaf and shortleaf southern pine	1600	120	380	1,600,000
1600 f. Close-grained redwood	1600	80	267	1,200,000
1400 f. Southern cypress	1400	120	300	1,200,000
1400 f. Dense longleaf southern pine	1400	100	380	1,600,000
1400 f. Close-grained redwood	1400	80	267	1,200,000
1200 f. Dense shortleaf southern pine	1200	100	380	1,600,000
1200 f. Close-grained redwood	1200	70	267	1,200,000
1100 f. Port Orford cedar	1100	80	250	1,200,000
1100 f. Southern cypress	1100	100	300	1,200,000
1100 f. Eastern hemlock	1100	70	300	1,100,000
1000 f. Western red cedar	1000	100	200	1,000,000
Hardwoods				
1600 f. White ash, red and white oak	1600	120	500	1,500,000
1600 f. Beech, birch, hard maple	1600	120	500	1,600,000
1600 f. Rock elm	1600	120	500	1,300,000
1400 f. White ash, red and white oak	1400	120	500	1,500,000
1400 f. Beech, birch, hard maple	1400	120	500	1,600,000
1400 f. Rock elm	1400	120	500	1,300,000
1200 f. White ash, red and white oak	1200	100	500	1,500,000
1200 f. Beech, birch, hard maple	1200	100	500	1,600,000
1200 f. Rock elm	1200	100	500	1,300,000
1200 f. Soft elm	1200	100	250	1,200,000
1200 f. Black gum, red gum, tupelo	1200	100	300	1,200,000
1000 f. Chestnut	1000	100	300	1,000,000
1000 f. Soft elm	1000	100	250	1,200,000
1000 f. Black gum, red gum, tupelo	1000	100	300	1,200,000

Posts and Timbers—Standard Stress-Grades and Working Stresses

Grades and Species	Compression Parallel to Grain, Short Columns	Compression Perpendicular to Grain	Modulus of Elasticity
Softwoods			
1300 c. Dense Douglas fir (coast and inland)	1300	380	1,600,000
1300 c. Dense longleaf and shortleaf southern pine	1300	380	1,600,000
1200 c. Southern cypress	1200	300	1,200,000
1200 c. Close-grained Douglas fir (coast)	1200	345	1,600,000
1200 c. Dense longleaf and shortleaf southern pine	1200	380	1,600,000
1200 c. Close-grained redwood	1200	267	1,200,000
1100 c. Douglas fir (coast)	1100	325	1,600,000
1100 c. Close-grained redwood	1100	267	1,200,000
1000 c. Port Orford cedar	1000	250	1,200,000
1000 c. Southern cypress	1000	300	1,200,000
1000 c. Dense longleaf southern pine	1000	380	1,600,000
1000 c. Close-grained redwood	1000	267	1,200,000
900 c. Port Orford cedar	900	250	1,200,000
900 c. Dense shortleaf southern pine	900	380	1,600,000
800 c. Western red cedar	800	200	1,000,000
Hardwoods			
1300 c. Beech, birch, hard maple	1300	500	1,600,000
1300 c. Rock elm	1300	500	1,300,000
1200 c. White ash	1200	500	1,500,000
1200 c. Beech, birch, hard maple	1200	500	1,600,000
1200 c. Rock elm	1200	500	1,300,000
1100 c. Red and white oak	1100	500	1,500,000
1000 c. White ash	1000	500	1,500,000
1000 c. Beech, birch, hard maple	1000	500	1,600,000
1000 c. Rock elm	1000	500	1,300,000
1000 c. Red and white oak	1000	500	1,500,000
900 c. Chestnut	900	300	1,000,000
900 c. Soft elm	900	250	1,200,000
900 c. Black gum, red gum, tupelo	900	300	1,200,000

CHAPTER IV

STRUCTURAL SECTIONS

Classification of Sections. Structural steel members are generally designated by the shape of their cross sections. However, a distinction has to be made between the regular American Standard I beams and the Wide-Flange beams.

The other important sections are channels, angles, zees, 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.

Regular and Special Sections. Structural steel handbooks classify the various sections under two headings; namely, "Regular" and "Special." Under the classification "Regular Shapes," are the sections and sizes for which there is a constant demand and which can be procured from the mill readily in any size lots. Under the heading "Special Shapes" are shown the shapes and sizes for which there is a fluctuating demand and which are therefore rolled infrequently and then only by special arrangement. Consequently, the use of special sections should be avoided unless the quantity of any one size is sufficient to warrant a rolling.

Wide-Flange Beams. These beams are shaped like a capital letter I; however, as the name implies, most of the sections have flanges wider than the American Standard I beams. All wide-flange sections produced by the Carnegie-Illinois Steel Company mills have parallel face flanges, see Fig. 23. Wide-flange sections produced by the Bethlehem Steel Company have parallel face flanges with the following exceptions: all sizes with nominal depths from 36 to 16 inches inclusive (see Fig. 24); 14-inch, 30 pounds to 42 pounds; 12-inch, 25 pounds to 36 pounds; 10-inch, 21 pounds to 29 pounds; and 8-inch, 17 pounds to 21 pounds. These sections have a 5 per-cent slope on the inside faces of the flanges. A wide-flange beam of 36-inch nominal depth and weighing 300 pounds per foot is designated thus, 36″ WF300#.

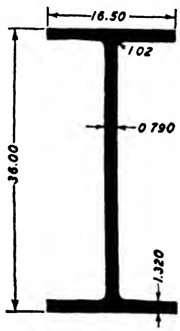


Fig. 23. A 36″ WF 240# Beam with Parallel Face Flanges

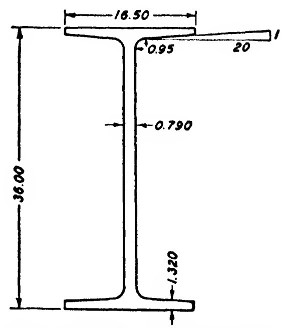


Fig. 24. A 36″ WF 240# Bethlehem Beam without Parallel Face Flanges

Wide-flange beams are so rolled that a higher efficiency per pound of steel is secured than with the American Standard I beams. This is attained by spreading both the horizontal and vertical rolls a varying amount, see Fig. 25. Equal thicknesses of metal are added to the web thickness and flange width and proportional additions are made to the flange thickness.

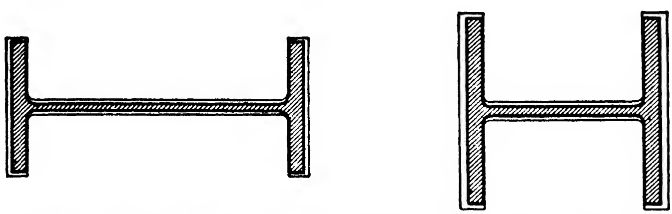


Fig. 25. Showing Method of Spreading Rolls to Increase Section of WF Beams

There is a wide range of wide-flange column sections. The weights of these sections vary from really light sections to 426 pounds per foot. These column sections are designed so they have a large moment of inertia about the minor axis.

Problems. 1. Look up in one of the steel handbooks the depth and weight of the deepest wide-flange beam rolled. What is the actual depth, width of flange, moment of inertia, and section modulus?

2. What is the moment of inertia about the major and minor axes of a 14″ WF 426# column section? Note the actual depth, width of flange, thickness of web, and thickness of flange.

American Standard I Beams. An I beam, Fig. 26, is designated by its depth and weight per linear foot, the same as a wide-flange beam, thus: 20″ I 100#. The deepest and heaviest I beam rolled is the 24″ I 120#. Since the advent of the wide-flange beams, the use of I beams has been restricted. These I beams are different from the wide-flange beams in that the depth of the beams in the various groups remains constant though the weight varies. The beams also have narrower flanges, and consequently a less moment of inertia about the minor axis.

The beams in a group are made with the same rolls, the minimum section being made with the rolls set close together, and the heavier sections are then made by spreading the rolls. In this manner the flanges are widened the same amount as the web is made thicker. In

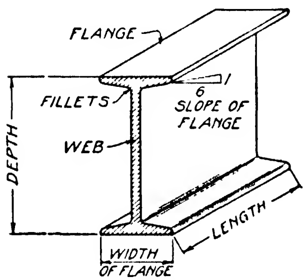


Fig. 26. Detail of an American Standard I Beam

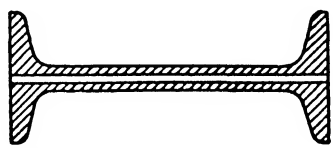


Fig 27. Showing Method of Increasing Section of American Standard I Beams

Fig. 27 the shaded portion represents the minimum section and the unshaded represents the metal added to produce the heavier section. These I beams are used where it is not more economical to use a wide-flange beam; also where it is required to have a thick web, narrow flange, or constant depth.

Problem. Look up the moment of inertia about the major axis of a 24″ I 120# and a 24″ WF 120#. How much greater, expressed in per cent, is the moment of inertia of the wide-flange beam?

American Standard Channels. A channel (Fig. 28), like I beam sections, is designated by its depth and weight per linear foot, thus: 15″ [33.9#. The maximum depth and weight of an American Standard channel is 15″ [55#. A group of 18″ channels is rolled having a maximum weight of 58# per linear foot; however, these sections are special car and shipbuilding channels.

Channels in a group are rolled with the same rolls, the heavier sections being made by spreading the rolls, see Fig. 29.

Due to the small moment of inertia about the minor axis, channels require more lateral support than I beams to keep them from buckling. They are therefore used only where conditions make them suitable. This occurs around openings in floors, against walls, and as roof purlins where nailing strips are to be bolted on. They are also used in wall spandrels or lintels, etc. One of the most important uses of channels is in built-up members, such as columns and truss members. For this purpose they are connected together with lacing, tie plates, cover plates, or with an I beam between them.

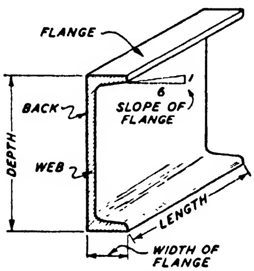


Fig. 28. Detail of an American Standard Channel Section

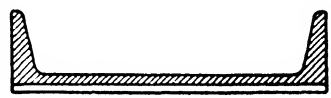


Fig. 29. Showing Method of Increasing Section of American Standard Channel

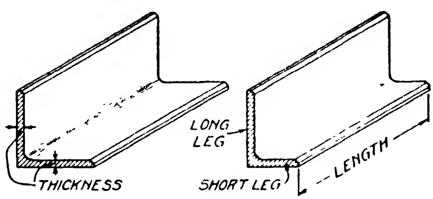


Fig. 30. Details of Angle Sections

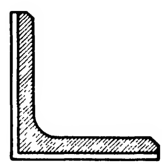


Fig. 31. Showing Method of Increasing Section of Angles

Problem. Look up in a steel handbook the moment of inertia about the major and minor axes of the following sections: 15″ [50#, 15″ I 50#, and 16″ WF 50#.

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, thus:

$$L\ 4'' \times 4'' \times \tfrac{5}{8}''$$
$$L\ 6'' \times 3\tfrac{1}{2}'' \times \tfrac{3}{8}''$$

The standard sizes of angles with equal legs are 1, $1\frac{1}{4}$, $1\frac{1}{2}$, $1\frac{3}{4}$, 2, $2\frac{1}{2}$, 3, $3\frac{1}{2}$, 4, 5, 6, and 8 inches.

The standard sizes of angles with unequal legs are $1\frac{3}{4}\times1\frac{1}{4}$, $2\times1\frac{1}{2}$, $2\frac{1}{2}\times2$, $3\times2\frac{1}{2}$, $3\frac{1}{2}\times3$, 4×3, $4\times3\frac{1}{2}$, $5\times3\frac{1}{2}$, 6×4, 7×4, 8×4, and 8×6 inches. The only special angle sections are a few with unequal legs.

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 except in special cases. 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.

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.

Plates and Bars. Flat steel is classified as bars, plates, or bearing plates. Flat steel six inches or less in width is called bars. Flat steel over six inches in width is classified as plates. When plates are over six inches in width and more than two inches in thickness, they are called bearing plates.

There are two styles of plates; namely, Universal Mill Plates and Sheared Plates. The Universal Mill Plate is edge-rolled to a finished width, whereas the Sheared Plate is sheared or flame-cut from a larger plate.

The sizes of plates produced by the different mills vary, therefore mill catalogs should be consulted in all except standard cases. The thickness of plates varies by sixteenths of an inch. Variation in the width of plates may be by $\frac{1}{2}$ inch; however, preferred practice is to vary by one inch. Plates are designated by their width and thickness, thus: Pl. $48''\times\frac{7}{16}''$.

Bearing plates are used extensively as column bases. These plates are rolled in various thicknesses up to and including eight inches. Up to and including two inches the plates are rolled flat. From two inches to four inches in thickness they can be straightened in a press. When these plates are over four inches in thickness, they should be planed.

A special type of plate is the Checkered plate. They are used for stair treads, walkway surfaces, etc. These plates have various styles of raised patterns. The nominal or ordered thickness is that of the flat plate exclusive of the height of the raised pattern. Fig. 32 shows one style of Checkered plate.

H Sections. These sections are also referred to as light column sections. They are designated by the depth and weight per linear foot, thus: 6″ **H** 20# or 6″×6″×20#. These sections are made with a flange width approximately the same as the depth. When larger and heavier sections than **H** sections are required, wide-flange sections can be used.

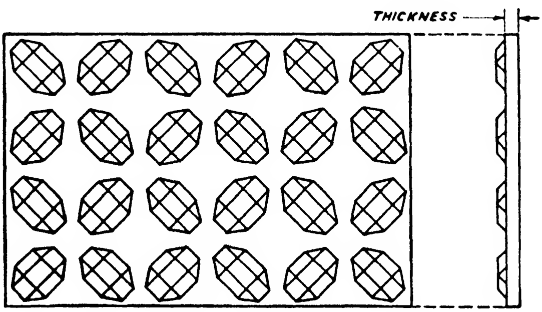


Fig. 32. One Style of Checkered Plate

Miscellaneous Sections. In addition to the regular structural sections just described, there are a number of special sections, Fig. 33, with which the designer should be familiar, *viz:*

(*a*) Bearing Piles
(*b*) Sheet Piles
(*c*) Subway Columns
(*d*) Structural Tees
(*e*) Tees
(*f*) Car and Ship-Building Channels
(*g*) Bulb Angles
(*h*) Zees
(*i*) Crane Rails

Rolling and Cutting Tolerances. All structural sections and plates are rolled and cut with certain allowed variations from exact dimensions. These allowed variations are called tolerances. The various steel handbooks devote several pages to the allowed tolerances for the different sections and plates. As an example, the allowed tolerance in depth for a wide-flange beam is plus or minus $\frac{1}{8}$ inch. The tolerance in the width of flange is plus $\frac{1}{4}$ inch and minus $\frac{3}{16}$ inch. The allowed tolerance on the length of leg of a 6-inch angle is plus or minus $\frac{1}{8}$ inch.

Problem. What are the allowed tolerances in depth and width of flange for a 12″ I 50#?

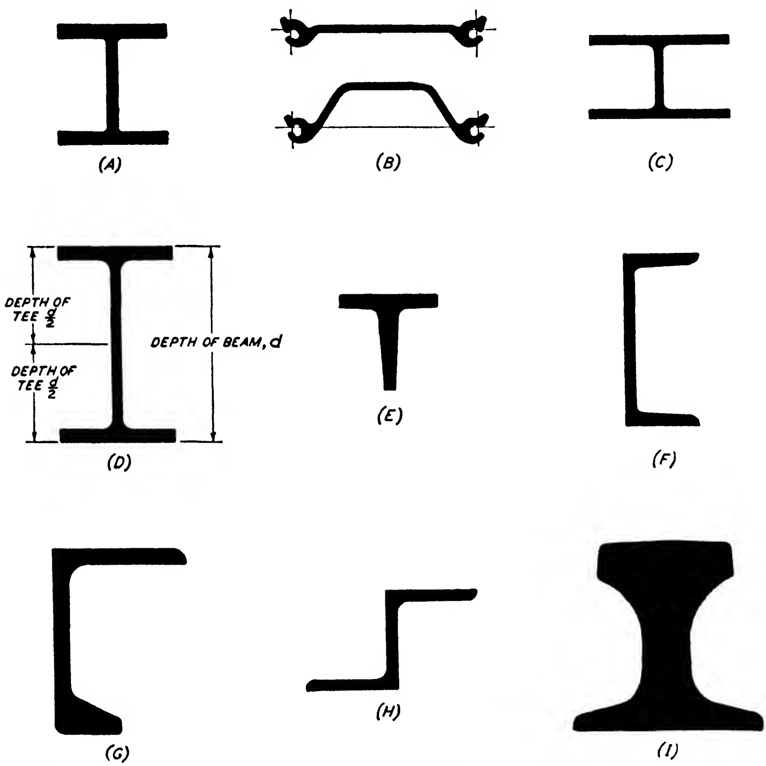


Fig. 33. Miscellaneous Sections. (*A*) Bearing Piles; (*B*) Sheet Piles; (*C*) Subway Columns; (*D*) Structural Tees; (*E*) Tees; (*F*) Car and Ship-building Channels; (*G*) Bulb Angles; (*H*) Zees; (*I*) Crane Rails

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.). The I beam, **H** section, and **Z**, Fig. 34, being symmetrical about both axes, the center of gravity is in the center of the web and no values are given in the handbook tables. The **C** section, Fig. 34, 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.

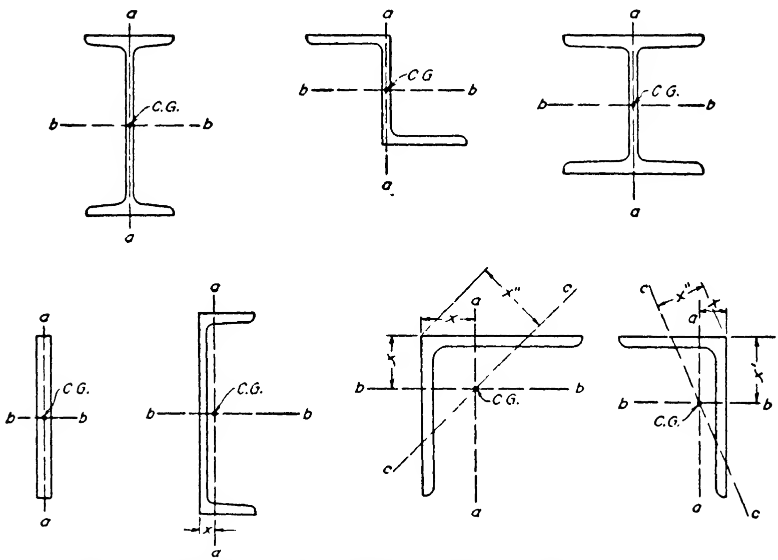


Fig. 34. Location of Center of Gravity of Sections Values of x, x', and x'' to be taken from Tables in Handbook

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. 34.

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.

To locate the center of gravity of an irregular surface, divide the surface into regular areas whose centers of gravity are known, figure the areas of these parts, pick two axes of reference, and figure for each axis the sum of the area moments of the parts about this axis, that is, multiply each area by the distance of its center of gravity from the axis and add the products. Divide this total area moment by the total area and you obtain the distance of the center of gravity from the axis of reference. The two distances obtained locate the center of gravity.

Illustrative Example. Compute the position of the center of gravity of L $4'' \times 4'' \times \frac{1}{2}''$, disregarding fillets and rounded corners, Fig. 35. Divide the angle into two rectangles (1) and (2) as shown. Their centers of gravity are at c_1 and c_2.

$$\text{Area of (1)} \quad 4\ '' \times \tfrac{1}{2}'' = 2.00 \text{ sq. in.}$$
$$\text{Area of (2)} \quad 3\tfrac{1}{2}'' \times \tfrac{1}{2}'' = 1.75 \text{ sq. in.}$$
$$\text{Total area} \quad 3.75 \text{ sq. in.}$$

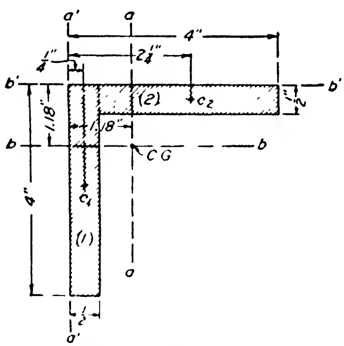


Fig. 35 Diagram Showing Computation of Position of Center of Gravity of Angle

Pick the edges $a'a'$ and $b'b'$ as reference axes.

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

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

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 $5'' \times 3'' \times \frac{3}{8}''$
15″ [33.9#

Moment of Inertia (I). 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 minimum I is given so that designers can figure the value of these angles, channels, and beams when used as compression members. The smaller the moment of inertia, the smaller is the radius of gyration, a quantity which indicates the narrowness of a column in any given direction. When a column buckles, it does so around the axis of minimum moment of inertia.

The moment of inertia enters into the formulas for bending and for deflection. It is also used in computing the radius of gyration. 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.*

To find the moment of inertia about any axis *a-a* of an irregular shaped area, proceed as follows:

Step 1. Divide the area into parts of which you know the centers of gravity and the moments of inertia.

Step 2. Look up the area and the moment of inertia of each part in the handbook. Be careful to pick the moment of inertia about an axis parallel to the given axis *a-a*.

Step 3. Figure the moment of inertia of rectangles with the formula $I_{cg} = \frac{1}{12} bd^3$ in which b is the dimension parallel to the axis *a-a* and d is the depth taken at right angles to *a-a*.

Step 4. Multiply each area by the square of the distance of its center of gravity from the axis *a-a*. Rivet holes count as negative areas.

Step. 5. Add all the products from Step 4 to the moments of inertia obtained from Steps 2 and 3. The grand total is the moment of inertia (I_a) about the axis *a-a*.

To summarize: I_a = Sum of I_{cg} + Sum of Ar^2 or the moment of inertia about *a-a* (I_a) equals the sum of the moments of inertia of the individual parts around parallel axes through their centers of gravity (obtained from the handbooks) plus the sum of the products obtained when each area is multiplied by the square of its center of gravity distance from the axis *a-a*.

Usually the new moment of inertia is taken about the axis through the center of gravity of the combined area.

Sometimes a shape or area is not given in the textbook and its own I_{cg} must be figured. If it is a rectangle, it is figured as given in Step 3. If it is not a rectangle, it must be divided into rectangles whose sides are parallel and at right angles to the axis *a-a*, and each part figured separately.

Illustrative Examples. 1. Compute I_a and I_b for the plate shown in Fig. 36.

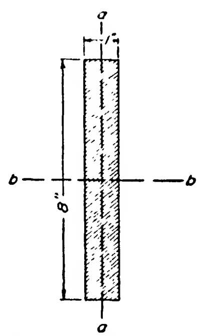


Fig. 36. Diagram for Moment of Inertia of Rectangular Plate

This problem illustrates computations similar to those of Step 3. The formula is

$$I_{cg}=\tfrac{1}{12}bd^3$$

For axis *a-a*, $b=8''$ $d=1''$

For axis *b-b*, $b=1''$ $d=8''$

Therefore

$$I_a=\tfrac{1}{12}\times 8\times 1\times 1\times 1=\tfrac{2}{3}\text{ in.}^4$$

$$I_b=\tfrac{1}{12}\times 1\times 8\times 8\times 8=42\tfrac{2}{3}\text{ in.}^4$$

2. Compute I_a for the plate girder section in Fig. 37 made of 1 Pl. $42''\times\frac{1}{2}''$ and 4 Ls $6''\times 6''\times\frac{1}{2}''$. (Steps 1 and 2)

for 1 Pl. $42''\times\frac{1}{2}''$ I_g (from tables) $=3087$ in.^{4}

for 4 Ls $6''\times 6''\times\frac{1}{2}''$ I_c (from tables) $4\times 19.91=$ 80 in.^{4}

for 4 Ls $6''\times 6''\times\frac{1}{2}''$ I_a $4\times 5.75\times 19.57\times 19.57=\underline{8809}$ in.^{4}

11,976 in.^{4}

Deductions for rivet holes at *m* (Step 4)

Area of 2 holes $=2\times1\frac{1}{2}''\times\frac{7}{8}''=2.625$ sq. in.
For 1 hole $I_d=\frac{1}{12}\times1\frac{1}{2}''\times\frac{7}{8}''\times\frac{7}{8}''\times\frac{7}{8}''=.08$ in.^{4}
(a value so small that it is neglected) (Step 3)
$I_a=2.625\times18.75\times18.75=$ 923 in.^{4}

Total net value I_a = 11,053 in.^{4}

Attention is called to the fact that only in special cases is it necessary to deduct for the rivet holes in designing plate girders for building work. The gross moment of inertia and gross section modulus can be used. See section 19 (a) of the A.I.S.C. Specification, in the Appendix.

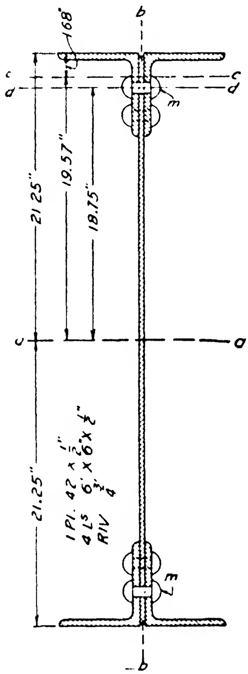


Fig 37 Diagram for Moment of Inertia of Plate Girder

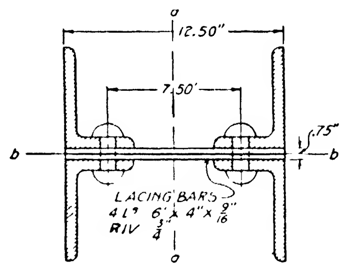


Fig. 38A Diagram for Moment of Inertia of Four Angles

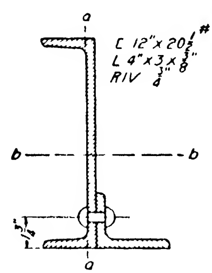


Fig. 38B Diagram for Moment of Inertia of Channel and Angle

Problems. 1. Compute the values of I for the section in Fig. 38A. Deduct rivet holes. The section is made up of 4 Ls 6"×4"×$\frac{9}{16}$" connected with lacing bars (lacing not figured).

2. Compute the values of I for the section shown in Fig. 38B.

1 [12"×20.7#
1 L 4"×3"×$\frac{3}{8}$"

The axes *aa* and *bb* are through the center of gravity. The section not being symmetrical, the position of the center of gravity must be computed.

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.

The section modulus $\frac{I}{c}$ is a very important quantity. It is the measure of the section's ability to withstand bending. Multiplying the section modulus by the safe unit stress in bending (20,000 lbs. per sq. in.) gives us the safe resisting moment of the section in inch-pounds. If this moment is greater than the bending moment on the section, we know that the section is strong enough.

Illustrative Examples. 1. Compute $\frac{I}{c}$ and the resisting moment for a 30″ WF 190# beam about the axis perpendicular to the web. From the handbook, $I=8825.9$. The distance c is half the depth $=15.06$.

$$\frac{I}{c}=\frac{8825.9}{15.06}=586.1 \text{ in.}^3$$

$$\begin{aligned}\text{Resisting moment} &= \frac{I}{c}\times 20{,}000 = 586.1\times 20{,}000\\ &= 11{,}722{,}000 \text{ in.-lbs.}\\ &= 11{,}722{,}000 \div 12\\ &= 976{,}833 \text{ ft.-lbs.}\end{aligned}$$

2. Compute $\frac{I}{c}$ and the resisting moment for a 12″ ⊏ 20.7# about the axis parallel to the web. Not being a symmetrical section, it has two values. From handbook, $I=3.9$; $c=2.94-.70=2.24$, and $c=0.70$.

$$\therefore \qquad \frac{I}{c}=\frac{3.9}{2.24}=1.74 \text{ and } \frac{I}{c}=\frac{3.9}{0.7}=5.57 \text{ in.}^3$$

$$\text{Resisting moment} = \frac{I}{c} \times 20{,}000 = 1.74 \times 20{,}000$$
$$= 34{,}800 \text{ in.-lbs.}$$
$$= 2{,}900 \text{ ft.-lbs.}$$

also,
$$\text{Resisting moment} = \frac{I}{c} \times 20{,}000 = 5.57 \times 20{,}000$$
$$= 111{,}400 \text{ in.-lbs.}$$
$$= 111{,}400 \div 12 = 9{,}283 \text{ ft.-lbs.}$$

For an unsymmetrical section having two values of $\frac{I}{c}$ the smaller value should always be used. The smaller value is the one shown in the handbooks.

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

14″ WF 167# about axis parallel to web
36″ WF 260# about axis perpendicular to web
15″ I 45# 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

Radius of Gyration (r). The radius of gyration, represented by r, is expressed algebraically as follows:

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

The radius of gyration in reference to a certain axis may be defined as the distance from this axis at which the area of the section may be concentrated to produce the moment of inertia of the section. Half the area can be considered above the axis and half the area below the axis.

Illustrative Examples. 1. Referring to Fig. 36, 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 $r = \sqrt{5\frac{1}{3}} = 2.31''$.

In other words if the area $A = 8$ square inches were concentrated at a distance of 2.31″ from b-b (Fig. 36) half (4 sq. in.) above the line and half below the line, the new section would have the same I_b ($42\frac{2}{3}$ in.^{4}) as the old. Fig. 39 illustrates this view of the radius of gyration.

2. Referring to Fig. 37, the value of $I_a = 11{,}976$ (disregarding rivet holes). To find the radius of gyration.

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

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

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

Again we can consider the total area of this example concentrated at 16.5″ from the axis a-a, 22 square inches above the axis and 22 square inches below the axis.

The radius of gyration is referred to in structural engineering, usually in connection with compression members. It indicates, in a way, the narrowness or stiffness of a member. The smaller the radius

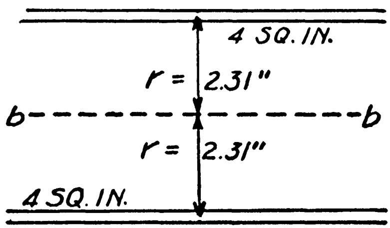


Fig. 39. Diagram for Radius of Gyration

$$I_b=Ar^2=8\times 2.31\times 2.31\times 42\tfrac{2}{3} \text{ in.}^4$$

of gyration, the more easily will the section bend or buckle when used in compression; and conversely, the greater the radius of gyration, the greater the resistance to bending or buckling.

Problems. 1. Compute the values of r for the sections given in Figs. 38A and 38B.

2. Check the values given in the handbook for r for a 12″ WF 65#.

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

GENERAL INFORMATION

Price Basis. It is desirable that the designer be posted on the basis of prices for structural steel. There are several basing points for steel prices. The delivered price in any case will be the closest basing-point price plus the freight rate. The basing points and prices per hundred pounds (first quarter 1938) for structural shapes and plates are as follows:

Bethlehem, Pa.	$2.35
Buffalo, N. Y.	2.35
Pittsburgh, Pa.	2.25
Chicago, Ill.	2.30
Gary, Ind.	2.30
Birmingham, Ala.	2.40
Gulf Ports	2.65
Pacific Coast Ports	2.80

Certain sizes are called "base" sizes and are sold at the base price. For those sections that are not base sizes, an extra charge is added to the base price.

The base sizes are: Angles, 3 to 6 inches; American Standard I Beams, 3 to 15 inches; Channels, 3 to 15 inches; Zees, 3 to 6 inches. The extra charge for other sizes is usually 10 cents per hundred pounds. All **H** beams and wide-flange bearing pile sections carry an extra charge of 10 cents per hundred pounds. All wide-flange beams have an extra charge of 5 to 20 cents per hundred pounds, the higher charge being for the deeper sections. If a quantity of less than 6000 pounds is ordered, an extra is also charged. The same applies if the lengths of the pieces are other than 8 to 80 feet.

There is an added charge of 10 cents per hundred pounds for copper-bearing steel. Alloy steels, of course, are higher priced. Structural silicon steel costs an additional 40 to 75 cents per hundred pounds, depending on the quantity ordered, the smaller charge applying on quantities over 75 tons.

The base prices, of course, vary slightly from time to time.

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 50 cents to $1.50 per hundred pounds.

Stocks of plain material are carried in all the larger cities.

Whether mill or stock material will be used depends upon the size of the job and the time service required. If delivery of fabricated material is required within 5 or 6 weeks, it will usually have to be taken from stock.

Variation in Weight. Attention is called to the provision in the specifications in the Appendix, which permits a slight variation in the weight of the finished steel as compared with its theoretical weight. This variation, in the case of structural shapes and universal mill plates up to and including 36 inches in width, is 2.5 per cent above or below the theoretical weight or cross-sectional area. This represents the practical limits in adjusting the rolls of the mill, and also wear in the rolls. 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, a much larger variation is allowed, amounting in some cases to as much as 19 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.

CHAPTER V

FABRICATION

General. The fabrication of structural steel consists in the cutting (Fig. 40), punching, drilling, assembling, riveting, welding, and painting of the material, as required. The fabricating companies also maintain engineering departments that prepare the detail drawings

Fig. 40. Large Plate Shear with Turntable
Courtesy of Thomas Machine Manufacturing Company, Pittsburgh, Pennsylvania

used by the fabricating shop (Fig. 41). Some structural steel is fabricated by welding; however, the major portion of the work is still done by the riveted method.

RIVETS

A system of conventional rivet signs is used by the designer and steel detailer to indicate the kind of rivets desired. Fig. 42 shows the conventional signs for riveting.

Rivet Sizes. The size of rivets to be used depends in general upon the stresses involved and the width and thickness of material used. In building work the rivet sizes most commonly used are $\frac{3}{4}$ and $\frac{7}{8}$ inch in diameter. Specifications require holes to be drilled or sub-punched,

Fig. 41. New Structural Steel Fabricating Shop, Moore Drydock Co., Oakland, California
Courtesy of Thomas Machine Manufacturing Company, Pittsburgh, Pennsylvania

and reamed if the material is the thickness of the rivet diameter plus $\frac{1}{8}$ inch. Therefore if any appreciable amount of the steel in a building is $\frac{7}{8}$ inch thick or thicker, it will be economical to use $\frac{7}{8}$-inch rivets.

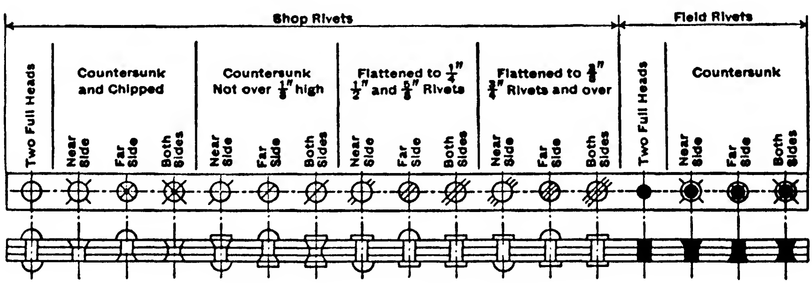


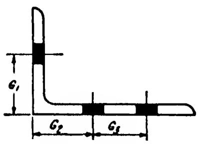
Fig. 42. Conventional Signs for Riveting

For very heavy work 1-inch, $1\frac{1}{8}$-inch, and $1\frac{1}{4}$-inch rivets are used. In Table I is shown the maximum size of rivets that can be used in various sizes of angles.

The use of more than one size of rivets on a job is to be avoided as much as practicable. It is inconvenient and costly to punch more

TABLE I

Gages for Angles



Leg	8"	7"	6"	5"	4"	3½"	3"
G1	4½"	4"	3½"	3"	2½"	2"	1¾"
G2	3"	2½"	2¼"	2"	...	...	...
G3	3"	3"	2½"	1¾"	...	...	...
Max. Rivet	1⅛"	1"	⅞"	⅞"	⅞"	⅞"	⅞"

Leg	2½"	2"	1¾"	1½"	1⅜"	1¼"	1"	¾"
G1	1⅜"	1⅛"	1"	⅞"	⅞"	¾"	⅝"	½"
G2	...	...	...	...	...	...	...	...
G3	...	...	...	...	...	...	...	...
Max. Rivet	¾"	⅝"	½"	⅜"	⅜"	⅜"	¼"	¼"

than one size of hole or drive more than one size of rivet in a structural member.

Spacing Rivets. 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.

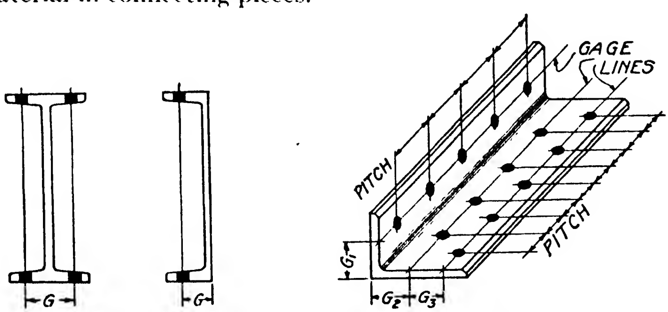
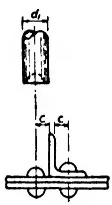


Fig. 43 Diagrams Showing Gage and Pitch Lines

Gage. The term "gage" is used to designate the spacing of rivet lines parallel to the axis of the member. For example, Fig. 43 illustrates the gage lines of beams, channels, and angles. Standard values are assigned in the handbooks to the gage lines in the flanges of I beams and channels, and in angles. Table I gives the gages for angles.

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

Pitch. By the pitch of rivets is meant the spacing along the gage lines, Fig. 43. Some of the rules for this spacing are given in the A.I.S.C. Specifications shown in the Appendix. 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 $\frac{7}{8}$-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.



	Diameter of Rivet, d, Inches									
	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
d_1	$1\frac{1}{2}$	$1\frac{3}{4}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$	3	$3\frac{1}{4}$	$3\frac{1}{2}$	$3\frac{3}{4}$
c	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	2

Fig. 44. Clearance Allowed for Riveting

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 the A.I.S.C. Specifications in the Appendix.

Clearance. A hole cannot be punched close against the web of a 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 *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. 44.

Closely associated with the amount of clearance is the accessibility for driving the rivets, Fig. 45. For power driving, the rivet must be so situated that it can be brought between the jaws of the riveting

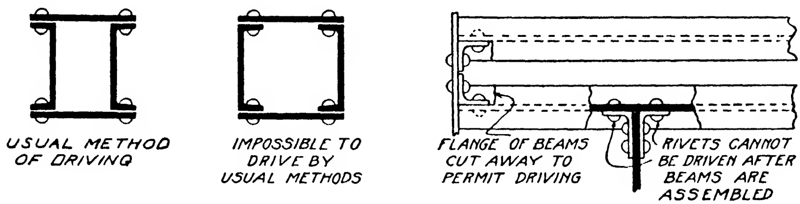


Fig 45. Difficult Situations for Riveting

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. 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. 45. This matter is generally looked after in making shop drawings, but needs some attention in designing.

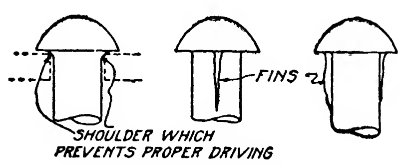
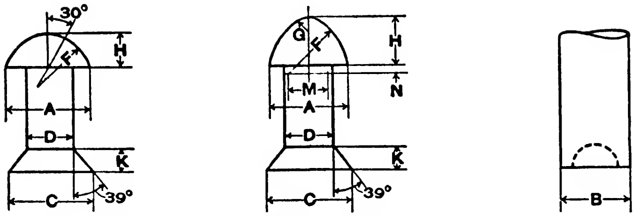


Fig. 46. Defective Rivets

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. 46. If the corners of the die become



DRIVEN HEADS MANUFACTURED HEADS DIE

Manufactured Heads: American Institute of Bolt, Nut and Rivet Manufacturers, December, 1932

Diam. of Rivet, Inches				1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
Driven Head Inches	Button Head	A	1.5 D+1/8	7/8	1 1/16	1 1/4	1 7/16	1 5/8	1 13/16	2	2 3/16	2 3/8
		H	.425 A	3/8	7/16	9/16	5/8	11/16	3/4	7/8	15/16	1
		F	1.5 H	9/16	11/16	13/16	29/32	1 1/32	1 5/32	1 9/32	1 13/32	1 1/2
	Ctsk.	C		29/32	1 1/8	1 11/32	1 19/32	1 13/16	2 1/32	2 1/4	2 1/2	2 23/32
		K	.5 D	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4
Manufactured Head Inches	High Button Head	A	1.5 D+1/32	13/16	1	1 3/16	1 3/8	1 9/16	1 3/4	1 15/16	2 1/8	2 5/16
		H	.75 D+1/8	1/2	5/8	11/16	13/16	7/8	1	1 1/16	1 3/16	1 1/4
		F	.75 D+9/32	21/32	3/4	27/32	15/16	1 1/32	1 1/8	1 7/32	1 5/16	1 13/32
		M	.50	1/2	1/2	1/2	1/2	1/2	1/2	1/2	1/2	1/2
		N	.093	3/32	3/32	3/32	3/32	3/32	3/32	3/32	3/32	3/32
		G	.75 D−9/32	3/32	3/16	9/32	3/8	15/32	9/16	21/32	3/4	27/32
	Ctsk.	C		29/32	1 1/8	1 11/32	1 19/32	1 13/16	2 1/32	2 1/4	2 1/2	2 23/32
		K	.5 D	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4
Die, In.		B		1 3/4	2	2 1/4	2 1/2	2 3/4	3	3 1/4	3 1/2	3 3/4

Fig. 47. Dimensions of Structural Rivets

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.

Button Head. 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 so that the diameter at

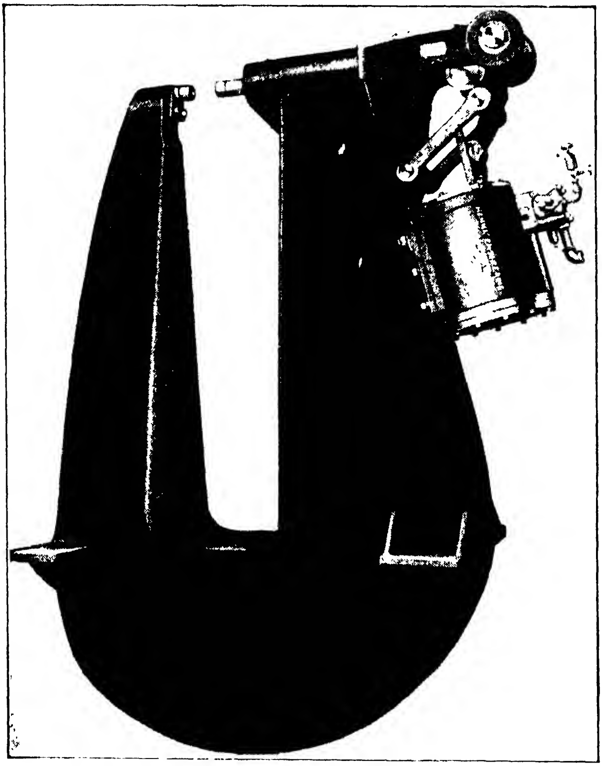


Fig. 48. Bull Riveter, 75-Inch Reach, 21-Inch Gap, 70 Tons Pressure
Courtesy of Hanna Engineering Works, Chicago, Illinois

the outside is greater than at the inside of the plate. The sizes of rivet heads are shown in Fig. 47. 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.

Countersunk rivets are more expensive than button-head rivets, therefore they should not be used unless absolutely required by conditions.

Driving Rivets. 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

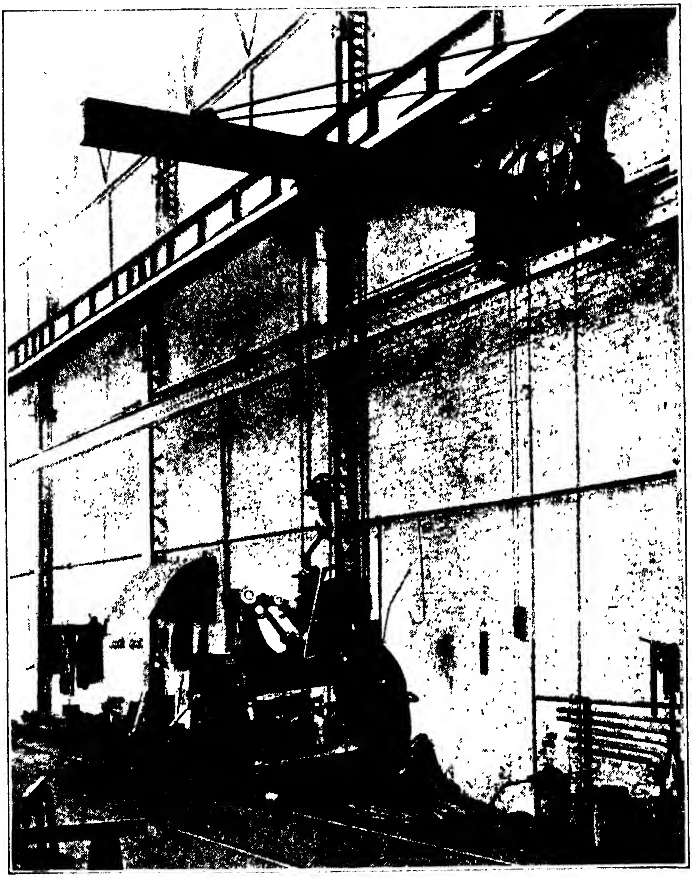


Fig. 49. Yoke Riveter Suspended from Traveling Jib Crane
Courtesy of Hanna Engineering Works, Chicago, Illinois

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 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.

Riveting Machines in Shop. In the shop, rivets are driven with

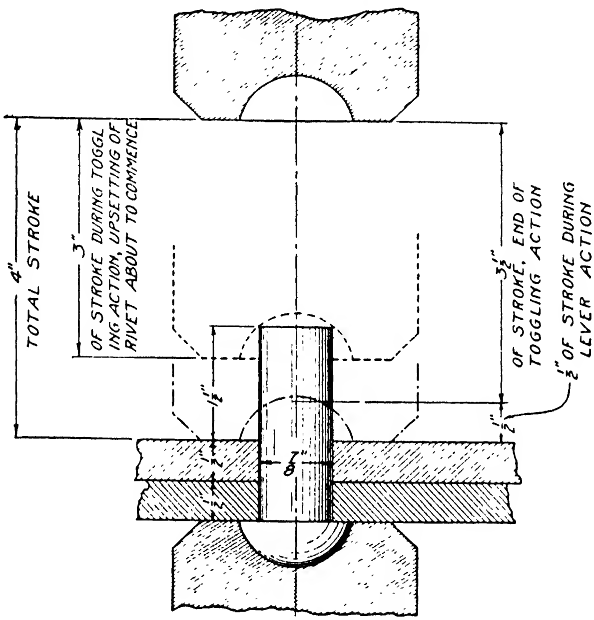


Fig. 50. Rivet Ready for Driving

pneumatic riveters called Yoke or Bull Riveters, Figs. 48 and 49. The machines consist essentially of a yoke which spans the members to be riveted, Fig. 50. 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. 51. The power is applied by means of hydraulic or pneumatic pressure. The pressure 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 with

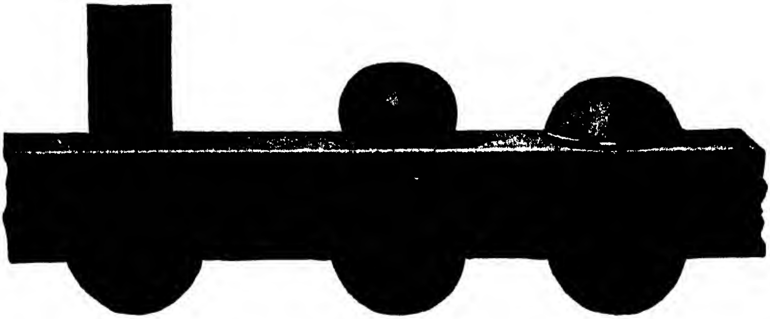


Fig. 51. Three Stages in Process of Riveting

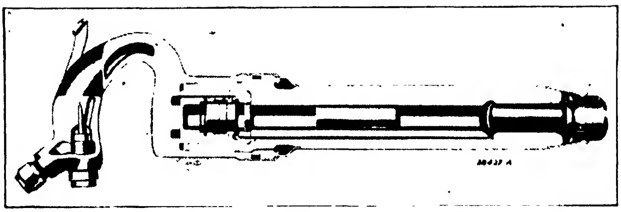


Fig. 52. Sectional View of a Pneumatic Riveting Hammer
Courtesy of Ingersoll-Rand Company, New York, New York

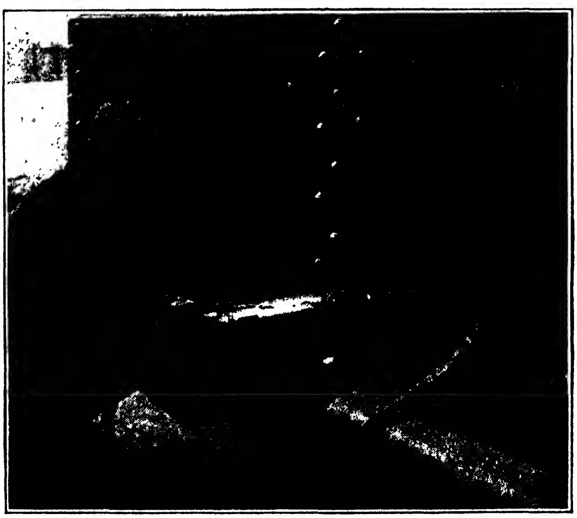


Fig. 53. Driving Rivets with Pneumatic Hammer and Bucking with Jam Riveter
Courtesy of Ingersoll-Rand Company, New York, New York

a pneumatic hammer. The rivet is inserted in the hole and held in place by means of a die or bucking-up tool pressed against the head of the rivet. Pneumatic bucking-up machines are also used. The pneumatic riveter, or air gun, Fig. 52, 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 done by means of the pneumatic hammer.

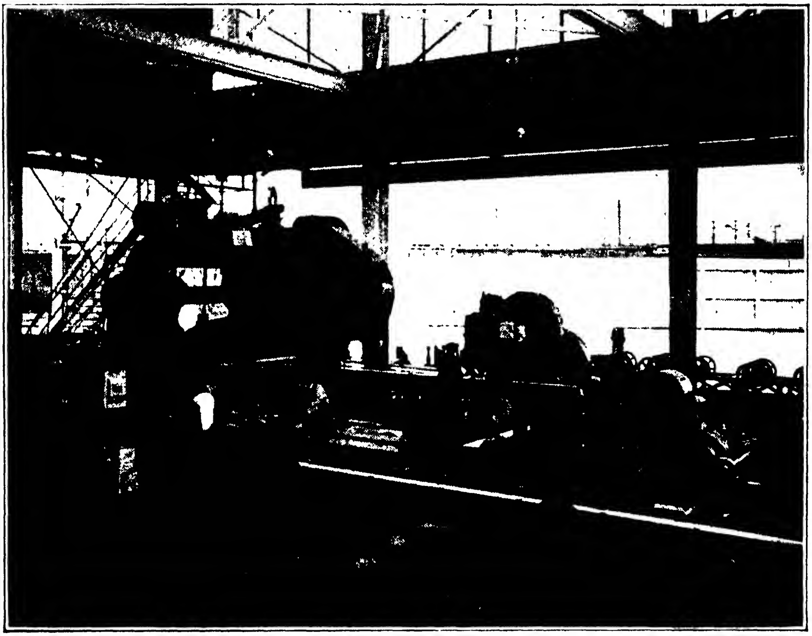


Fig. 54. Multiple Punch with Automatic Spacing Table
Courtesy of Thomas Machine Manufacturing Company, Pittsburgh, Pennsylvania

Fig. 53 shows riveting being done with a pneumatic riveting hammer and a pneumatic bucking-up tool called a jam riveter.

Punching and Reaming. Rivet holes in structural steel work are ordinarily punched in the metal by means of powerful punching machines. Fig. 54 shows a multiple punch with a spacing table. This spacing table automatically spaces the holes to be punched. The punching machine shown in the picture can punch two sections at a time or one plate up to 96 inches in width. In punching machines the die is made $\frac{3}{32}$ inch larger than the punch, and is placed directly under the punch 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.

For material not thicker than the diameter of the rivet plus $\frac{1}{8}$ inch, the holes are accurate enough; however, for thicker material it

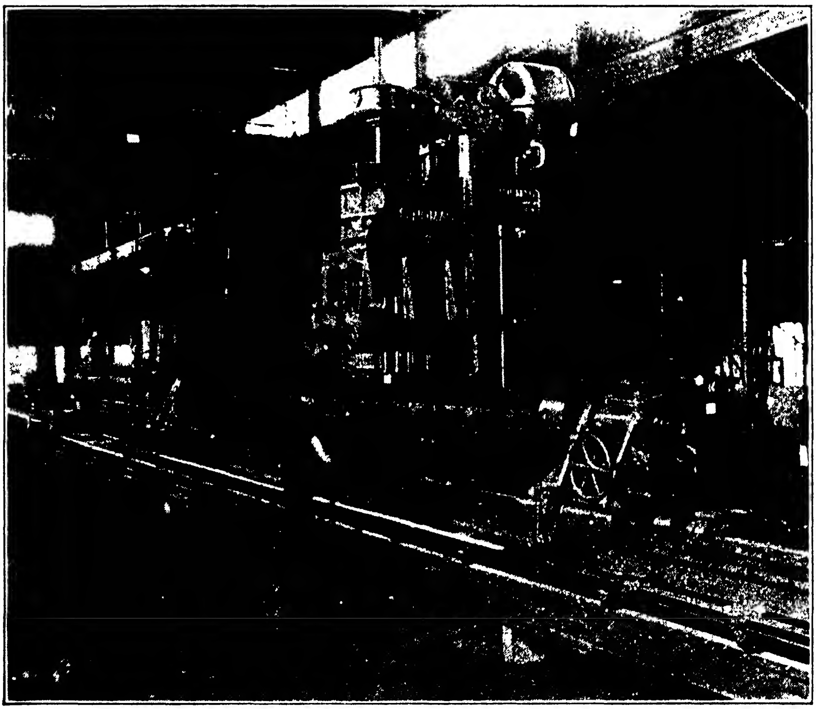


Fig. 55. Multiple Drill for Drilling Columns
Courtesy of Thomas Machine Manufacturing Company, Pittsburgh, Pennsylvania

is required that the holes be either drilled or sub-punched and reamed. When the material is sub-punched and reamed, a hole is punched smaller than the size of the rivet—called "sub-punching"—and it is then enlarged to proper size by means of a reamer. Fig. 55 shows a multiple drill used for drilling heavy column sections. This machine can rotate the sections so that holes can be drilled in the web or either flange.

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 usually used for this purpose unless there is some 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.

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.

This is the case when the rivets after cooling not only hold the plates tightly together but have been unable to contract the normal amount due to the drop in temperature. The rivet then develops tension in its shank and is pressing the riveted plates together with forces of from 8000 to 10,000 pounds.

Tests conducted at the University of Illinois showed that hot-driven rivets with button heads developed an initial tension of 70%, or more, of the yield-point strength of the rod from which they were made. If the grip of the rivet was 3 inches, the initial tension was found equal to 90% of the yield-point strength.

The load path or stress path from plate to plate will pass by skin friction along the contact surfaces between the plates. To help make this friction effective is one of the reasons for omitting fitting-up paint.

Thus in Fig. 56 at *a* the load of 4000 pounds will pass from the $\frac{3}{8}$-inch plate *n* to the $\frac{1}{2}$-inch plate *m* through the skin friction of the surface of contact. This would only happen if the rivet is stressed in tension sufficiently to develop 4000 pounds friction between the plates. The rivet then will not be stressed in shear nor in bearing. Similarly in Fig. 56 at *b* if the rivet is sufficiently tensed, the 8000 pounds can be transmitted from plate *m* to plates *l* and *n* by skin friction along the two surfaces of contact. The rivet acts just like a clamp with three sheets of paper between its jaws.

It was formerly the custom to specify that rivets should not be 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

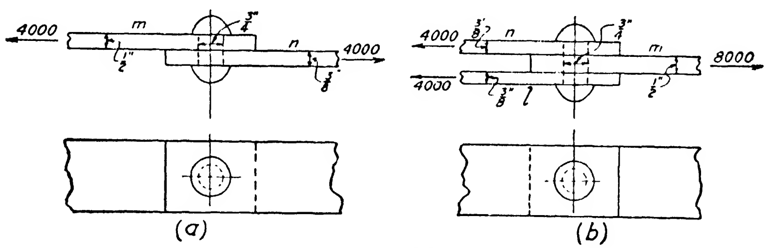


Fig. 56. Diagrams Showing Stresses in Rivets

reliable in this respect and, if properly driven, can be subjected to tension as safely as bolts. If, however, the rivet is not in high tension, it must transmit the loads in bearing, in shear, and in bending.

Bearing. Fig. 56 at *a* represents two pieces, *m* and *n*, riveted together, so that the stress (4000 pounds) in *m* is transmitted to *n*. Fig. 56 at *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*.

When the connection is such that the rivets tend to shear along one plane, the rivets are in single shear; likewise if the connection is such that the rivets tend to shear along two planes, they are in double shear. The A.I.S.C. specification allows a unit bearing stress on rivets in single shear of 32,000 pounds per square inch and in double shear, 40,000 pounds per square inch.

In Fig. 56 at *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 bearing allowed in single shear is 32,000 pounds per square inch; thus the allowed values in bearing are

for m $\quad 0.375 \times 32{,}000 = 12{,}000\#$

for n $\quad 0.281 \times 32{,}000 = 8{,}992\#$

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

Problem. Compute the bearing values for m and n using $\frac{7}{8}$-inch rivets.

In Fig. 56 at b the stress is transmitted from plate m to the plates l and n and divided equally between them. In this connection the rivet is in double shear. The maximum working stress in bearing for plates l and n is 32,000 pounds per square inch, whereas for plate m it is 40,000 pounds per square inch.

The bearing areas are

for m $\quad \frac{1}{2} \times \frac{3}{4} = 0.375$ sq. in.

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

The allowed values in bearing are

for m $\quad 0.375 \times 40{,}000 = 15{,}000\#$

for l and n combined $\quad 0.5625 \times 32{,}000 = 18{,}000\#$

It will be noticed that for this connection the bearing on the middle plate m governs. When using the above unit bearing stresses, it will not be necessary to compute the bearing on the outside plates if their combined thickness is 25 per cent greater than the thickness of the center plate.

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.

Problem. Compute the bearing values for l, m, and n for $\frac{7}{8}$-inch rivets. Assume $m = 1$ inch thick and l and n, $\frac{5}{8}$ inch thick. The combined thickness of plates l and n is what per cent of the thickness of plate m?

Shear. Referring again to Fig. 56 at a, the forces acting on the two plates tend to cut or shear the rivet. This shearing action is resisted by the cross-section area of the rivets. This sectional area is

$\frac{\pi d^2}{4}$, or $\frac{3.1416}{4} \times \frac{3}{4} \times \frac{3}{4}$, or 0.4418 sq. in. The unit stress allowed in shear on rivets is 15,000 pounds per square inch. Then the allowable value for one $\frac{3}{4}$-inch rivet is $15{,}000 \times 0.4418$ or 6627. This is greater than the actual stress applied and is sufficient.

Problem. Compute the shearing value of a $\frac{7}{8}$-inch rivet.

In Fig. 56 at *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 2×6627 or 13,254 pounds, and is sufficient to carry the actual load, which is 8000 pounds.

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.

Problem. Compute the shear value for 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.

Illustrative Example. In the case illustrated in Fig. 56 at *a*, what thickness of plate *n* is required to make the bearing value equal the shearing value? The shearing value is 6627 pounds. The bearing area required is $\frac{6627}{32{,}000}$ or 0.207 sq. in. The diameter of rivet being 0.75 in., the thickness required to give the required area is 0.207 sq. in.$\div$ 0.75 or 0.276 in. The next higher commercial size is 0.3125 in. or $\frac{5}{16}$ in. thick.

Problems. 1. In the case illustrated in Fig. 56 at *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, in single shear, are required to transmit 175,000 pounds?

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

5. Assume 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 thicknesses 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. We have seen that 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. The A.I.S.C. Specification allows a unit stress of 15,000 pounds per square inch on rivets in tension. It is desirable to avoid rivets in tension if possible to do so; however if conditions require their use, it is satisfactory to so use them.

Bending. Relatively short rivets have very little bending stress; however, in joints where filler plates are used, or the plates are many, or their thickness great, the bending stress becomes more important. It is difficult to compute the correct moment on rivets as the lever arm or span length that applies cannot be exactly determined. Also, the theory of flexure does not hold and conservative results are obtained when used for beams with such small values of span to depth ratio. To provide for the bending stress in long rivets, the A.I.S.C. Specifications require that where the grip of a rivet is more than five diameters, their number shall be increased one per cent for each additional $\frac{1}{16}$ inch in the rivet grip. It also requires that special care be used in heating and driving such rivets.

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.

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.

In a perfect design these three elements would be equal in value,

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

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. 57 illustrates a splice of two plates, each $7'' \times \frac{1}{2}''$. Rivets $\frac{3}{4}''$ diameter. Allowable stress in bearing is 32,000 pounds per square inch; in shear it is 15,000 pounds per square inch; and in tension it is 20,000 pounds per square inch.

(a) Using all of the ten rivets,

Shear value	$10 \times 6627 =$	66,270#
Bearing value	$10 \times 12{,}000 =$	120,000#
Tension value at (*1*)	$6\frac{1}{8} \times \frac{1}{2} \times 20{,}000 =$	61,250#
Tension value at (*2*)	$5\frac{1}{4} \times \frac{1}{2} \times 20{,}000 =$	52,500#
Loss of tension value between (*1*) and (*2*) =		8,750#

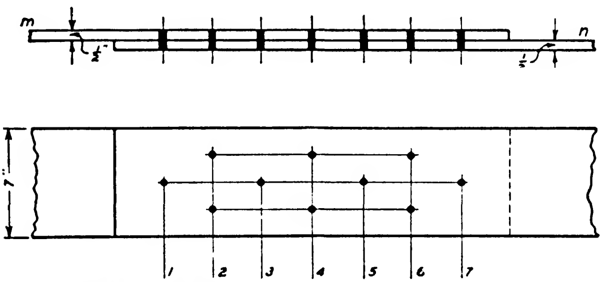


Fig. 57. Diagrammatic Views of a Riveted Joint

As this loss is more than the amount transmitted from *m* to *n* by the 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 $52{,}500 + 6627 = 59{,}127\#$.

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

Shear value	$8 \times 6627 =$	53,016#
Bearing value	$8 \times 12{,}000 =$	96,000#
Tension value at (*2*)	$5\frac{1}{4} \times \frac{1}{2} \times 20{,}000 =$	52,500#

The strength of the joint is the tension value at (*2*), i. e., 52,500#.

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

Shear value $8 \times 6627 =$ 53,016#

Tension value at (*2*) plus shear value of rivet at (*1*) as above = 59,127#

The strength of the joint is the shear value 53,016#.

(d) Finally omit the rivet at (3).
Shear value $9 \times 6627 = 59{,}643\#$
Strength of joint same as in (a) $= 59{,}127\#$

From the above it is clear that the maximum strength of the joint that can be made in this case is 59,127 pounds. It requires 9 rivets as in (d).

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.

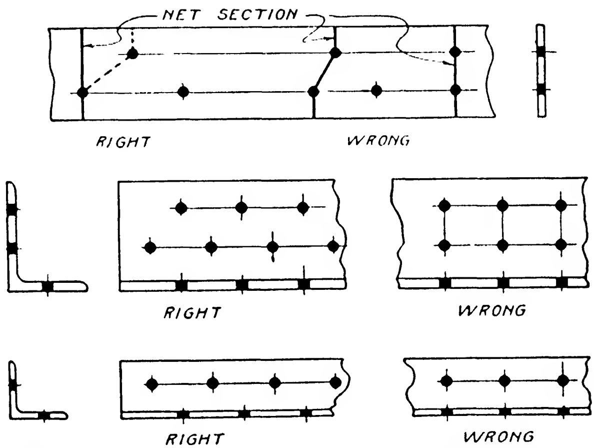


Fig. 58. Diagrams Showing Right and Wrong Arrangement of Rivet Holes in Tension Members

Whenever a rivet hole is punched through a plate, the tensile strength of the plate at the line of punching is reduced. The fewer the holes used in the first row of rivets, the less is the reduction in tensile strength. The plate at the second row of rivets has had some of its load transferred to the other plate by the first row of rivets. Each rivet, however, can only transfer its own value so that the plate at the second row of rivets is worth its own tensile strength plus the value of the rivets of the first row. From the point of view of the maximum value of riveted connections in tension the idea is to begin with one rivet in the end rows and increase the number of rivets in the other rows as fast as the tensile strength of the plate, the stress remaining in the plate, and the value of the rivets will permit.

Problem. Go through the operations of the given example on the basis of $\frac{7}{8}''$ rivets.

Net Section. Holes in tension members should be so spaced that maximum net section is secured. Unless space is limited, holes in plates or angles usually can be arranged so that only one hole need be deducted when two or three rows are used, and two holes out with four rows of holes. Fig. 58 illustrates the right and wrong arrangement of holes in a number of cases. In computing the net area, the diameter of a rivet hole is taken as $\frac{1}{8}$ inch greater than the nominal

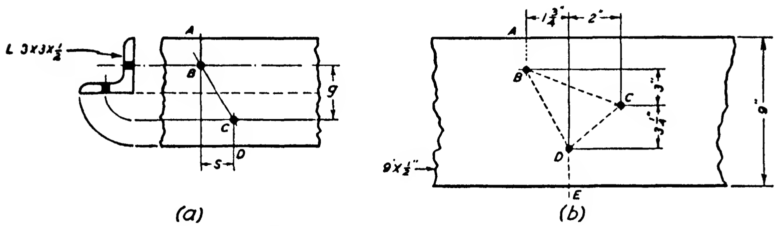


Fig. 59. Illustrating Net Width

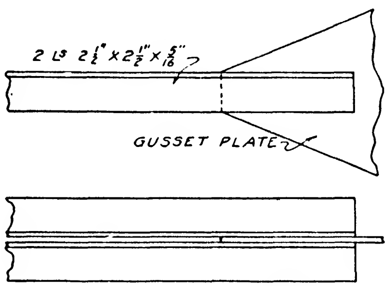


Fig. 60. Two Angles Attached to a Gusset Plate

diameter of the rivet. The A.I.S.C. Specifications require net section to be determined as follows:

(*c*) In the case of a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain, and adding, for each gage space in the chain, the quantity $\frac{s^2}{4g}$, where

s = longitudinal spacing (pitch) of any two successive holes.

g = transverse spacing (gage) of the same two holes.

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

(*d*) For angles, the gross width shall be the sum of the widths of the legs less the thickness. The gage for holes in opposite legs shall be the sum of the gages from back of angle less the thickness.

Illustrative Examples. In Fig. 59 at *a*, if the gage $g=3$ inches and the pitch $s=1\frac{1}{2}$ inches, with $\frac{3}{4}''$ rivets the net section will be as follows:

$$\text{Gross width} = 3''+3''-\tfrac{1}{2}'' = 5\tfrac{1}{2}''$$

Net width on zigzag line $A\ B\ C\ D$

$$= \text{gross width} - 2\times\tfrac{7}{8} + \frac{s^2}{4g}$$
$$= 5\tfrac{1}{2}'' - 1\tfrac{3}{4}'' + \frac{(1.5)^2}{4\times 3}$$
$$= 5.5 - 1.75 + 0.19$$
$$= 3.94 \text{ in.}$$

$$\text{Net section} = 3.94\times\tfrac{1}{2} = 1.97 \text{ sq. in.}$$

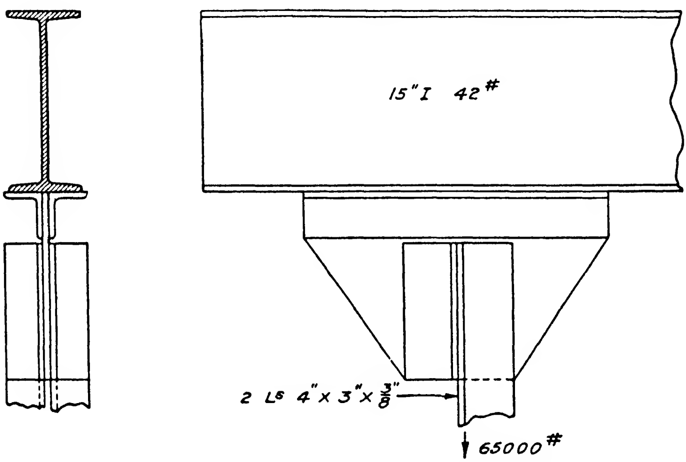


Fig. 61. Side and End View of a Riveted Hanger

The pitch of the rivets in this angle is such that it requires the deduction of nearly two holes.

Problem. In Fig. 59 at *a* what pitch would be required so that only one hole would have to be deducted?

In Fig. 59 at *b* it is necessary to investigate two possible zigzag lines to determine the minimum net section.

Section $A\ B\ C\ D\ E$

$$\text{Gross width} = 9''$$

$$\text{Net width} = 9 - 3\times\tfrac{7}{8} + \frac{(3.75)^2}{4\times 3} + \frac{2^2}{4\times 3\frac{1}{4}}$$
$$= 9 - 2.63 + 1.17 + .31$$
$$= 7.85 \text{ in.}$$

Section $A\ B\ D\ E$

$$\text{Net width} = 9 - 2\times\tfrac{7}{8} + \frac{(1.75)^2}{4\times 6\tfrac{1}{4}}$$
$$= 9 - 1.75 + 0.12$$
$$= 7.37 \text{ in.}$$
$$\text{Net section} = 7.37\times\tfrac{1}{2} = 3.685 \text{ sq. in.}$$

As the section along zigzag line $A\ B\ D\ E$ has the minimum net section, it governs.

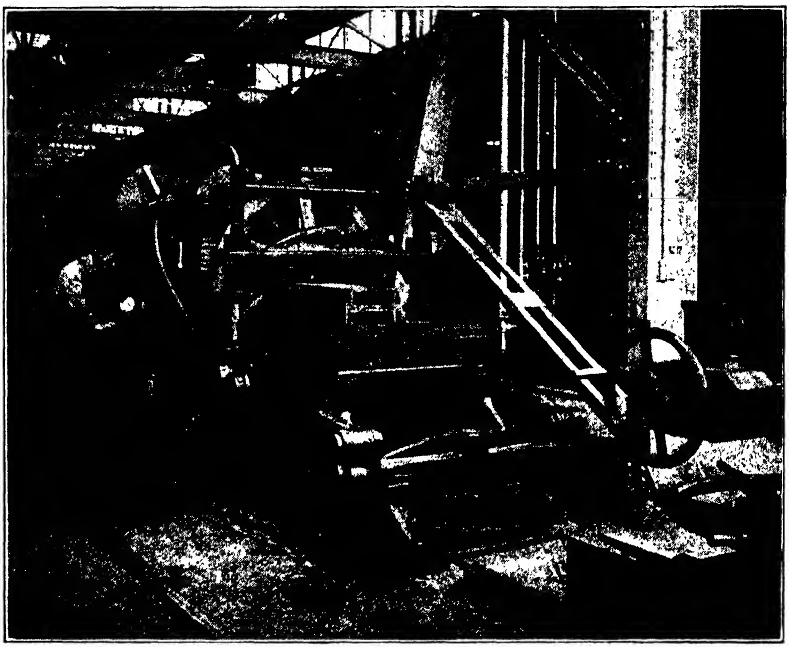


Fig. 62. Column Facing Machine
Courtesy of Thomas Machine Manufacturing Company, Pittsburgh, Pennsylvania

Problems. 1. Compute the minimum net section for Fig. 59 at b using $\frac{7}{8}''$ rivets.

2. Fig. 60 shows two angles in tension to be connected to a gusset plate with rivets. Determine the following:

Size of rivets
Net section of angles
Tension value of net section
Thickness of gusset plate to develop full double shearing value of rivets
Number of rivets
Locate gage line and space the rivets
Draw plan, elevation, and section of joint at $\frac{3}{4}$-inch scale

3. Fig. 61 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 rivets to connect lug angles to main angles (assume that one-half of load is transmitted through the lug angles)
Number of rivets to connect hanger to gusset plate
Number of rivets to connect gusset plate to top angles
Number of 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 handbook page numbers for all references used in these operations.

In the fabrication of steel the facing or milling of the ends of columns is an important procedure. Practically all building columns require one or both ends to be faced so that a uniform and true bearing is secured. In Fig. 62 is shown a column facing machine.

BOLTS

There are three types of bolts in structural use; namely, machine bolts, turned bolts, and rib bolts.

Machine Bolts. Machine bolts are made from rods as they come from the rolling mill and are not finished to exact size. These bolts are therefore referred to as unfinished bolts.

The A.I.S.C. Specifications allow a unit stress on machine bolts of about two thirds of those allowed for rivets or turned bolts, see Section 10-a of the specifications. Machine bolts are allowed for field connections in buildings under 125 feet in height, see Section 17-d of the A.I.S.C. Specifications. Machine bolts are used a great deal in the erection of industrial buildings, as they can then be readily salvaged or taken down and moved.

Holes for machine bolts are made $\frac{1}{16}$ inch larger than the nominal diameter of the bolt.

Turned Bolts. Turned bolts are made in an automatic lathe machine which turns the bolts to exact diameter. All holes for turned bolts are either drilled or sub-punched and reamed after the parts to be connected are assembled. These holes are usually made $\frac{1}{32}$ inch larger than the diameter of the bolt; however, the A.I.S.C. Specifications require that the hole be $\frac{1}{50}$ inch larger.

For turned bolts in reamed or drilled holes the same unit stresses are allowed as for rivets.

Rib Bolts. Rib bolts are a comparatively recent development. These bolts have ribbed or fluted shanks, the outside diameter being larger than the hole, so that on driving a bolt with a hammer the shank deforms and a tight fit results.

Tests have been made of multiple-bolt joints using this type of bolt and they were found to be as satisfactory as rivets in all respects. Advantages of this type of bolt are, the reduction in construction noise, elimination of the equipment necessary for riveting, and being able to make them of high-strength alloy or heat-treated steels. When

Fig. 63. Rivet Bolt with Dardelet Self-locking Thread
Courtesy of Dardelet Threadlock Corporation

these bolts are used, the holes in the connected members must match fairly, more so than for rivets; otherwise, reaming is necessary. These rib bolts are usually sold and used with a lock nut.

Fig. 63 shows a "Rivet Bolt." The nut has Dardelet self-locking thread. The nut is also cupped so as to eliminate the need for a washer.

Nuts and Washers. There are various types of nuts and washers in common use. Some types of nuts have locking devices incorporated in them, and others are also cupped on the under side so as to eliminate the need for washers. It is necessary that the shank of a bolt and none of the thread bear against the members connected.

A plain nut, of course, can not be turned down tight without using a washer unless the thread is cut long enough to extend into the hole. Therefore, unless a cupped type of nut is used, it is necessary to place a washer under the nut. Washers can be either plain cut washers or some type of spring or lock washer, of which there are several makes.

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 inch. 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 $0.30 \times 15{,}000$ or 4500 pounds.

Length of Rivets and Bolts. The grip of a rivet or bolt is the thickness of the material through which it passes.

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 washer, plus the thickness of the nut, plus $\frac{1}{2}$ inch.

BALCONY FRAMING IN KEITH MEMORIAL THEATRE, BOSTON, MASS.
Courtesy of American Institute of Steel Construction

CHAPTER VI

WELDING

General. In the last ten to fifteen years the welding of buildings by the electric arc process has come into common practice. There have been a considerable number of tall buildings with from ten to twenty stories either completely welded or field welded. In Fig. 64 is shown a typical welded building in the process of construction.

Advantages. The advantages of welded construction are as follows: It is a silent erection process compared to riveting. This is a very important consideration when making additions to buildings or building new structures near hospitals, schools, or other institutions, and in some business areas.

There is economy of main material, also of connection details. Changes in design and correction of errors can be made during erection with greater ease and less expense than if riveting were used.

Connections can be made to develop the full bending strength of beams and girders. This is important in buildings requiring special wind or earthquake bracing where it is desirable to make the girder to column connections rigid and develop the full bending resistance of the members. Also, bracing for wind or earthquake resistance is more compact and, for the same depth, has greater rigidity than riveting.

Welded construction also permits the use of continuous beams, which will allow the use of shallow beams or result in greater stiffness and less deflection.

Disadvantages. The disadvantages of welded construction are as follows: Making the plans for a welded structure requires more engineering work to take advantage of the saving in material; this is also the case in working out connection details.

Closer inspection is required, both in the shop and the field, than for riveted work. Welders must be tested and qualified at certain intervals.

Most fabricating shops are equipped to do economical riveted work, as the major portion of the work is still done that way.

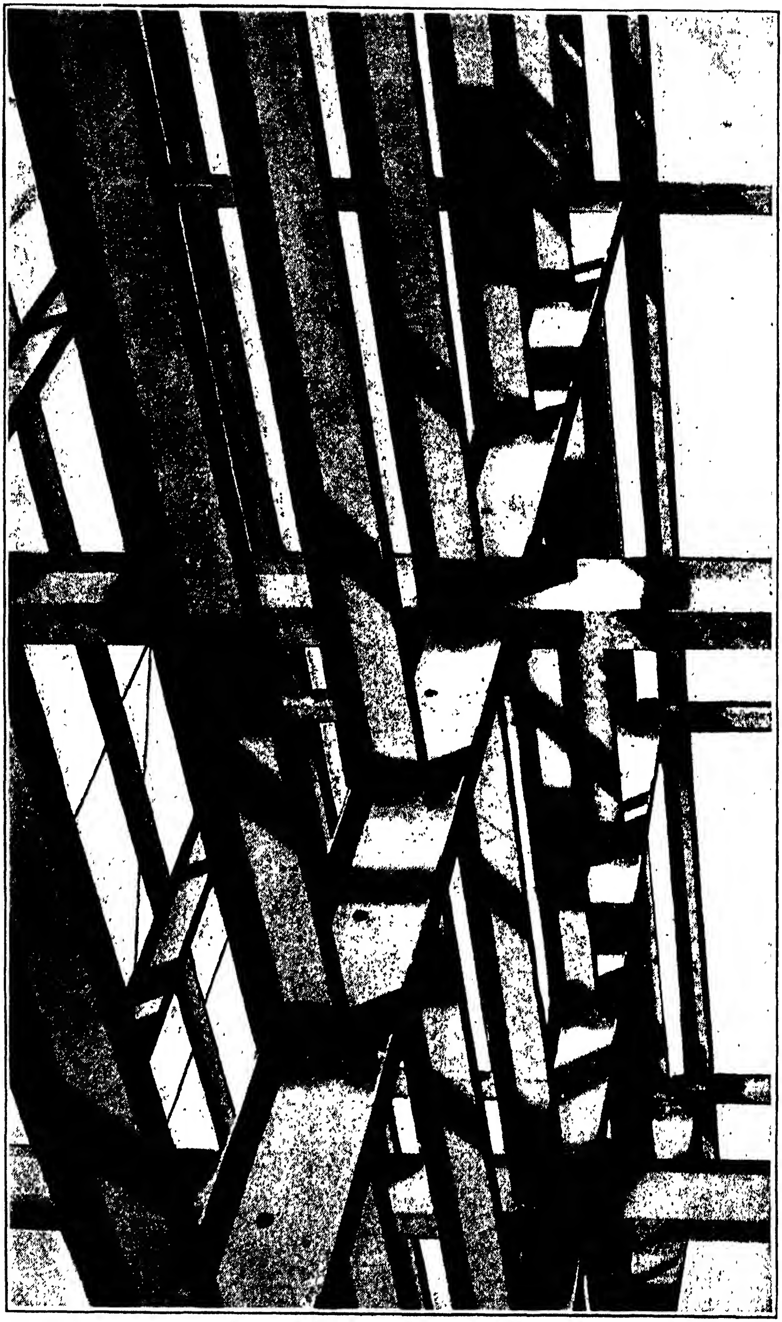


Fig. 64. Welded Building Construction
Courtesy of The Lincoln Electric Company, Cleveland, Ohio

Specifications. The outstanding specifications on welding as applied to buildings are those of the American Welding Society: Code 1—Code for Fusion Welding and Gas Cutting in Building Construction, Part A—Structural Steel; Code 2—Code for Resistance Welding of Structural Steel in Building Construction. These specifications cover the design, fabrication, and erection by welding of structural steel for buildings, and are printed in the Appendix.

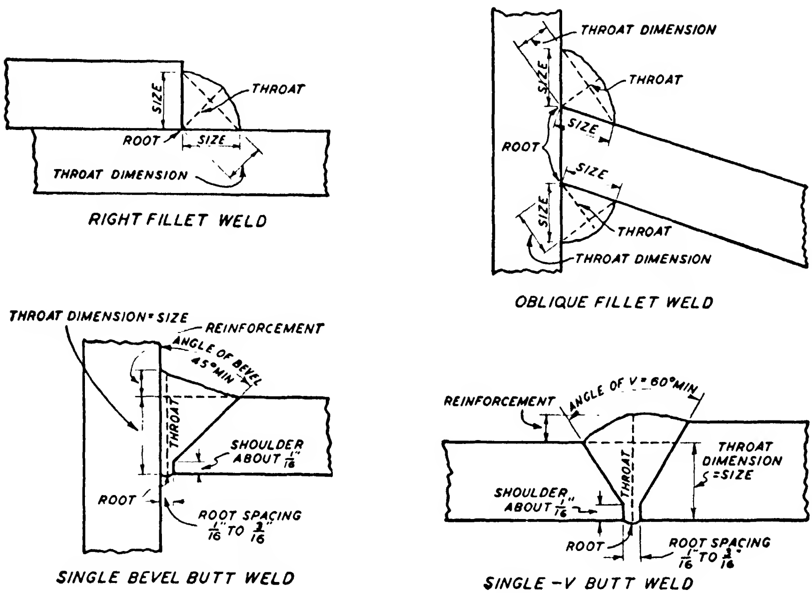


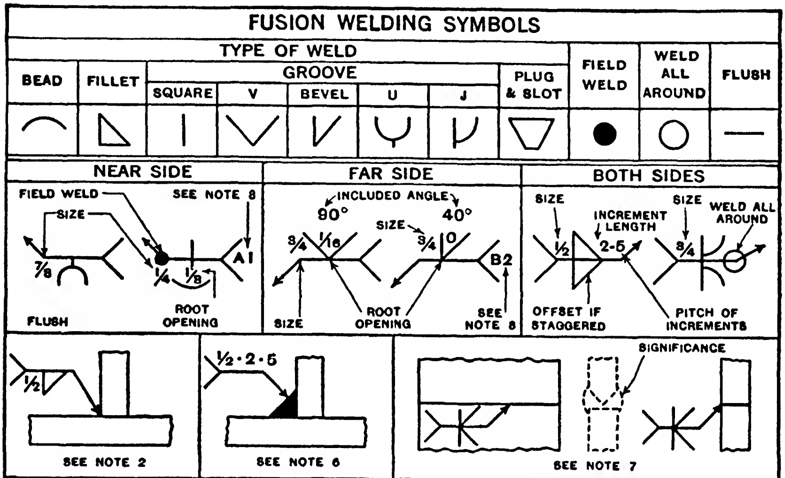
Fig. 65. Fillet and Butt Welds

The student should study these specifications and become familiar with the various features and requirements.

The A.W.S. Specifications allow the use of gas-welding as well as welding by the electric arc; however, for building work, the electric arc is used practically entirely. The electric arc method is usually more economical, also comparatively little heat is generated in the process. Gas welding employs an oxyacetylene flame, and it is necessary to heat the welding rod and contact surfaces of the members to be connected, to a molten stage—which heats up the members somewhat and may cause temperature stresses in the welded members.

Types of Welds and Symbols. Fig. 65 illustrates butt welds and fillet welds. When butt welds are used, one or both edges of the base-metal parts to be joined shall be beveled if the throat dimension exceeds $\frac{1}{4}$ inch, unless the weld is to transmit only compressive stress.

When welding inside a hole or slot in a plate or member to join it to an underlying part, fillet welds shall be used; however, the hole or



1. IN PLAN OR ELEVATION, NEAR, FAR AND BOTH SIDES LOCATIONS REFER TO NEAREST MEMBER PARALLEL TO PLANE OF DRAWING AND NOT TO OTHERS FARTHER BEHIND.
2. IN SECTION OR END VIEWS ONLY, WHEN WELD IS NOT DRAWN, THE SIDE TO WHICH ARROW POINTS IS CONSIDERED NEAR SIDE.
3. WELDS ON BOTH SIDES ARE OF SAME SIZE UNLESS OTHERWISE SHOWN.
4. SYMBOLS GOVERN TO BREAK IN CONTINUITY OF STRUCTURE OR TO EXTENT OF HATCHING OR DIMENSION LINES.
5. ALL WELDS ARE CONTINUOUS AND OF USER'S STANDARD PROPORTIONS AND ALL EXCEPT V-AND BEVEL-GROOVED WELDS ARE CLOSED UNLESS OTHERWISE SHOWN.
6. WHEN WELDS ARE DRAWN IN SECTION OR END VIEWS, OBVIOUS INFORMATION IS NOT GIVEN BY SYMBOL.
7. IN JOINTS IN WHICH ONE MEMBER ONLY IS TO BE GROOVED, ARROWS POINT TO THAT MEMBER.
8. TAIL OF ARROW USED FOR SPECIFICATION REFERENCE.

Fig. 66. Fusion Welding Symbols
Courtesy of American Welding Society

slot should not be filled or partially filled so as to form a weld-metal connection between opposite walls.

It is desirable for designers and detailers of welded steel to use conventional signs or symbols to indicate the type and location of welds, the same as is done in riveted work. Fig. 66 shows the symbols for indicating fusion welds on structural drawings.

Allowable Unit Stresses. The A.W.S. Specifications allow the following unit stresses in kips.

Kind of Stress	For Welds Made with Filler Metal of	
	Grade 2, 4, 10 or 15	Grade 20, 30 or 40
Shear on section through weld throat..........	13.6	11.3
Tension on section through weld throat........	15.6	13.0
Compression (crushing) on section through throat of butt weld........................	18.0	18.0

The stress in a fillet weld is considered as shear for any direction of the applied stress.

The following design shear strengths per linear inch for fillet welds are expressed in terms of the weld dimension.

Allowed Stress in Pounds per Linear Inch

Size of Weld Inch	Grade of Filler Metal		Size of Weld Inch	Grade of Filler Metal	
	20, 30, or 40	2, 4, 10, or 15		20, 30, or 40	2, 4, 10, or 15
1/4	2000	2400	1/2	4000	4800
5/16	2500	3000	5/8	5000	6000
3/8	3000	3600	3/4	6000	7200

Connections. There are no standard welded beam connections such as exist for riveted work. However, there has been developed a large variety of welded joints, which gives a wide choice to the designer. It is necessary to provide all beam, girder, and column connections with sufficient bolts and seats so that the structure can be erected and plumbed before field welding.

There has been some experimenting with, and use of, special slotted erection seats that are welded on columns in the shop, eliminating the use of erection bolts.

Welds are usually not made smaller than $\frac{1}{4}$ inch in size, unless the plates or members connected are less than $\frac{1}{4}$ inch in thickness. The cost of welds over $\frac{3}{8}$ inch in size increases rapidly. The following figures show the approximate speed on flat work and quantities of welding rod for different sizes of fillet welds.

Size of fillet welds.......................inch	1/4	3/8	1/2
Speed per hour...........................feet	12	7	5
Pounds of welding rod per foot of fillet weld...	0.2	0.4	0.6

Fig. 67. Fabricating Structural Steel by Electric Welding
Courtesy of The Lincoln Electric Company, Cleveland, Ohio

In designing or detailing welded joints, the designer should guard against calling for welds at points where they are difficult or impossible to place.

Fig. 67 shows a shop for fabricating structural steel by welding. In the right foreground is shown the electric arc-welding unit.

The design of various types of joints will be illustrated by examples. In these problems the allowed stress used will be that for filler metal of Grade 20, 30, or 40.

Illustrative Example 1. Required to splice a $6'' \times \frac{1}{2}''$ bar having a tension stress of 60,000#, see Fig. 68.

The splice plates will have to be 5 inches wide so as to leave room on the sides for a fillet weld.

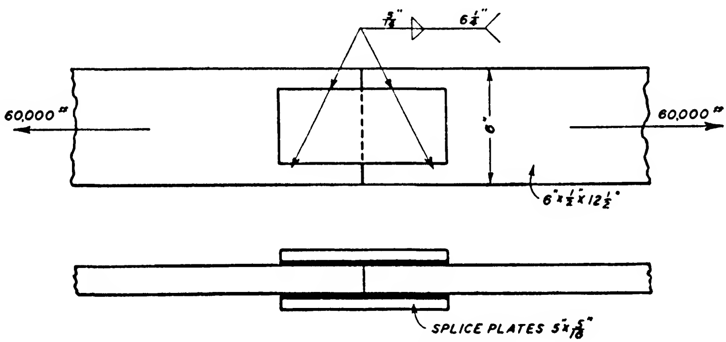


Fig. 68. Method of Splicing a Bar

$$\text{Thickness of splice plates} = \frac{60{,}000}{20{,}000 \times 5} = 0.6''$$

$\therefore$ use two $\frac{5}{16}''$ plates $= 0.625''$

As the splice plates are $\frac{5}{16}''$ thick it will be necessary to use a maximum fillet weld of $\frac{5}{16}''$.

$$\text{Length of } \tfrac{5}{16}'' \text{ fillet weld required} = \frac{60{,}000}{4 \times 2500} = 6''$$

The specification requires the over-all length of a fillet weld to be the theoretical length plus $\frac{1}{4}''$, therefore, $12\frac{1}{2}''$ long splice plates welded symmetrically over the joint should be used.

Shorter splice plates could be used if fillet welds were also placed on the ends; however, it is not desirable to place welds so as to cut the fibers of stress in the main members unless necessary. This is especially important where the members are subjected to reversals of stress.

The welds on these splice plates could also have been made $\frac{1}{4}$-inch fillets, which would have required slightly longer splice plates; however, this would have been perfectly satisfactory.

Illustrative Example 2. Required to weld a $4'' \times 3'' \times \frac{1}{2}''$ angle hanger to a plate. Assembled load, 65,000#; see Fig. 69.

If $\frac{3}{8}''$ fillet weld is used, the required length will be

$$\frac{65{,}000}{3000} = 21.7''.$$

As the stress acts along the center of gravity, it is necessary to make the welding l_2 longer than l_1 so that the resistance of the welds

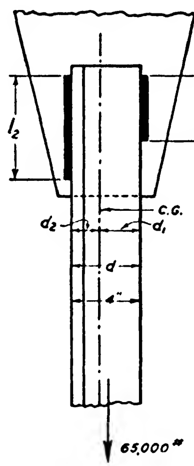


Fig. 69.
Welded Hanger

will coincide with the stress in the angle. The lengths of the welds l_2 and l_1 are computed as follows:

From the Steel Handbook we find $d_1 = 2.67$ and $d_2 = 1.33$

$$l_2 = \frac{d_1}{d} \times 21.7 = \frac{2.67}{4} \times 21.7 = 14.5''$$

$$l_1 = \frac{d_2}{d} \times 21.7 = \frac{1.33}{4} \times 21.7 = 7.2''$$

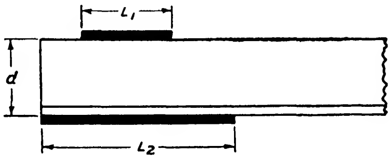
Over-all length of weld $l_2 = 14.5 + 0.25 = 14.75''$

Over-all length of weld $l_1 = \ \ 7.2 + 0.25 = \ \ 7.45''$

Ordinarily, these welds would be made 15″ and 8″ long, respectively.

Due to this angle being connected to the plate by one leg, there will be a bending stress in the welds; however, this is usually ignored if the outstanding leg is not wider than the other leg.

The following table gives the lengths of welds for single-angle connections using the stresses allowed for filler metal of Grade 20, 30, or 40. If filler metal of Grade 2, 4, 10, or 15 is used, multiply the lengths given in the table by $\frac{5}{6}$.



Length of Weld in Inches

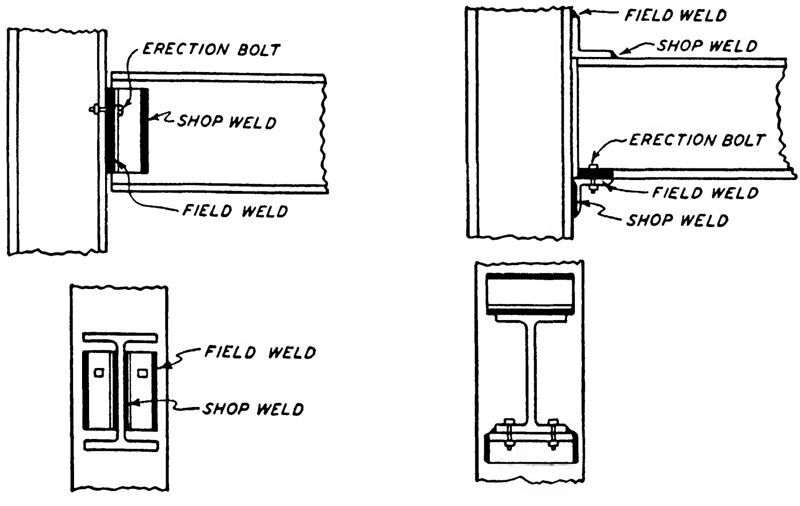
For Standard Single-Angle Connections

Size of Angle	d	Weld	UNIT STRESS 20,000 lb □" L_1	20,000 lb □" L_2	18,000 lb □" L_1	18,000 lb □" L_2
3 x 3 x ¼	3	¼	4.0	10.4	3.6	9.4
5/16	3	5/16	4.1	10.1	3.7	9.1
⅜	3	⅜	4.2	9.9	3.8	8.9
3½ x 3½ x 5/16	3½	5/16	4.7	12.0	4.2	10.8
⅜	3½	⅜	4.8	11.7	4.3	10.6
7/16	3½	⅜	5.7	13.5	5.1	12.1
4 x 4 x ⅜	4	⅜	5.5	13.6	4.9	12.3
7/16	4	⅜	6.4	15.7	5.8	14.1
½	4	⅜	7.4	17.6	6.7	15.9
6 x 6 x ⅜	6	⅜	8.0	21.1	7.1	19.1
½	6	⅜	10.8	27.6	9.7	24.8
⅝	6	⅜	13.7	33.8	12.3	30.4
⅝	6	½	10.2	25.4	9.2	22.8
3 x 2½ x ¼	3	¼	4.0	9.1	3.6	8.2
5/16	3	5/16	4.2	9.3	3.6	8.1
⅜	3	⅜	4.1	8.7	3.7	7.8
4 x 3 x ⅜	4	⅜	5.3	11.2	4.8	10.1
7/16	4	⅜	6.2	12.9	5.6	11.6
½	4	⅜	7.2	14.5	6.5	13.0
5 x 3½ x ⅜	5	⅜	6.5	13.8	5.9	12.4
7/16	5	⅜	7.7	15.8	6.9	14.3
½	5	⅜	8.9	17.8	8.0	16.0
6 x 4 x ⅜	6	⅜	7.8	16.3	7.0	14.7
½	6	⅜	10.5	21.2	9.5	19.0
⅝	6	⅜	13.3	25.8	11.9	23.3
⅝	6	½	9.9	19.4	8.9	17.4

Fig. 70. Joining Rafters and Columns by Welding in Construction of Addition to the Lincoln Electric Company Plant, Cleveland, Ohio

Fig. 70 shows an operator welding a roof rafter to a column section by means of an electric arc.

Beam Connections. One of the advantages of welded construction is that continuity of members is possible. However, where continuous construction is used, the connected members should be designed for the bending moments that apply. If it is not desired to have fixed or continuous beam action, a type of connection shown in Fig. 71 should be used.

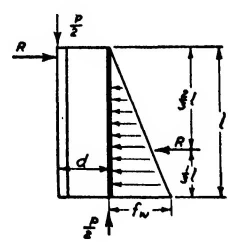


(a) (b)

Fig. 71. Welded Beam Connections

The angle connection shown in Fig. 71 at *a* is more flexible than a similar riveted one, due to being connected on the outside edges of the legs instead of along the gage lines.

Illustrative Example. Assume a beam 16 inches deep having an end reaction of 38,000# is to be connected by two $3'' \times 3'' \times \frac{3}{8}''$ Ls 12 inches long.



The welds will have to resist both shear and bending. The forces acting on the angles and weld are as shown.

P = Beam Reaction

Horizontal stress on the weld is assumed to have a triangular variation

$$R \times \frac{2}{3} l = \frac{P}{2} \times d$$

$$R = \frac{3Pd}{4l} = \frac{3 \times 38{,}000 \times 3}{4 \times 12} = 7125\#$$

f_w = max. horizontal stress in weld.

$$\frac{f_w \times l}{2} = R = 7125\#$$

$$f_w \times 6 = 7125\#$$

$$f_w = \frac{7125}{6} = 1188\# \text{ per inch}$$

The unit vertical stress in the weld due to the load $\frac{P}{2}$ is $\frac{P}{2l}$

$$\frac{P}{2l} = \frac{38{,}000}{24} = 1583\# \text{ per inch}$$

The maximum stress on the weld will be the resultant (f_r) of the vertical and horizontal stresses.

$$f_r = \sqrt{1188^2 + 1583^2}$$

$$f_r = 1980\# \text{ per inch}$$

Therefore $\frac{1}{4}$-inch fillet weld should be used.

Tests made at Lehigh University show that a connection as shown in Fig. 71 at *b*, using 3×3-inch top angles of various thicknesses will produce satisfactory beam connections providing for fifty per cent restraint. In computing the required welding for this connection, the end moment should be divided by the depth of the beam plus the length of angle leg, to determine the top tension and the bottom thrust.

After the top tension force is known, it should be divided by the length of the top angle; the result will be the unit stress the weld will have to resist. Likewise, the bottom thrust is divided by the length of weld that can be placed on the bottom seat angle against the bottom flange.

The vertical reaction of the beam is taken by the weld along the sides of the vertical leg of the seat angle.

Fig. 72 shows an operator field welding a beam connection to a column.

Fig. 73 shows various types of fixed or continuous beam connections.

In designing fixed or rigid end beam connections, as shown in Fig. 73, the end moment is divided by the beam depth to determine the top tension and bottom thrust. The reaction is figured as taken by the fillet welds on the sides of the vertical leg of the shop-welded seat.

Illustrative Example. A connection as shown in Fig. 73 at *a* has an end moment of 320,000 inch pounds. The beams connecting to the girder are 16 inches in depth and have a total uniform load of 42,000# each.

The top tension and bottom thrust equals

$$P=\frac{320,000}{16}=20,000\#$$

Fig. 72. Arc Welding Floor Beam to Column
Courtesy of The Lincoln Electric Company, Cleveland, Ohio

If $\frac{1}{4}$-inch fillet weld is used, length of weld required equals

$$l=\frac{20,000}{2000}=10''$$

or 5″ on each side of top plate. The weld along the bottom flange connecting to the seat angle cannot be made this long, therefore $\frac{3}{8}$-inch weld should be used.

$$l_2=\frac{20,000}{3000}=6.67''$$

or 3.34″ on each side of bottom flange. The thrust at the bottom flange can also be resisted by butt welding the bottom flange to the girder web. It is not necessary to bevel the end of the flanges, as all strength requirements are satisfied by just filling the gap with weld metal.

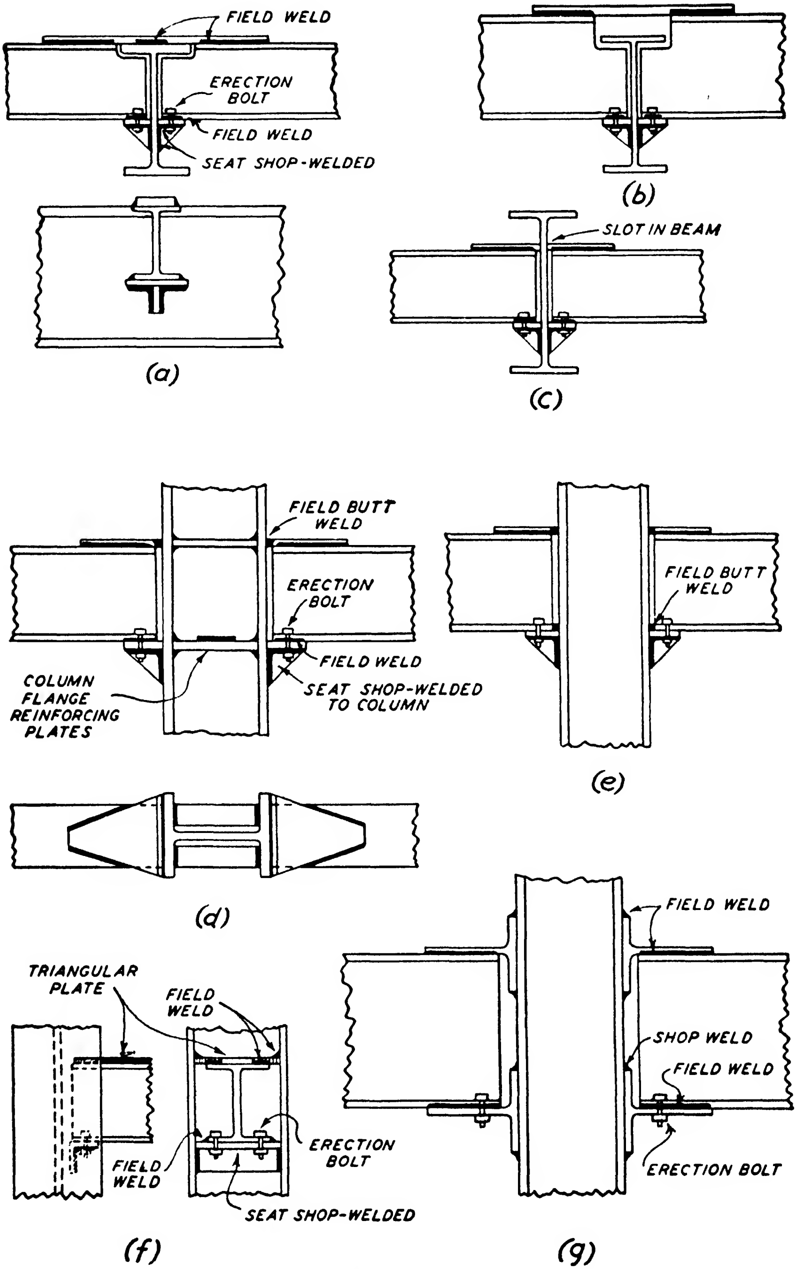


Fig. 73. Types of Fixed or Continuous Beam Connections

The length and size of weld on the sides of the vertical leg of the seat angle equals

$$l_3 = \frac{21{,}000}{3000} = 7''$$

of $\frac{3}{8}$-inch fillet weld, or $3\frac{1}{2}''$ on each side.

Fig. 64 shows various types of welded beam connections using beam seats.

Column Splices. Generally, column splice plates serve only to hold the sections rigidly in line. Even when there is bending, it rarely

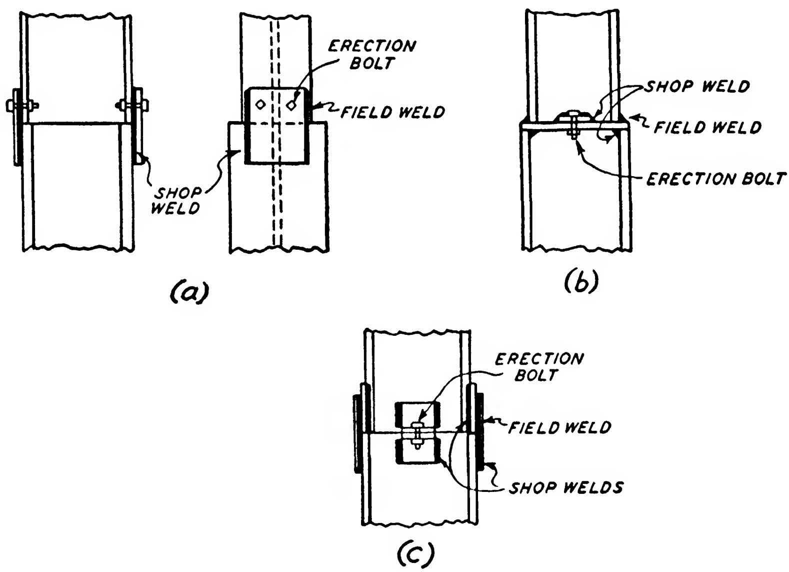


Fig. 74. Column Splices

happens that there is not compression over the whole cross section. Various types of column splices are shown in Fig. 74.

Column Bases. If there is no bending at the base of a column and therefore no uplift on the anchor bolts, the connections need be only sufficient to provide a means of bolting the column to the base. In Fig. 75 are shown various column bases. Types (*d*) and (*e*) are for uplift.

Fig. 76 shows an operator field welding on a school building. In the foreground is shown a portable electric welding unit.

PRACTICE PROBLEMS

1. Write the definitions of Fusion Welding, Root, Throat, Throat Dimension, Fillet Weld, Butt Weld, Weld Length, Weld Dimensions, and Gas Cutting.

2. Compute and tabulate the design tension strength of the following welds per linear inch, $\frac{1}{4}''$, $\frac{5}{16}''$, $\frac{3}{8}''$, $\frac{1}{2}''$, $\frac{5}{8}''$, and $\frac{3}{4}''$.

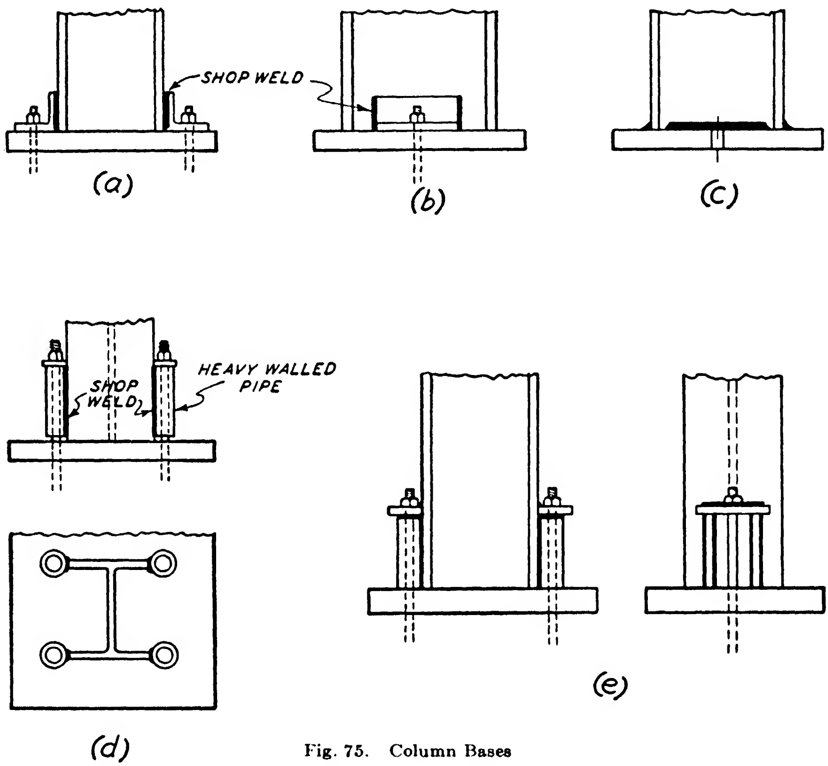


Fig. 75. Column Bases

3. Show the symbol for a fillet weld on the near side and on both sides.

4. Compute the required fillet welding and size and length of splice plates necessary to splice a bar $6'' \times 1''$ with a tension stress of 120,000#.

5. Compute the necessary fillet welding required on a $3\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$ angle to connect it to a plate. The angle is a hanger and has a tension stress of 60,000#. The welding should be done on the $3\frac{1}{2}''$ leg.

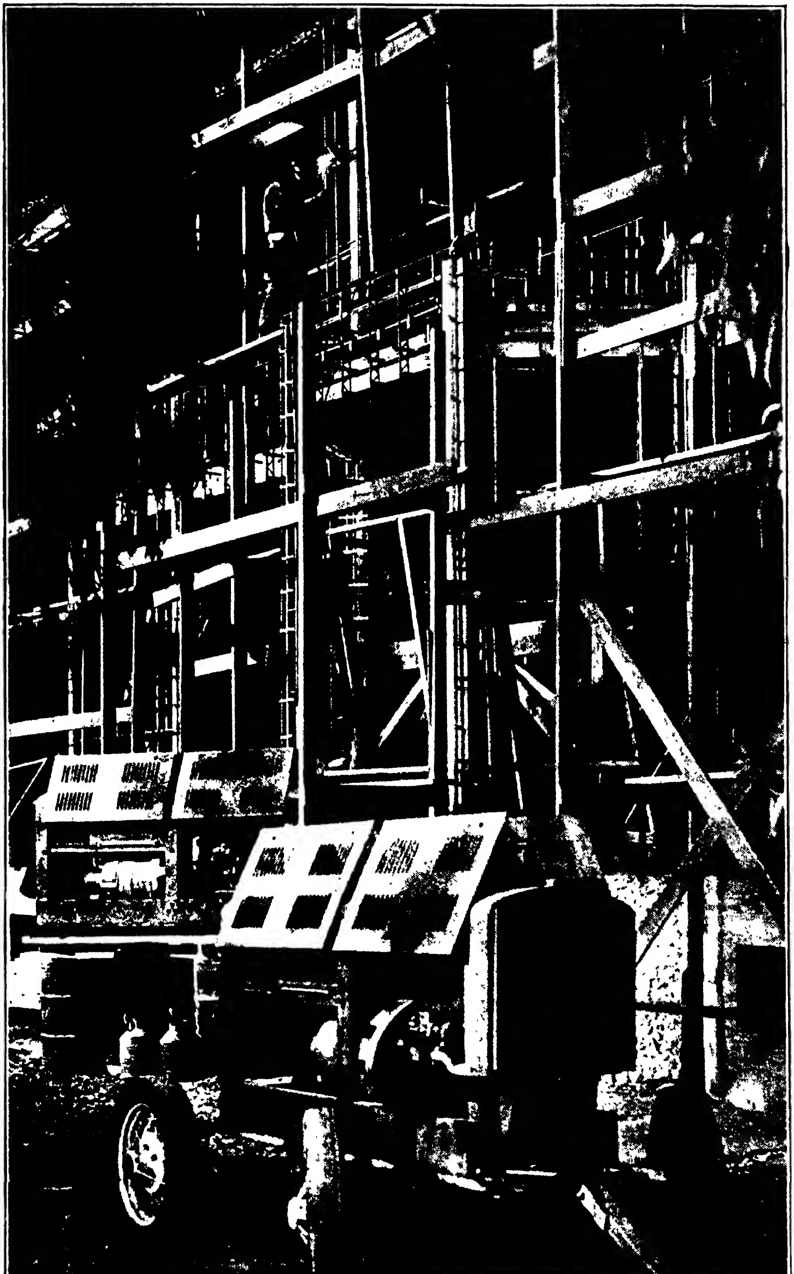
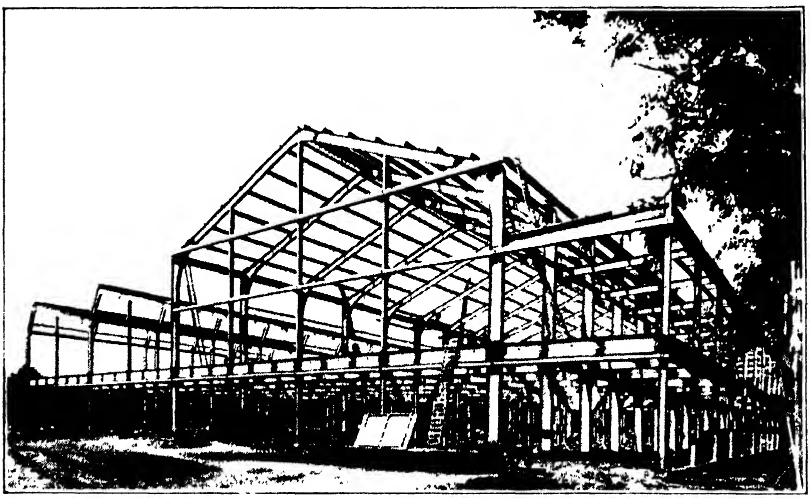


Fig. 76. Arc Welding the Steel Frame of a School Building
Courtesy of The Lincoln Electric Company, Cleveland, Ohio

6. If an 18″ beam with an end connection as shown in Fig. 71 at *a* had an end reaction of 48,000# what length of $\frac{1}{4}$-inch fillet weld would be required on the connection angles?

7. Design the connection shown in Fig. 73 at *c*. Beam depth = 24″; end reaction, 32,000#; end moment, 480,000 inch-pounds.



WELDED BUILDING, ADDITION TO PLANT OF LINCOLN ELECTRIC COMPANY, CLEVELAND, OHIO

CHAPTER VII

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 or between columns.

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. 77 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 riveted steel construction are somewhat restrained by their end connections, they are treated as simple beams in designing. 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 one or both supports.

Sections. The structural steel sections most used as beams are the I beam and wide-flange beam. They are designed for this purpose and are 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

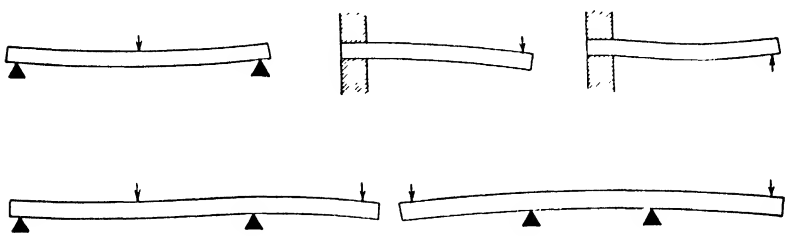


Fig. 77. Simple, Cantilever, and Restrained Beams

beam and are used for 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. 78, shows that the deflections in the joists were so great as to cause a bad crack in the marble floor above the steel girder.

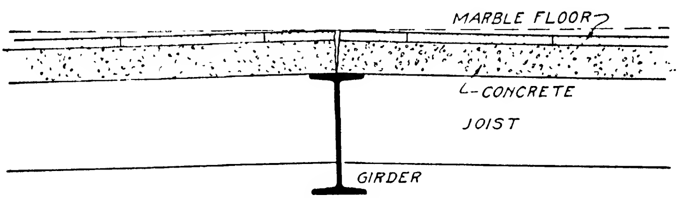


Fig. 78. Floor Section Showing Crack Over Girder, Due to Deflection of Joists

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. 79. The stress on the extreme fiber—not the average stress—governs in designing. The working or unit stress allowed is 20,000 pounds per square inch in both the tension and the compression flanges.

*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.

In Fig. 79 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. The resisting moment is expressed by the formula

$$M = \frac{SI}{c}$$

in which M is resisting moment in inch-pounds; I is the moment of inertia in terms of inches^4; 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.

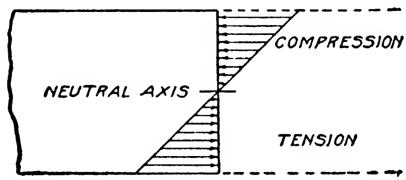


Fig. 79. Graphical Representation of Stresses in the Fibers of a Beam

The steel handbooks tabulate the quantity $\frac{I}{c}$, known as the section modulus. By examining the formula it is readily seen that the resisting moment M is obtained by multiplying the section modulus $\frac{I}{c}$ by the unit stress S.

Assume a 12″ WF 28#. From the handbook the value of I is 213.5. The distance from the neutral axis to the extreme fiber is 6 inches. The allowable unit stress on the extreme fiber is 20,000 pounds per square inch. Then

$$M = \frac{SI}{c} = \frac{20{,}000 \times 213.5}{6} = 711{,}667 \text{ inch-lbs.}$$

In the steel handbooks the section modulus is shown for a 12″ WF 28# beam as 35.6 inches^3; therefore, its resisting moment can be found by multiplying by 20,000. We obtain again

$$M = 20{,}000 \times 35.6 = 712{,}000 \text{ inch-lbs.}$$

In comparing the moment of resistance with the bending moment, the designer must always be careful that the same units are used—either inch-pounds or foot-pounds; not both. A girder designed for a resisting moment in inch-pounds to withstand the numerically equal bending moment in foot-pounds would surely fail. On the other hand (though this mistake is rarely made), a girder designed for a resisting moment in foot-pounds to withstand a bending moment in inch-pounds numerically the same would be about ten times too heavy.

It must not be forgotten that the moment of inertia, the section modulus, and the resisting moment are all figured for a definite axis and that these quantities only then indicate the resisting qualities of the section when this section is used with the reference axis horizontal. An **I** beam with its web horizontal has between one-eighth and one-tenth the resisting moment of the same beam with its web vertical. In the same manner a wooden joist laid flat can only carry from one-ninth to one-twenty-fifth of the load it can safely support when held upright. It is particularly important when designing angles that the tables are correctly read. For example, a $4'' \times 3'' \times \frac{1}{2}''$ L has a resisting moment of 3170 foot-pounds when used with the long leg vertical and a resisting moment of only 1830 foot-pounds when used with the short leg vertical. In general, *a section has greatest resistance in bending when used with its greatest dimension in a vertical position or parallel to the forces producing bending stresses.*

Problems. 1. Pick from the steel handbook the lightest **I** beam and WF beam that will safely resist at 20,000 pounds per square inch a bending moment of 380,000 ft. lbs.

2. Find the lightest **I** beam and WF beam that will safely carry the bending moment in Problem 1 but with an allowable unit fiber stress of 18,000 pounds per square inch.

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

$$S = \frac{Mc}{I}$$

Assume that the bending moment is 500,000 foot-pounds and that the beam is 30″ WF 116#, then,

$$S = \frac{500{,}000 \times 12 \times 15}{4919.1} = 18{,}300\# \text{ per sq. in.}$$

Vertical Shear. Fig. 80 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 50,000 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

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

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

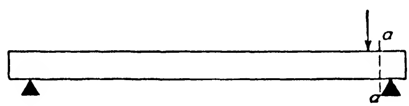


Fig. 80. Diagram Illustrating Shear on a Beam

is difficult to compute, but it can be determined approximately by assuming that the entire shear is resisted by the web of the beam, 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 31.8#. The area of the web is approximately $12'' \times .35'' = 4.2$ sq. in. Then $S_s = \frac{50{,}000}{4.2}$ or 11,900 pounds per square inch. The allowable value in shear is 13,000 pounds per square inch, and therefore the beam is safe 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

$$V = 13{,}000 \times 4.2 = 54{,}600\#$$

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.

To figure the unit shear on the web, first compute the gross area of the web, and then divide the maximum shear by this area.

Problem. Find the unit stress in shear on the I beams, channels, and angles picked out in the previous problems for a maximum shear of 100,000#. Use your handbook to find the dimensions of the web.

Deflection. 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.

(1) For a uniformly distributed load $d=\frac{5}{384}\frac{Wl^3}{EI}$

(2) For a load concentrated at center of span $d=\frac{1}{48}\frac{Wl^3}{EI}$

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, the modulus of elasticity, equals 30,000,000 lbs. per sq. inch.

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 units 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.

The modulus of elasticity may be considered as the unit stress that would stretch or compress a member for its full length, i.e., that would double the length of a tension member or reduce a compression member to 0 length, if the member were to retain perfect elasticity and strength without limit.

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.

Joists. The loads on joists are usually distributed uniformly along the length of the beam. Assume that the simple beam, Fig. 81, 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:

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

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

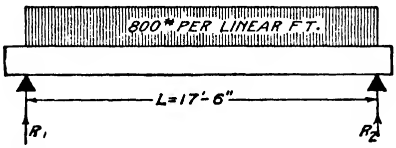


Fig. 81. Diagram of Beam Uniformly Loaded

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

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

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

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

The formula for this bending moment is

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

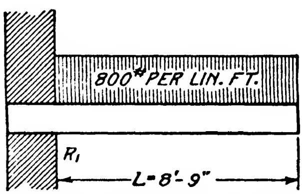


Fig. 82. Diagram of Cantilever Uniformly Loaded

Cantilever Beam. Fig. 82 represents a cantilever beam supporting a uniformly distributed load. Assume the length L of cantilever to be 8′-9″, and the load, 800 pounds per lineal foot; then

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

$$R_1 = \qquad 7000\#$$

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. 83. 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

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

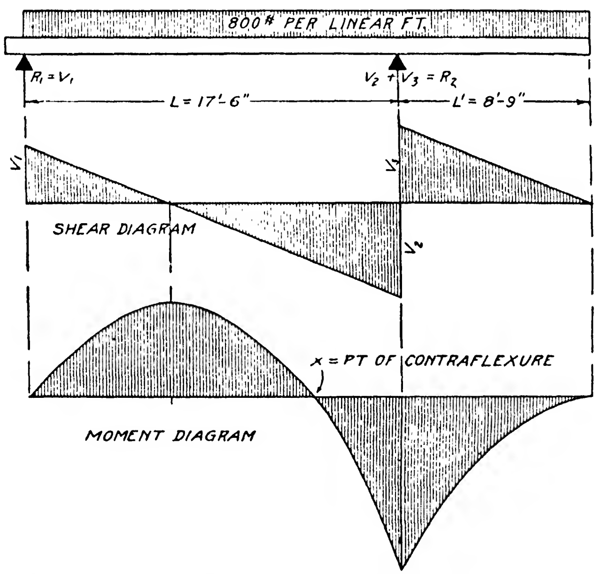


Fig. 83. An Overhanging Beam with Shear and Moment Diagrams

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

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

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

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

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 contra-flexure" 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.

PRACTICE PROBLEMS

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

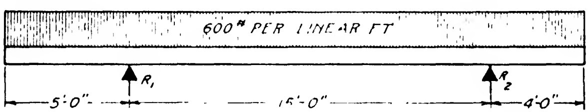


Fig. 84. Uniformly Loaded Beam Overhanging at Both Ends.

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. 84 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. 85 represents a simple beam supporting the concentrated loads P_1, P_2, P_3, and P_4. The loads are

$$\begin{aligned} P_1 &= 60{,}000\# \\ P_2 &= 80{,}000\# \\ P_3 &= 80{,}000\# \\ P_4 &= \underline{50{,}000\#} \\ \text{Total load} &= 270{,}000\# \end{aligned}$$

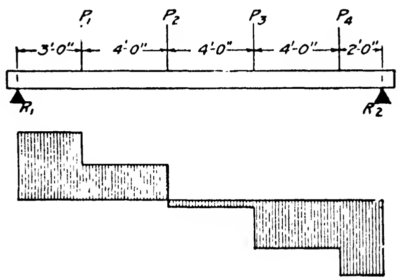


Fig. 85. Simple Beam with Concentrated Loads Shear Diagram

To determine the reaction R_2, take moments about R_1:

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

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

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

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

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

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

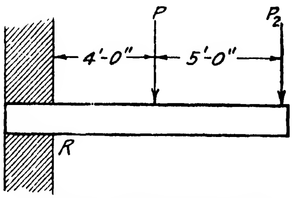


Fig. 86. Cantilever Beam with Concentrated Loads

$$\begin{array}{rl} +7\times 130{,}588 = & +914{,}116 \\ -4\times 60{,}000 = & -240{,}000 \\ \hline & +674{,}116 \text{ ft.-lb.} \end{array}$$

Computing on the right gives

$$\begin{array}{lll} +10\times 139{,}412 = & & +1{,}394{,}120 \\ -\ 4\times 80{,}000 = 320{,}000 & & \\ -\ 8\times 50{,}000 = 400{,}000 & & -\ \ 720{,}000 \\ \hline & & +\ \ 674{,}120 \text{ ft.-lb.} \end{array}$$

Cantilever Beam. Fig. 86 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

$$\begin{array}{rl} -4\times 30{,}000 = & -120{,}000 \\ -9\times 40{,}000 = & -360{,}000 \\ \hline & -480{,}000 \text{ ft.-lb.} \end{array}$$

Simple Beams on Two Supports and Projecting at Both Ends. Fig. 87 represents a beam resting on two supports and projecting beyond both of them. It supports concentrated loads as shown. The loads are

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

To determine the reaction R_2, take moments about R_1:

$$
\begin{array}{rlrr}
 & 0\times 15{,}000\ (P_2) & = 00{,}000 & \\
 & 4\times 15{,}000\ (P_3) & = 60{,}000 & \\
 & 8\times 15{,}000\ (P_4) & =120{,}000 & \\
 & 12\times 15{,}000\ (P_5) & =180{,}000 & \\
 & 16\times 30{,}000\ (P_6) & =\underline{480{,}000} & 840{,}000 \text{ ft.-lb.} \\
- & 4\times 15{,}000\ (P_1) & = & -\ \underline{60{,}000} \\
\multicolumn{3}{l}{\text{Moment of reaction } R_2=} & 780{,}000 \text{ ft.-lb.}
\end{array}
$$

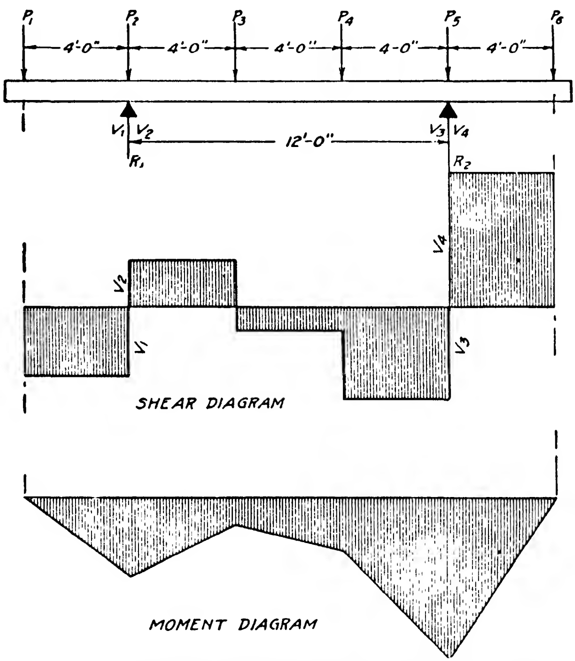


Fig. 87. Overhanging Beam with Concentrated Loads. Shear and Moment Diagrams

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

To determine the reaction R_1, take moments about R_2:

$$
\begin{array}{rlrr}
 & 0\times 15{,}000\ (P_5) & = 00{,}000 & \\
 & 4\times 15{,}000\ (P_4) & = 60{,}000 & \\
 & 8\times 15{,}000\ (P_3) & =120{,}000 & \\
 & 12\times 15{,}000\ (P_2) & =180{,}000 & \\
 & 16\times 15{,}000\ (P_1) & =\underline{240{,}000} & 600{,}000 \text{ ft.-lb.} \\
- & 4\times 30{,}000\ (P_6) & = & -\underline{120{,}000} \\
\multicolumn{3}{l}{\text{Moment of reaction} R_1=} & 480{,}000 \text{ ft.-lb.}
\end{array}
$$

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

$$\text{at } R_1 \quad M = \quad -4 \times 15{,}000 = \qquad -\ 60{,}000 \text{ ft.-lb.}$$
$$\text{at } R_2 \quad M = \quad -4 \times 30{,}000 = \qquad -120{,}000 \text{ ft.-lb.}$$

$$\text{at } P_3 \quad M = \begin{cases} +4 \times 40{,}000 = +160{,}000 \\ -4 \times 15{,}000 = -\ \ 60{,}000 \\ -8 \times 15{,}000 = \underline{-120{,}000} - 20{,}000 \text{ ft.-lb.} \end{cases}$$

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.

PRACTICE PROBLEMS

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?

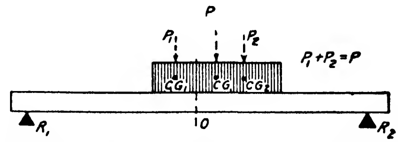


Fig. 88. Simple Beam with Uniformly Distributed Load over Part of Span

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. 88, 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. 89.

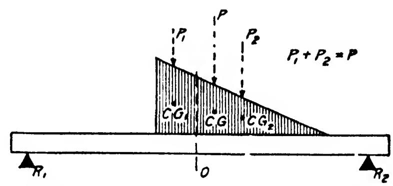


Fig. 89. Simple Beam with Variable Load

Full Length Distributed Load and Concentrated Load. Fig. 90 illustrates a beam with a uniformly distributed load full length and a concentrated load, as shown.

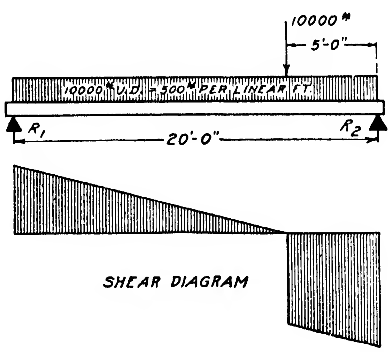


Fig. 90. Simple Beam with Uniformly Distributed Load over Entire Length and One Concentrated Load

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

Moments about R_1 are

$$10\times10{,}000=100{,}000$$
$$15\times10{,}000=150{,}000$$
$$\overline{250{,}000 \text{ ft.-lb.}}$$

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

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

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

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

The bending moment computed on the left is

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

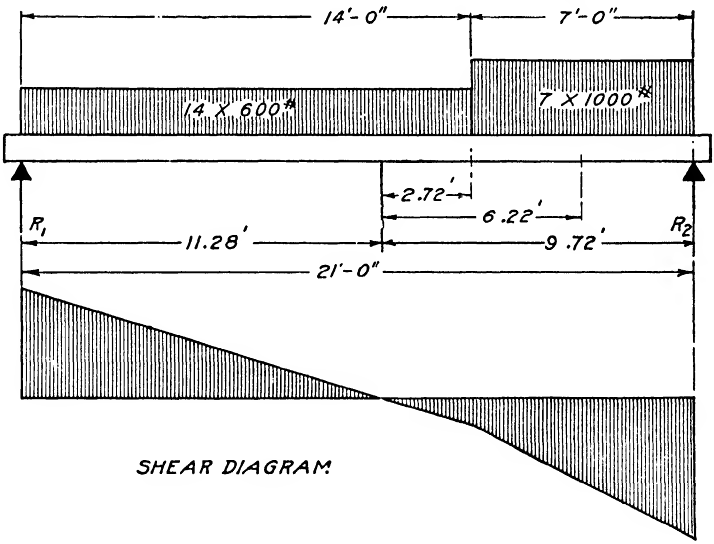


Fig. 91. Simple Beam with Two Rates of Uniformly Distributed Load

Two Uniformly Distributed Loads Not Overlapping. Fig. 91 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

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

Moments about R_1 are

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

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

Moments about R_2 are

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

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

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

The bending moment computed on the left is

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

Two Distributed Loads and Concentrated Loads. Fig. 92 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

$$\begin{array}{lr} \text{u.d.} \quad 12 \times 1{,}000 = & 12{,}000 \\ \text{u.d.} \quad 4 \times 500 = & 2{,}000 \\ \text{concentrated} \quad = & \underline{10{,}000} \\ \text{Total load} \quad = & 24{,}000\# \end{array}$$

Moments about R_1 are

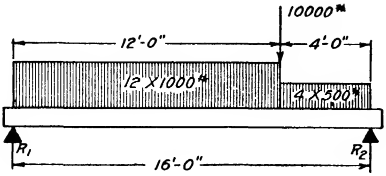


Fig. 92. Simple Beam with Mixed Loads

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

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

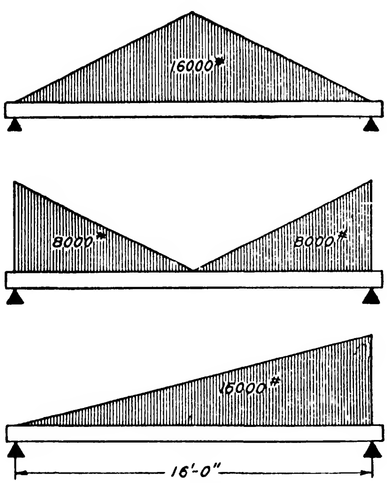


Fig. 93. Simple Beams with Variable Loads

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.

PRACTICE PROBLEMS

1. Compute the bending moments for the loads illustrated in Fig. 93. 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″×42.9#×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

W_1 = variable load 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

A = variable fraction

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

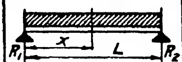
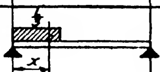
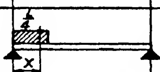
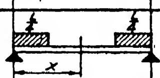
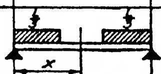
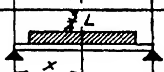
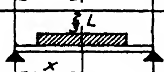
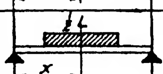
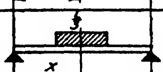
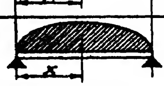
	FORM OF LOAD	R_1	R_2	MAXIMUM BENDING MOMENT	x	REMARKS
		Lbs.	Lbs.	Foot-Lbs.		
1		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{1}{8}WL$	$\frac{1}{2}L$	
2		$\frac{3}{4}W$	$\frac{1}{4}W$	$\frac{9}{64}WL$	$\frac{3}{8}L$	
3		$\frac{5}{6}W$	$\frac{1}{6}W$	$\frac{25}{216}WL$	$\frac{5}{18}L$	
4		$\frac{7}{8}W$	$\frac{1}{8}W$	$\frac{49}{512}WL$	$\frac{7}{32}L$	
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		$\frac{1}{3}W$	$\frac{2}{3}W$	$0.128WL$	$0.57L$	
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 II—(Continued)

Reactions and Bending Moments for Typical Loadings

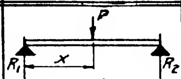
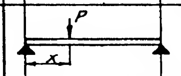
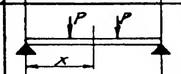
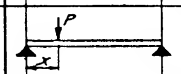
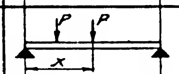
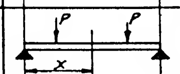
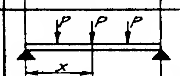
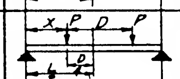
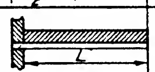
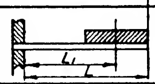
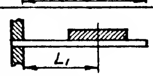
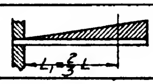
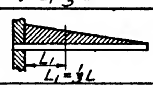
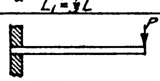
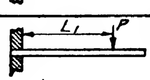
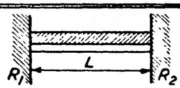
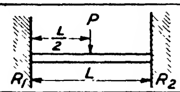
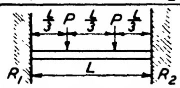
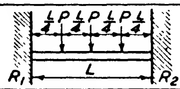
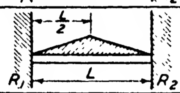
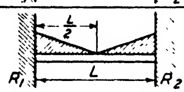
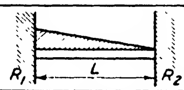
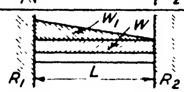
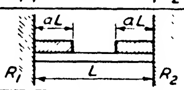
	FORM OF LOAD	R_1 Lbs.	R_2 Lbs.	MAXIMUM BENDING MOMENT Foot-Lbs.	x	REMARKS
16		$\frac{1}{2}P$	$\frac{1}{2}P$	$\frac{1}{4}PL$	$\frac{1}{2}L$	POSITION OF ONE CONCENTRATED LOAD FOR MAXIMUM BENDING MOMENT
17		$\frac{2}{3}P$	$\frac{1}{3}P$	$\frac{2}{9}PL$	$\frac{1}{3}L$	
18		P	P	$\frac{1}{3}PL$	$\frac{1}{3}L$ 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		$P\left(1-\frac{1}{2}\frac{D}{L}\right)$	$P\left(1+\frac{1}{2}\frac{D}{L}\right)$	$\frac{P}{2}\left(L-D+\frac{D^2}{4L}\right)$	$\frac{L}{2}-\frac{D}{4}$	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	

TABLE II—(Continued)

Reactions and Bending Moments for Typical Loadings

	FORM OF LOAD	R_1 LBS.	R_2 LBS.	MAXIMUM BENDING MOMENT FOOT-LBS	LOCATION OF MAXIMUM BENDING MOMENT
31		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{1}{12}WL$	AT SUPPORTS
32		$\frac{1}{2}P$	$\frac{1}{2}P$	$\frac{1}{8}PL$	AT CENTER AND AT SUPPORTS
33		P	P	$\frac{2}{9}PL$	AT SUPPORTS
34		$\frac{3}{2}P$	$\frac{3}{2}P$	$\frac{15}{48}PL$	AT SUPPORTS
35		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{5}{48}WL$	AT SUPPORTS
36		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{1}{16}WL$	AT SUPPORTS
37		$\frac{2}{3}W$	$\frac{1}{3}W$	$\frac{1}{10}WL$	AT SUPPORT R_1
38		$\frac{2}{3}W_1+\frac{1}{2}W$	$\frac{1}{3}W_1+\frac{1}{2}W$	$\frac{1}{10}W_1L+\frac{1}{12}WL$	AT SUPPORT R_1
39		$\frac{1}{2}W$	$\frac{1}{2}W$	$\frac{a(3-2a)}{12}WL$	AT SUPPORTS

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.

$$W = 9 \times 320 \quad = 2880\#$$
$$V_1 = R_1 = \tfrac{3}{4}W = \tfrac{3}{4} \times 2880 = 2160\ \#$$

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

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.

PRACTICE PROBLEMS

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

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

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." Its values for WF beams, 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 20,000 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

$$\frac{I}{c}=\frac{M}{S}=\frac{1{,}200{,}000}{20{,}000}=60$$

Referring to the tables for beams it is found that the sections having the nearest higher value of the section modulus are
16″ WF 40#, 15″ I 45#, 14″ WF 42#, 18″ ⊏ 42.7, 12″ WF 50#, 10″ WF 54#, or 8″ WF 67#

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 20,000 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.

Tables are given, however, listing the section modulus in numerical order for the various sections. Table III is a page from the A.I.S.C. handbook showing the Section Modulus Table. The section modulus of a section can also be selected from the table of Properties for Designing. Table IV is a page of these tables.

If the load on a beam is uniformly distributed, the computations may be still further shortened by means of tables given in the handbooks. These tables give the allowable uniform load for various lengths of spans. For example, a beam 20 feet long supports a load of 2700# per linear foot. The total load is $20\times2700=54{,}000$#. From the tables, the most economical beam is an 18″ WF 47#.

TABLE III

SECTION MODULUS TABLE
FOR SECTIONS USED AS BEAMS

Section Modulus Inches³	1	2	3	4	5	6	7	8	9	10	11
61.0	16WF40	18 ⊔ 42.7	14WF43	12WF50	15 I 50	10WF60					
60.7	16WF40	14WF42	18 ⊔ 42.7	12WF50	15 I 50	10WF60					
60.4	16WF40	14WF42	18 ⊔ 42.7	15 I 45	12WF50	10WF54	8WF67				
58.9	16WF40	14WF42	18 ⊔ 42.7	15 I 42.9	12WF50	10WF54	8WF67				
58.2	16WF40	14WF42	18 ⊔ 42.7	15 I 42.9	12WF45	10WF54	8WF67				
57.2	16WF40	14WF42	18 ⊔ 42.7	15 I 42.9	12WF45	10WF54	15 ⊔ 55	8WF67			
56.3	16WF36	14WF42	18 ⊔ 42.7	15 I 42.9	12WF45	10WF54	15 ⊔ 55	8WF67			
54.6	16WF36	14WF38	18 ⊔ 42.7	15 I 42.9	12WF45	10WF49	15 ⊔ 55	8WF67			
53.6	16WF36	14WF38	18 ⊔ 42.7	15 I 42.9	12WF45	10WF49	15 ⊔ 50	8WF67			
53.2	16WF36	14WF38	18 ⊔ 42.7	15 I 42.9	12WF45	10WF49	15 ⊔ 50	12 I 55	8WF58		
51.9	16WF36	14WF38	12WF40	18 ⊔ 42.7	15 I 42.9	10WF49	15 ⊔ 50	12 I 55	8WF58		
50.9	16WF36	15 M 36	14WF38	12WF40	18 ⊔ 42.7	15 I 42.9	10WF49	15 ⊔ 50	12 I 55	8WF67	
50.3	16WF36	15 M 36	14WF38	12WF40	18 ⊔ 42.7	15 I 42.9	10WF49	12 I 50	15 ⊔ 50	8WF58	
49.8	16WF36	15 M 36	14WF38	12WF40	18 ⊔ 42.7	15 I 42.9	15 ⊔ 45	10WF49	12 I 50	8WF58	
49.1	16WF36	15 M 36	14WF38	12WF40	18 ⊔ 42.7	15 I 42.9	15 ⊔ 45	10WF45	12 I 50	8WF58	
48.7	15 M 33	16WF36	14WF38	12WF40	18 ⊔ 42.7	15 I 42.9	15 ⊔ 45	10WF45	12 I 50	8WF58	
48.5	15 M 33	14WF34	12WF40	18 ⊔ 42.7	15 I 42.9	15 ⊔ 45	10WF45	12 I 50	8WF58		
47.3	15 M 33	14WF34	12WF40	18 ⊔ 42.7	15 I 42.9	15 ⊔ 45	12 I 45	10WF45	8WF58		
46.2	15 M 33	14WF34	12WF40	15 ⊔ 40	15 I 42.9	12 I 45	10WF45	8WF58			
45.9	15 M 33	14WF34	12WF36	15 ⊔ 40	15 I 42.9	12 I 45	10WF45	8WF58			
45.2	15 M 33	14WF34	14 M 34	12WF36	15 ⊔ 40	15 I 42.9	12 I 45	10WF45	8WF58		
44.8	15 M 33	14WF34	14 M 34	12WF36	15 ⊔ 40	12 I 40.8	10WF45	8WF58			
44.5	15 M 33	14WF34	14 M 34	12WF36	15 ⊔ 40	12 I 40.8	10WF41	8WF58			
43.2	15 M 33	14WF34	14 M 34	12WF36	15 ⊔ 40	12 I 40.8	10WF41	8WF48			
42.5	14 M 30	14WF34	15 ⊔ 35	12WF36	12 I 40.8	10WF41	8WF48				
41.8	14WF30	14 M 30	15 ⊔ 33.9	12WF36	12 I 40.8	10WF41	8WF48				
40.7	14WF30	14 M 30	12WF32	15 ⊔ 33.9	12 I 40.8	10WF41	8WF48				
39.9	14WF30	14 M 30	12WF32	15 ⊔ 33.9	10WF37	12 I 40.8	8WF48				
37.8	14WF30	14 M 30	12WF32	15 ⊔ 33.9	12 I 35	10WF37	8WF48				
36.0	14WF30	14 M 30	12 I 31.8	12WF32	15 ⊔ 35.9	10WF37	8WF48				
35.6	12WF28	14 M 30	12 I 31.8	15 ⊔ 33.9	10WF37	8WF40					
35.0	12WF28	14 M 30	12 I 31.8	10WF33	15 ⊔ 33.9	8WF40					

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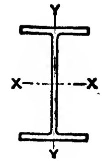
Table V shows a part of the Allowable Uniform Loads Table found in the handbooks. The loads given in Table V are for laterally supported beams. The specification requires that the ratio of the un-

TABLE IV

WF SECTIONS

PROPERTIES FOR DESIGNING



Nominal Size	Weight per Foot	Area of Section	Depth of Section	Flange Width	Flange Thickness	Web Thickness	Axis X-X I	Axis X-X S	Axis X-X r	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r
In.	Lb.	In.²	In.	In.	In.	In.	In.⁴	In.³	In.	In.⁴	In.³	In.
27 x 14	177	52.10	27.31	14.090	1.190	.725	6728.6	492.8	11.36	518.9	73.7	3.16
	163	47.93	27.12	14.035	1.095	.670	6141.5	452.9	11.32	468.7	66.8	3.13
	154	45.30	27.00	14.000	1.035	.635	5775.8	427.8	11.29	437.6	62.5	3.11
	145	42.68	26.88	13.965	.975	.600	5414.3	402.9	11.26	406.9	58.3	3.09
27 x 10	114	33.53	27.28	10.070	.932	.570	4080.5	299.2	11.03	149.6	29.7	2.11
	106	31.17	27.14	10.035	.862	.535	3761.2	277.2	10.98	136.1	27.1	2.09
	98	28.82	27.00	10.000	.792	.500	3446.5	255.3	10.94	122.9	24.6	2.07
	91	26.77	26.84	9.983	.712	.483	3129.2	233.2	10.81	109.0	21.8	2.02
24 x 14	160	47.04	24.72	14.091	1.135	.656	5110.3	413.5	10.42	492.6	69.9	3.23
	150	44.10	24.56	14.063	1.055	.628	4733.5	385.5	10.36	452.5	64.3	3.20
	140	41.16	24.41	14.029	.980	.594	4376.1	358.6	10.31	414.5	59.1	3.17
	130	38.21	24.25	14.000	.900	.565	4009.5	330.7	10.24	375.2	53.6	3.13
24 x 12	120	35.29	24.31	12.088	.930	.556	3635.3	299.1	10.15	254.0	42.0	2.68
	110	32.36	24.16	12.042	.855	.510	3315.0	274.4	10.12	229.1	38.0	2.66
	100	29.43	24.00	12.000	.775	.468	2987.3	248.9	10.08	203.5	33.9	2.63
24 x 9	94	27.63	24.29	9.061	.872	.516	2683.0	220.9	9.85	102.2	22.6	1.92
	87	25.58	24.16	9.025	.807	.480	2467.8	204.3	9.82	92.9	20.6	1.91
	80	23.54	24.00	9.000	.727	.455	2229.7	185.8	9.73	82.4	18.3	1.87
	74	21.77	23.87	8.975	.662	.430	2033.8	170.4	9.67	73.8	16.5	1.84
21 x 13	142	41.76	21.46	13.132	1.095	.659	3403.1	317.2	9.03	385.9	58.8	3.04
	132	38.81	21.31	13.087	1.020	.614	3141.6	294.8	9.00	353.8	54.1	3.02
	122	35.85	21.16	13.040	.945	.567	2883.2	272.5	8.97	322.1	49.4	3.00
	112	32.93	21.00	13.000	.865	.527	2620.6	249.6	8.92	289.7	44.6	2.96
21 x 9	103	30.27	21.29	9.071	1.010	.608	2268.0	213.1	8.66	119.9	26.4	1.99
	96	28.21	21.14	9.038	.935	.575	2088.9	197.6	8.60	109.3	24.2	1.97
	89	26.15	21.00	9.000	.865	.537	1919.2	182.8	8.57	99.4	22.1	1.95
	82	24.10	20.86	8.962	.795	.499	1752.4	168.0	8.53	89.6	20.0	1.93
21 x 8¼	73	21.46	21.24	8.295	.740	.455	1600.3	150.7	8.64	66.2	16.0	1.76
	68	20.02	21.13	8.270	.685	.430	1478.3	139.9	8.59	60.4	14.6	1.74
	63	18.52	21.00	8.250	.620	.410	1343.6	128.0	8.52	53.8	13.0	1.70
	59	17.36	20.91	8.230	.575	.390	1246.8	119.3	8.47	49.2	12.0	1.68

Courtesy of American Institute of Steel Construction

braced length to width of flange $\frac{l}{b}$ for compression flanges of rolled sections, plate girders, and built-up members subject to bending shall not exceed 40. If the ratio of $\frac{l}{b}$ exceeds 15, the unit stress must be reduced. See Section 12 of the specifications in the Appendix.

TABLE V

BEAMS 18

WF SECTIONS

ALLOWABLE UNIFORM LOADS IN KIPS FOR BEAMS LATERALLY SUPPORTED

For beams laterally unsupported, allowable loads must be reduced.

Span in Feet	Nominal Depth and Width—Weight per Foot: 18 x 11¾				18 x 8¾				18 x 7½			Deflection Inches
	124	114	105	96	85	77	70	64	55	50	47	
											162	
7									184	168	157	.06
8					250	224	206	188	164	148	137	.07
9					231	210	190	173	146	132	122	.09
10	316	286	264	242	208	189	171	156	131	119	110	.12
11	290	267	245	224	189	172	155	142	119	108	100	.14
12	266	245	225	205	173	157	142	130	109	99	91	.17
13	245	226	207	189	160	145	132	120	101	91	84	.19
14	228	210	193	176	149	135	122	111	94	85	78	.23
15	212	196	180	164	139	126	114	104	87	79	73	.26
16	199	183	169	154	130	118	107	98	82	74	69	.29
17	187	173	159	145	122	111	101	92	77	70	65	.33
18	177	163	150	137	116	105	95	87	73	66	61	.37
19	168	155	142	129	110	99	90	82	69	63	58	.42
20	159	147	135	123	104	95	86	78	66	59	55	.46
21	152	140	128	117	99	90	81	74	62	57	52	.51
22	145	133	123	112	95	86	78	71	60	54	50	.56
23	139	128	117	107	91	82	74	68	57	52	48	.61
24	133	122	112	103	87	79	71	65	55	49	46	.66
25	128	117	108	98	83	76	68	62	52	48	44	.72
26	123	113	104	95	80	73	66	60	50	46	42	.78
27	118	109	100	91	77	70	63	58	49	44	41	.84
28	114	105	96	88	74	68	61	56	47	42	39	.90
29	110	101	93	85	72	65	59	54	45	41	38	.97
30	106	98	90	82	69	63	57	52	44	40	37	1.03
31	103	95	87	79	67	61	55	50	42	38	35	1.11
32	100	92	84	77	65	59	53	49	41	37	34	1.18
33	97	89	82	75	63	57	52	47	40	36	33	1.25
34	94	86	79	72	61	56	50	46	39	35	32	1.33
35	91	84	77	70	60	54	49	45	37	34	31	1.41
36	89	82	75	68	58	53	48	43	36	33	31	1.49
37	86	79	73	67	56	51	46	42	35	32	30	1.57
38	84	77	71	65	55	50	45	41	35	31	29	1.66

PROPERTIES AND REACTION VALUES

	124	114	105	96	85	77	70	64	55	50	47
S in.³	239.0	220.1	202.2	184.4	156.1	141.7	128.2	117.0	98.2	89.0	82.3
V kips	158	143	132	121	125	112	103	94	92	84	81
R kips	82	74	68	62	63	56	51	46	43	39	38
G kips	15.6	14.3	13.3	12.3	12.6	11.4	10.5	9.7	9.4	8.6	8.4
N in.	8.4	8.3	8.3	8.3	8.4	8.5	8.4	8.4	8.7	8.7	8.7

Courtesy of American Institute of Steel Construction

At the bottom of Table V are tabulated the following useful values: S = Section modulus, in inches³. V = maximum web shear, in kips. R = maximum end reaction, in kips, for $3\frac{1}{2}$-inch bearing. G = increase in R, in kips, for each additional inch of bearing. N = length of bearing in inches, to develop V.

For relatively short spans the allowable loads for beams and channels may be limited by the shearing or buckling strength of the web, instead of by the maximum bending stress allowed in the flanges. This limit is indicated in the tables by solid lines. Loads above these lines will produce the maximum allowable shear on the beam webs.

In the column at the right in the table of Allowable Loads are given the deflections for beams of various spans carrying the tabulated allowable loads. These deflections are figured on the basis of the nominal depth of the beam. Below the horizontal dotted lines the deflection exceeds $\frac{1}{360}$ of the span.

The table of Allowable Uniform Loads also shows by dotted and dashed lines whether standard or special end connections must be used.

PRACTICE PROBLEMS

1. Two angles are required to support a load of 4400 pounds uniformly distributed on a span of 6 feet. Determine the section by means of the section modulus.

2. A channel having a span of 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. 122.

4. What is the formula for computing the allowed unit stress for laterally unsupported beams?

5. A 12″ WF 40# beam is not laterally supported for 14 feet. Compute the allowed unit stress. Check by the table in Section 12 of the specifications in the Appendix.

6. From the tables of Allowable Uniform Loads select a 21″ WF beam of 20-foot span capable of carrying 6000 pounds per linear foot. Is a deeper beam more economical?

7. Compute the section modulus required for the beam in Problem 6. Look up in the Section Modulus Table the various beams that could be used.

8. What are the maximum deflections for the various beams that can be used in Problem 7?

9. Does the load on the beam in Problem 6 exceed the shear value for any of the beams that can be used?

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.

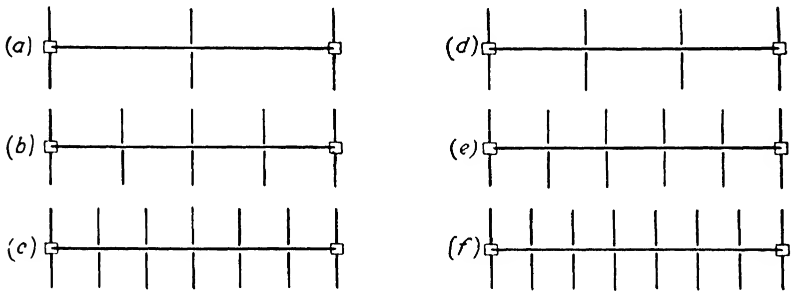


Fig. 94. Diagrams of Girders Showing Types of Joist Spacing

This can be applied to designing girders for floor panels. Fig. 94 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. 94, 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. 94.

Fig. 94-a shows plainly that the carrying of uniform floor load by joists to a girder does *not* change the moments at the points of concentration. These are the same as the bending moments for the same

load when carried uniformly distributed on the girder. This is only true, however, when there are joists at the supports of the girder, that is, at A and B. Omission of these end joists would result in bending moments higher than those due to an equal, uniformly distributed load.

Shearing Resistance. The statement is made on p. 104 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

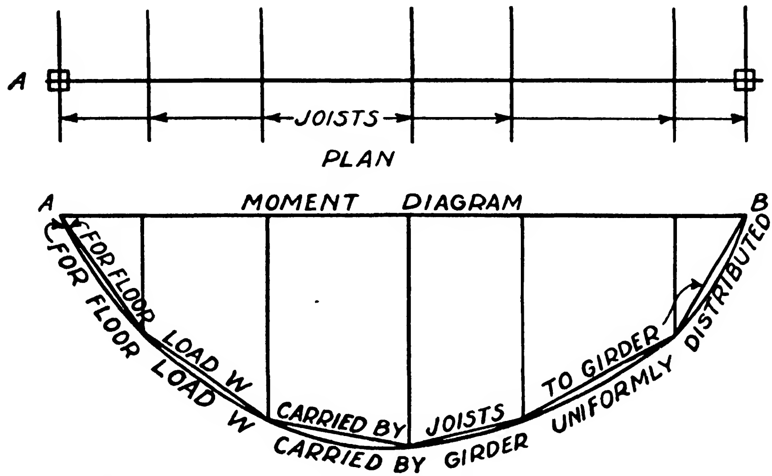


Fig. 94-a. This Diagram Illustrates the Influence of Joist Concentration

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 13,000 pounds per square inch. For example, to determine the shearing resistance of a 12″ I 40.8#:

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

then $$V = 5.52 \times 13{,}000 = 71{,}760\#$$

Problems. Compute the shearing resistance of the following beams and compare your results with those in the Steel Handbook: 12″ WF 50#, 12″ I 50#, 36″ WF 300#, 16″ WF 50#, 18″ WF 70#, and 18″ I 70#.

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.

Of more importance is the *strength of the standard riveted end connections* for beams. These are discussed later on in this chapter. 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. 78. 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. 105) 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 31.8#, span 15 feet, or 180 inches, load u. d. 32,000 pounds. The value of I for this beam is 215.8. Then

$$d=\frac{5}{384}=\frac{32{,}000\times180\times180\times180}{29{,}000{,}000\times215.8}=0.39''$$

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

$$d=\frac{1}{48}\times\frac{32{,}000\times180\times180\times180}{29{,}000{,}000\times215.8}=0.62''$$

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.,

$$d = 0.39'' + 0.62'' = 1.01''$$

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. The handbooks use a modulus of elasticity of 29,000,000 pounds per square inch; however, 30,000,000 pounds is also commonly used.

Safe Span Length. Based on a maximum deflection of $\frac{1}{360}$ of the span, a modulus of elasticity of 30,000,000, and on a unit stress of 20,000 pounds per square inch, the permissible span is *20 times the depth* for a uniformly distributed load and *25 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 Allowable Uniform Loads for Beams, indicate by dotted lines the loads for which deflections are greater than $\frac{1}{360}$ of the span. If, however, the unit stress is less than 20,000 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 15,000# u. d. The bending moment is 56,250 foot-pounds, which requires a 12″ WF 28#.

The length of this beam is 30 times its depth, therefore the deflection of a 12-inch beam will be excessive unless the stress is less than

20,000#/□″. If it is decided to make the depth of the beam $\frac{1}{20}$ of the span, the section required is 18″ WF 47#. This beam, if loaded to full capacity, would deflect just to the allowed limit. But the resisting moment of an 18″ WF 47# beam is 137,160 foot-pounds (more than twice the bending moment computed), 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 14″ WF 34# and compute the deflection from the formula

$$d=\frac{5}{384}\ \frac{Wl^3}{EI}=\frac{5}{384}\times\frac{15{,}000\times360\times360\times360}{30{,}000{,}000\times339.2}=0.9''$$

As the computed deflection is less than the allowed amount, the 14″ WF beam is satisfactory.

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

$$I=\frac{5}{384}\times\frac{Wl^3}{Ed}=\frac{5}{384}\times\frac{15{,}000\times360\times360\times360}{30{,}000{,}000\times1}=304$$

The beam having a value of I next higher than 304 is a 14″ WF 34#.

It will be noted that in this example a modulus of 30,000,000 pounds per square inch is used. If a value of 29,000,000 pounds were used, the results would change only slightly.

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. 78 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 beam is required to support a u. d. load of 7500 pounds on a span of 24 feet, the permissible deflection being $\frac{1}{2}$ 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 the 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 15 times the width. The unit stresses given in the A.I.S.C. Specification permit full load when the unsupported length is 15 times the width.

To illustrate the use of these tables assume a 12″ I 31.8# 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 Section 4 of the A. I. S. C. Specification, in the Appendix, the table of "Allowable Unit Stresses for Beams and Girders Laterally Unsupported" shows that only 85.2 per cent of the unit stress given by the beam formula can be used.

Problems. 1. What is the safe resisting moment of an 8″ WF 17# 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. 95 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 $5'\text{-}4\frac{1}{2}''$ *c. c.*

Area supported by one joist $16 \times 5\frac{3}{8} = 86$ sq. ft.
dead load on one joist $86 \times 80 =$ 6880#
live load on one joist $86 \times 100 =$ 8600#
Total load 15,480#

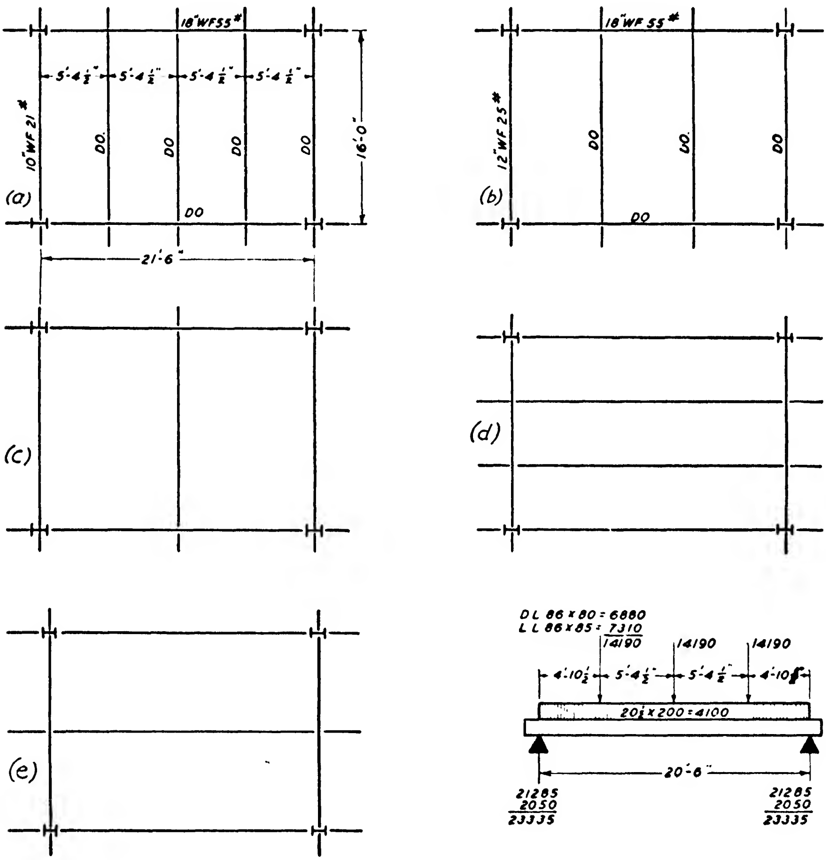


Fig. 95. A Panel of Floor Framing

This total load, 15,480 pounds, is uniformly distributed on a span 16 feet. The table of safe loads indicates 10″ WF 21#.

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 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 $\begin{cases} +21{,}285\times 10\frac{1}{4} = 218{,}171 \\ -14{,}190\times \ 5\frac{3}{8} = \underline{-76{,}271} \end{cases} = 141{,}900$

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

From the tables in the steel handbooks a 18″ WF 55# 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″ WF 25#; and for the girders 18″ WF 55#.

PRACTICE PROBLEMS

1. Determine sizes of joists and girders required for Scheme (*c*).
2. Determine sizes of joists and girders required for Scheme (*d*).
3. In Scheme (*d*) 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 beams required for Scheme (*e*).
5. Investigate all the beams in the foregoing problems as to shear, deflection, and strength of standard end connections.
6. Compute the weight of the 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″ WF 21#×15′-11″=1340#
   1 18″ WF 55#×20′- 6″=1130#

7. 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. There are several types of floor systems in common use. Some types of floor construction do not utilize steel joists at all. Floor systems are discussed in the chapter on Multistory Buildings.

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.

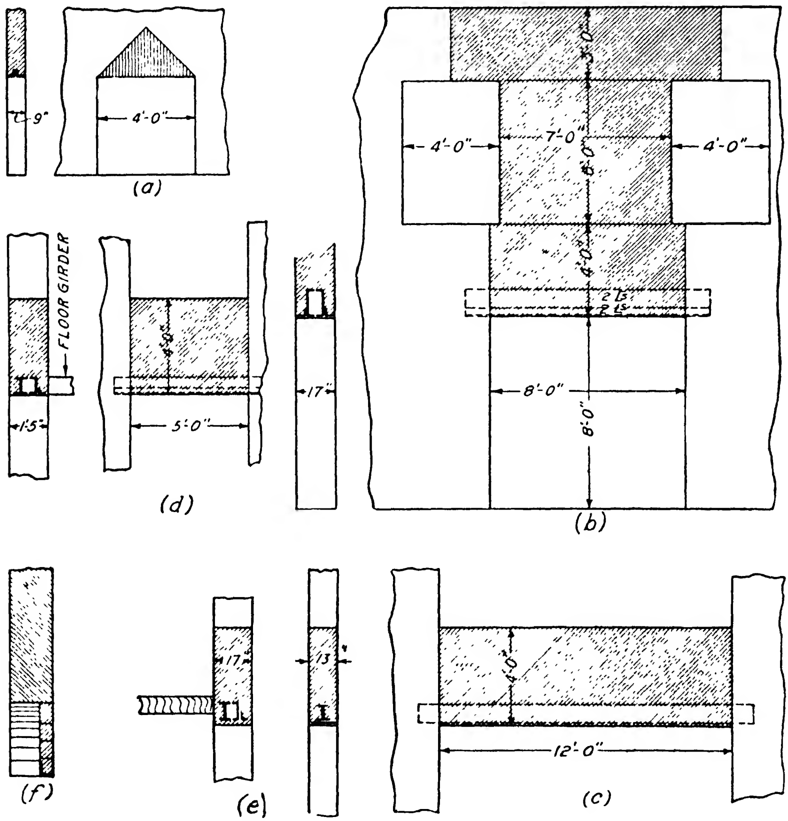


Fig. 96. Types of Lintels

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. 96 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*.

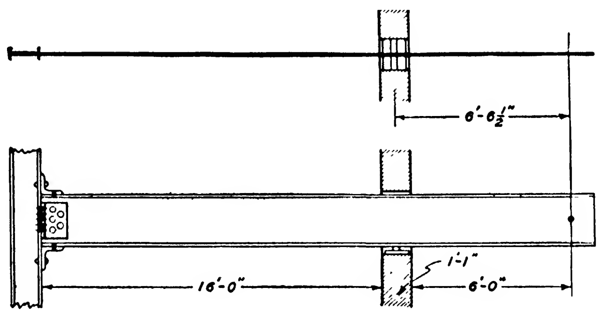


Fig. 97. Cantilever Construction

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.

PRACTICE PROBLEMS

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 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

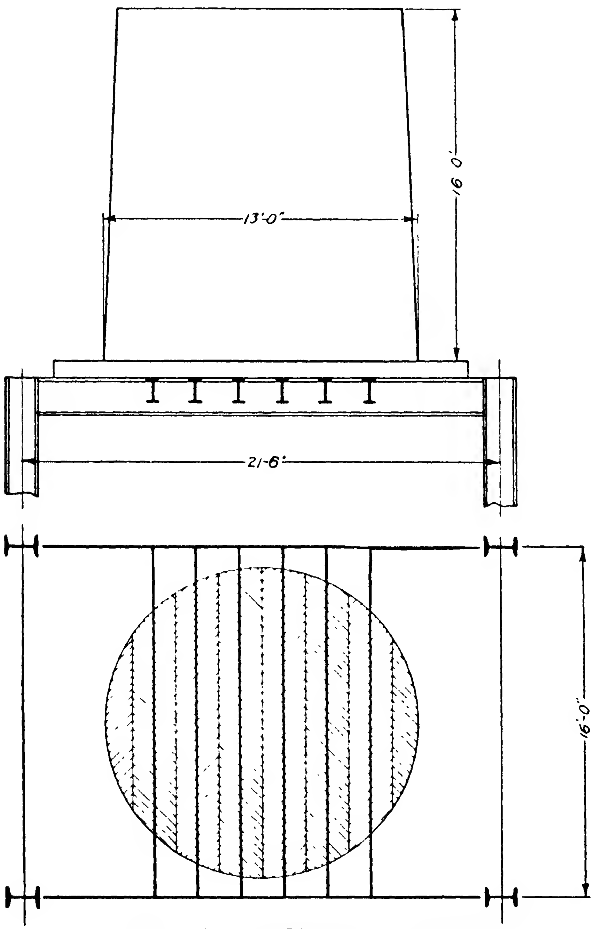


Fig. 98. Plan and Elevation of Tank Support

channel are required? Neglect the value of the angle.

5. In Case *c* 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. 97 shows a beam projecting beyond the wall of a building, that is, a cantilever beam. The projection is 6 feet from

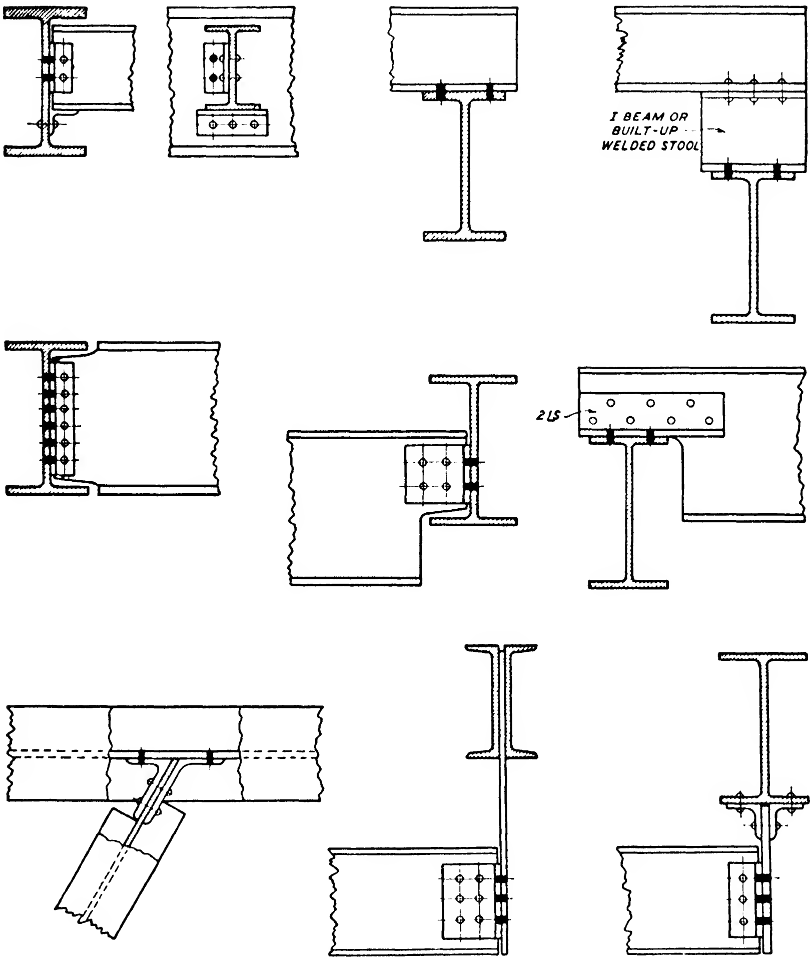


Fig. 99. Types of Beam Connections

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 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. 98 illustrates the framework for supporting a wood water tank. The tank rests on $4'' \times 6''$ 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.

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 beams. Check for strength of end connections.

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. 99. No stress is transmitted by these rivets or bolts. They serve simply to hold the beams in position.

Angle Connections. The most common method of connecting one beam to another is by means of angles riveted to the web. In Fig. 100 are shown standard beam connections. A connection in Series *A* is used for the general run of work—provided the strength is adequate. A connection in Series *H* is used where extra strength is required due to thin web, concentrated load, or short span. A Series *M* connection is used only where unavoidable. The steel handbooks show tabulated the various beams and minimum span that can be used with the different types of connections.

The strength of the two-angle connections may be limited by

1. Rivets in joist in double shear

2. Rivets in girder in single shear
3. Rivets in bearing in web of joist
4. Rivets in bearing in web of girder

For example, take the standard connection for an 18″ WF 64#. See Fig. 100, Series *A* connections. All rivets $\frac{7}{8}$″ in diameter.

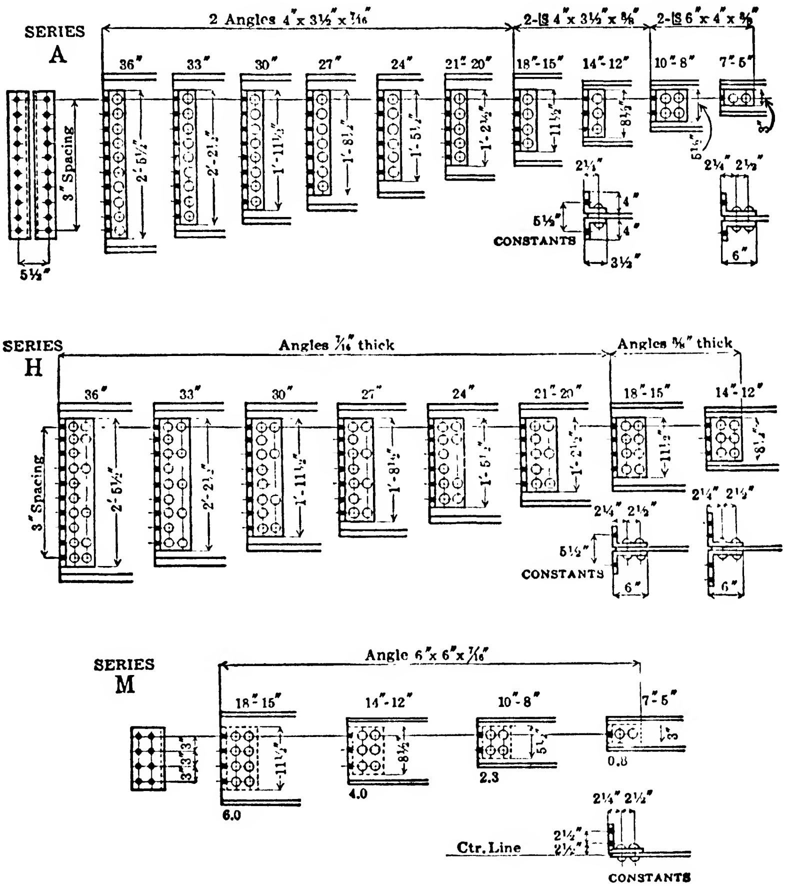


Fig. 100. Standard Beam Connections

1. 4 rivets in joist in double shear

$$4 \times 18{,}040 = 72{,}160\#$$

2. 8 rivets in girder in single shear

$$8 \times 9020 = 72{,}160\#$$

3. 4 rivets in bearing in web of joist

$$4\times.403\times.875\times40{,}000=56{,}300\#$$

4. 8 rivets in bearing in web of girder. The thickness of the web is not given. It must be the following thickness for a connection on one side only to have the same strength as the rivets in single shear.

$$t=\frac{9020}{.875\times32{,}000}=0.32 \text{ inch}$$

If there is a connection on the opposite side, the thickness of web would have to be

$$t=\frac{18{,}040}{.875\times40{,}000}=.52 \text{ inch}$$

The strength of this connection, 56,300#, corresponds to the maximum safe u. d. load on a span of about 14 feet. It is less than the shearing strength 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 frequent. If it does happen, however, angles with 6-inch legs may be used to provide space for more rivets.

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. 101. A number of special connections are shown in Fig. 99 and need no explanation.

The sketches shown in Fig. 101 indicate the standard methods used to provide clearance for beams connecting to beams or columns. Where possible, a minimum clearance of $\frac{1}{2}$ inch should be provided. The fabricator should be consulted regarding his standard copes and blocks.

The coping or blocking of beams should be avoided wherever possible. When the construction will permit, the elevation of the connecting beams or joists should be established a sufficient distance below the top of girders to clear the thickest girder flange.

Unusually long or deep copes and blocks, or blocks in beams with thin webs, may materially affect the capacity of the beam. Such beams must be investigated for both shear and moment at lines *A* and B and, when necessary, additional section must be provided. Some fabricators designate all the operations shown in Fig. 101 by the term "cuts."

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.

Seat Connections. The seat connection is shown in Fig. 102. This seat or bracket is made up of a shelf angle, one or two stiffener angles,

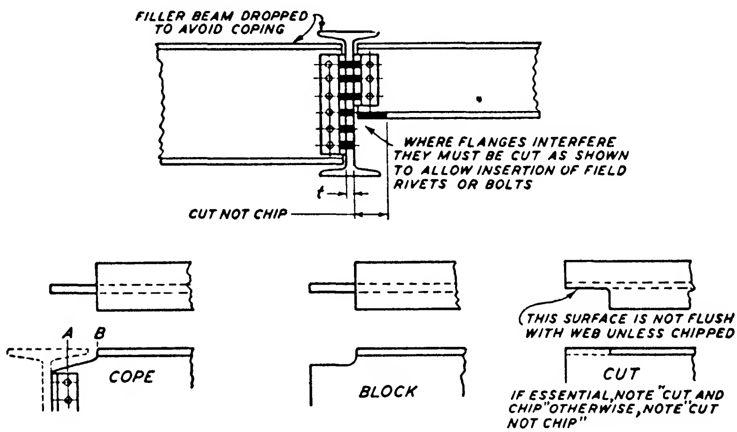


Fig. 101. Coping and Blocking of Beams

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. 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-

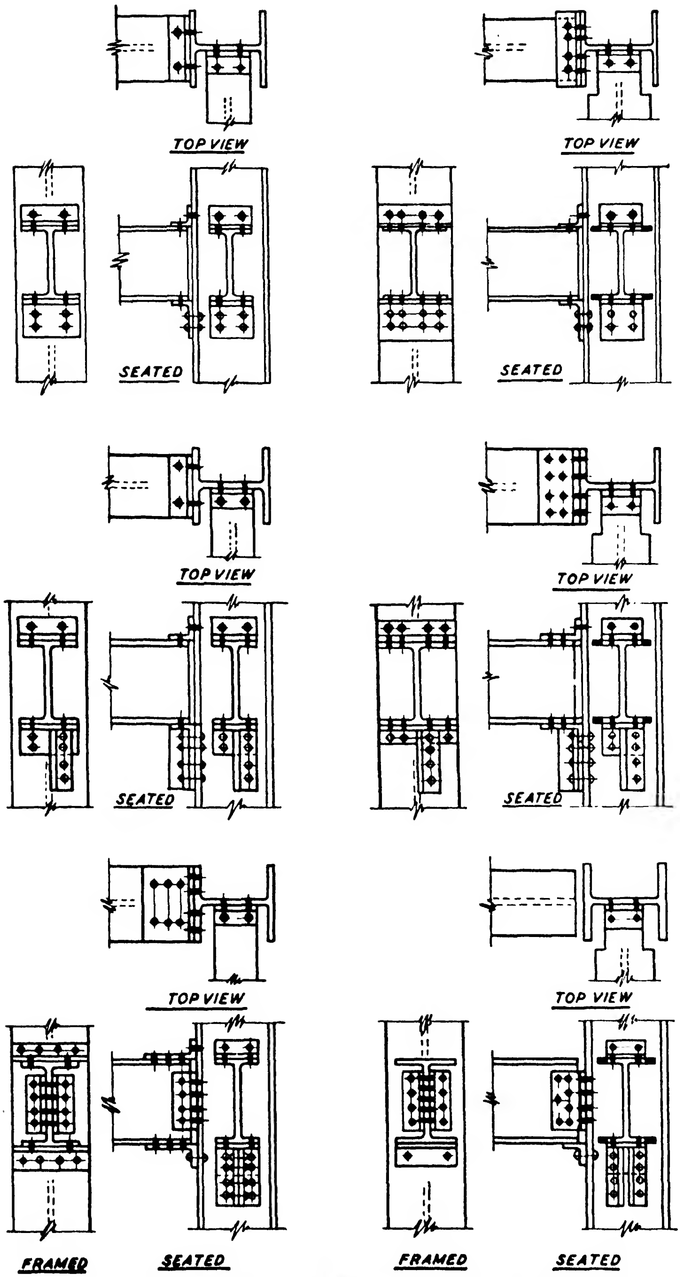


Fig. 102. Types of Seated or Framed Connections of Beams to Columns

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. 102 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 number of field rivets is small

Web Connections. The web or framed connection is made by means of two angles, Fig. 102. The legs parallel to the beam rivet to

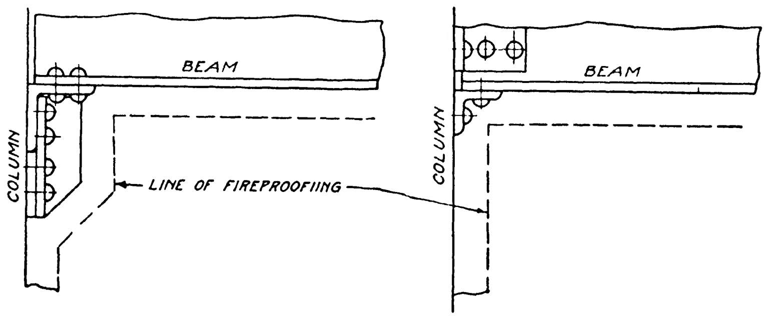


Fig. 103. Diagrams Showing Disadvantage of Seat Connection for Fireproofing

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 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. 103.

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.

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 satisfactory, Fig. 104. This consists of a piece of gas pipe with a bolt running through it. This form is used in lintels and in grillage beams.

A type of separator commonly used is made of angles or angles and plate. This type not only serves as a spacer, but also stiffens the web and helps to transmit the load from one beam to the other in case one is loaded more heavily. Specifications usually require that separators be spaced not over six feet apart. They should be placed under concentrated loads and over bearings.

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 usually 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 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.

For further information regarding this type of floor system, see

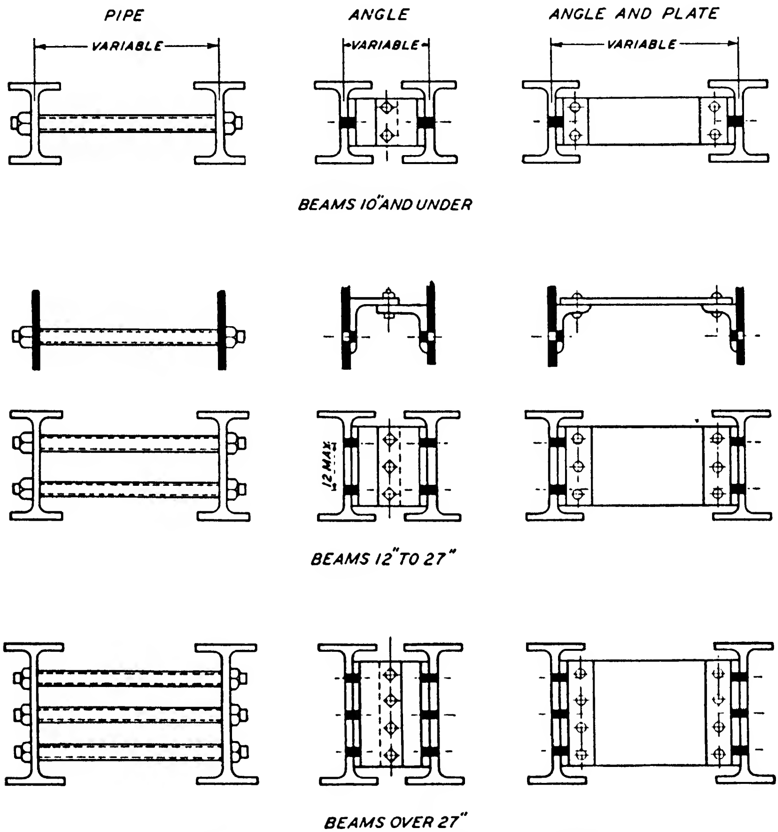


Fig. 104. Typical Built-up Beam Separators

the chapter on Multistory Buildings. Tie-rods are also used between purlins on sloping roofs of mill buildings, and when thus used are called sag rods. See chapter on Industrial Buildings.

Bearings. When a beam is supported on a masonry wall or pier, it is necessary to distribute the beam reaction over an area sufficient to keep the pressure on the masonry within that allowed. Steel bearing plates are usually used for this purpose. An arrangement, as shown in Fig. 107, using a sole and masonry plate should be used.

It has been common practice to use only one bearing plate at the reactions; however, this is not good construction, as, when the beam deflects, the bearing is put on the front edge of the masonry plate. This causes an eccentric load on the masonry wall or pier, and also

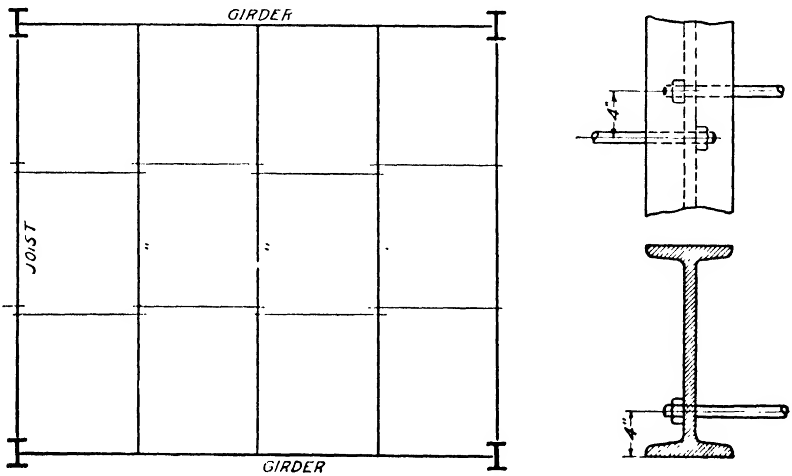


Fig. 105. Layout Showing Tie-Rod Connections Between Joists

tends to cause cracking and spalling of the masonry under the inside edge of the masonry plate, due to the high bearing stress.

To illustrate the design of a bearing, assume a 14″ WF 43# beam has an end reaction of 30,000#. The bearing is on hard brick in cement

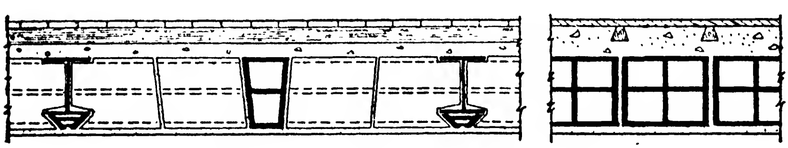


Fig 106. Hollow Tile Arch Floor Construction

mortar, for which the allowed unit bearing is 250 pounds per square inch. Then the required area of the masonry plate is

$$\frac{30{,}000}{250} = 120 \text{ sq. in.}$$

A plate 10″×12″ would be used.

The sole plate or bearing rib is made between 3 and 4 inches wide, $\frac{1}{2}$ inch thick, and as long as the width of the flange. This sole plate can be welded or riveted to the bottom flange.

With a $10'' \times 12''$ masonry plate and a $4'' \times 8''$ sole plate, the overhang of the masonry plate is 3 inches in one direction and 2 inches in the other.

To determine the thickness, assume a strip 1 inch wide. Use the largest overhang of 3 inches. Then the cantilever beam is 1 inch wide, 3 inches long, with a load of 250 pounds per inch.

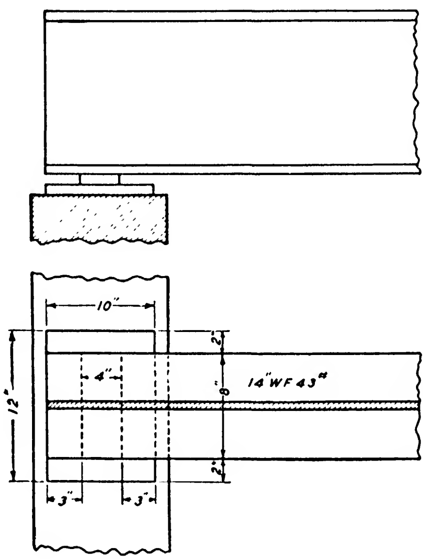


Fig. 107. Bearings for Beams

The bending moment is

$$3 \times 250 \times \tfrac{3}{2} = 1125''\#$$

From the bending moment, the required section modulus $\frac{I}{c}$ can be obtained

$$\frac{I}{c} = \frac{M}{S} = \frac{1125}{20{,}000} = 0.056$$

From the section modulus the thickness t can be computed

$$I = \frac{bt^3}{12} \qquad b = 1 \qquad c = \frac{t}{2}$$

$$\frac{I}{c} = \frac{t^3}{12\frac{t}{2}} = \frac{1}{6}\,t^2$$

$$t^2 = 6 \times \frac{I}{c} = 6 \times .056 = .336$$

$$t = \sqrt{.336} = .58, \text{ or } \tfrac{5}{8}'' \text{ thick}$$

The square root can be obtained from tables in the steel handbooks.

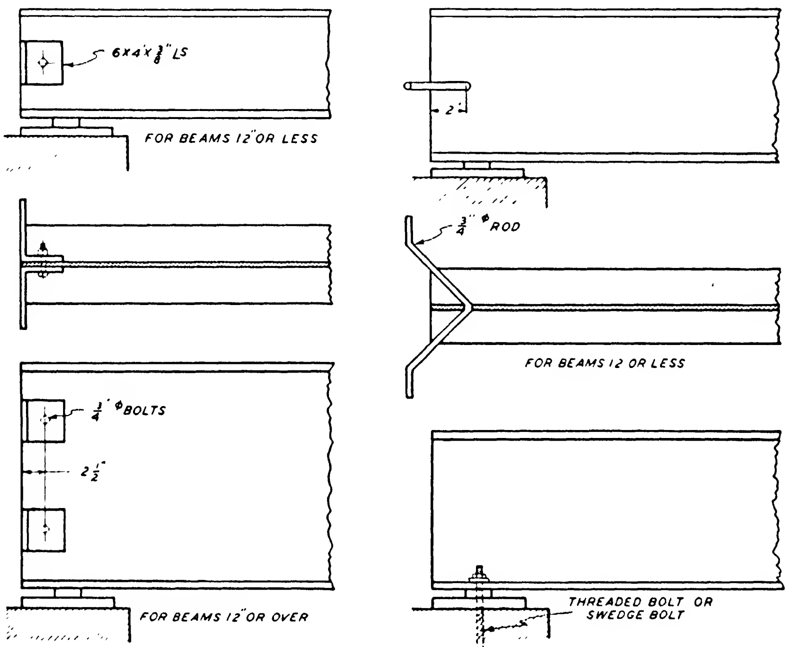


Fig. 108 Anchors for Beams

These masonry plates are made a minimum of $\frac{1}{2}$ inch thick. For heavy reactions the steel masonry plates may be reduced in size by setting them on stronger masonry or concrete built into the wall or pier in order to distribute the load over the weaker masonry.

Where the beam and reaction are such that it would require the use of a wider bearing than the 4 inches provided by the sole plate, end stiffeners are used on the beam.

Anchors. Steel beams supported by masonry should always be properly anchored to the wall or pier. Fig. 108 shows some typical beam anchors.

Fig. 109 shows shop detail drawings of beams used in an office building.

Miscellaneous Details. Almost every structure presents some conditions requiring special details of the beams. The relative posi-

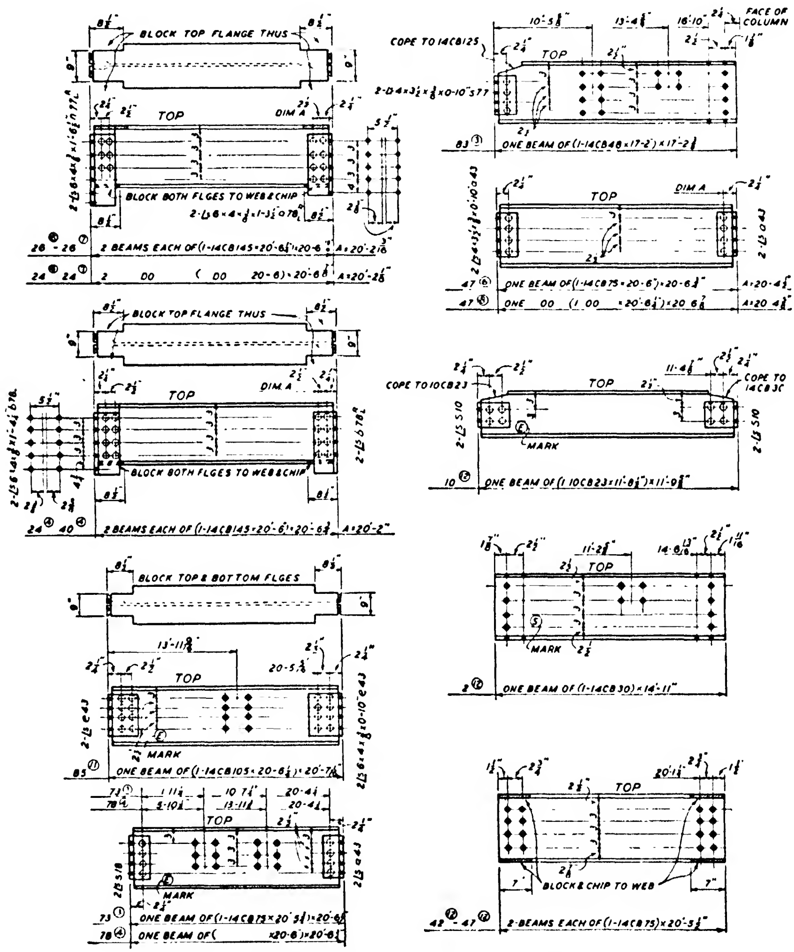


Fig. 109. Shop Detail Drawing of Beams for Office Building
Courtesy of A. T. & S. F. Railway System

tion 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.

CHAPTER VIII

BUILT-UP GIRDERS

Plate girders or built-up girders are used whenever the situation requires a girder of greater flexural or shearing capacity than available in the rolled sections. The largest section now rolled is the 36″ WF 300# beam which has a resisting moment of 1,841,800 foot-pounds and a web shear resistance of 451,100 pounds. The most common forms of built-up girders are shown in Fig. 110.

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. 111.

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 rolled section, such as the I beam, is taken from the tables in the handbook, but for the built-up girder it must be computed.

Chord Stress Method. The second method of designing built-up 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. 111, t and c represent, respectively, the total tension and total com-

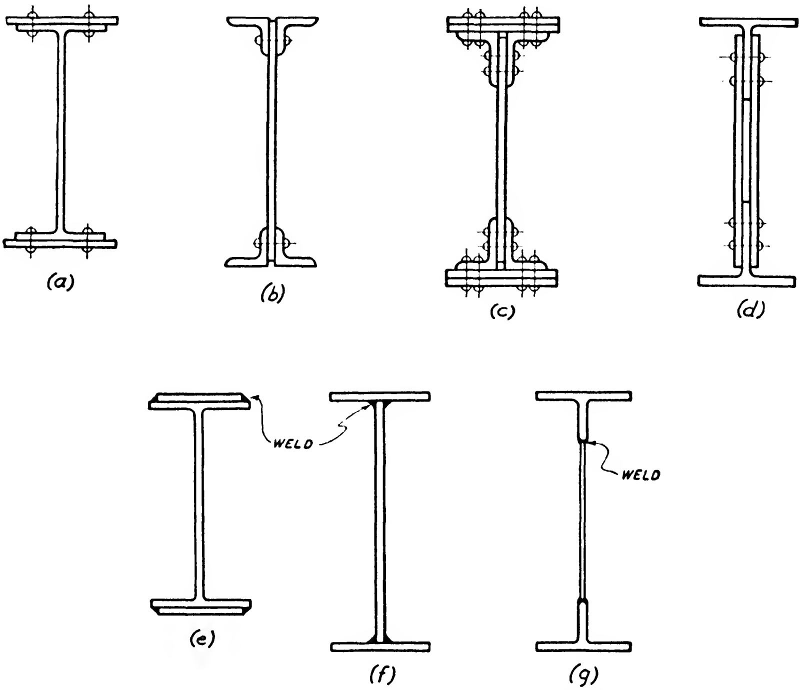


Fig. 110. Built-up Riveted and Welded Girders

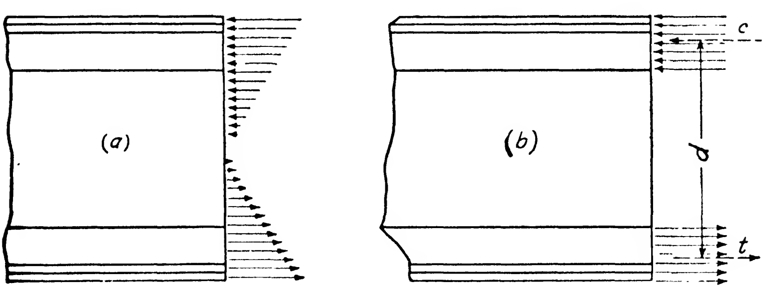


Fig. 111. Diagram of Bending Stresses in a Riveted Girder. (a) Moment of Inertia Method, (b) Chord Method

pression 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 18,000 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, 20,000 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 20,000, or 18,000 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; this using net area as the basis in designing. If design is based on gross area, as now permitted by A.I.S.C. Specifications, $\frac{1}{6}$ of the web area should be used.

The chord stress or flange stress method is usually used only for selecting a trial girder section and then checking the section by the moment of inertia method.

Illustrative Example. Assume M equals 580,000 foot-pounds; total depth of girder, 44 inches; approximate value of $d=42$ inches. To find the required section.

$$M = 580{,}000 \times 12 = 6{,}960{,}000 \text{ in.-lbs.}$$

$$\tfrac{42}{44} \times 20{,}000 = 19{,}100$$

$$A = \frac{6{,}960{,}000}{19{,}100 \times 42} = 8.7 \text{ sq. in.}$$

$$A = \begin{cases} \frac{1}{6} \text{ web } 43\frac{1}{2} \times \frac{3}{8} = 2.7 \text{ sq. in.} \\ 2\,\text{L}^s\ 5 \times 3\frac{1}{2} \times \frac{3}{8} = \underline{6.1} \text{ sq. in.} \end{cases}$$
$$8.8 \text{ sq. in.}$$

As the area of the chosen section is greater than the calculated value, it is satisfactory.

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.

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. 112. The span is 45′-0″; load 6000 pounds per lineal foot equals total load of 270,000 pounds; end shear 135,000 pounds; maximum bending moment 18,225,000 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. The effects of some of these conditions can be stated in general terms as follows:

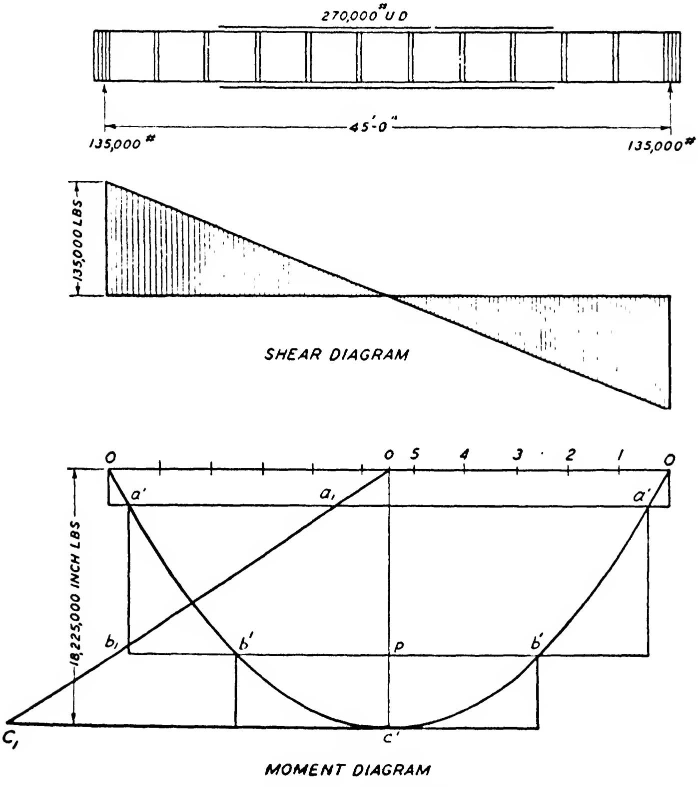


Fig. 112. Plate Girder Design

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 results.

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. 112, 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. The specification allows a minimum web thickness of $\frac{5}{16}$ inch for outside work and $\frac{1}{4}$ inch for inside locations.

The unit stress in shear assumed for this design is 13,000. This is the average shear on the net cross section of the web. In the example, Fig. 112, the maximum shear is 135,000 pounds; then the net area of the web must be $\frac{135,000}{13,000}$ or 10.4 square inches. The thickness required to give the net cross section is $\frac{10.4}{48}$ or 0.22 inches. Hence a plate 0.22 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. Specification requires that a plate girder web shall have a thickness of not less than $\frac{1}{170}$ of the unsupported distance between flanges.

$$48\tfrac{1}{2} - 12 = 36\tfrac{1}{2}'' \text{ between flanges}$$

$$\tfrac{5}{16} \div 36.5 = \frac{1}{117} > \frac{1}{170} \quad \text{therefore } \tfrac{5}{16}\text{-inch web is satisfactory.}$$

Problem. What thickness of web is required for a shear of 220,000 pounds, depth 44 inches?

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, $I = Mc/S$.

In the example, Fig. 112, $M = 18{,}225{,}000$ in.-lb.; $S = 20{,}000\#$. 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

$$I = \frac{18{,}225{,}000 \times 24\frac{1}{4}}{20{,}000} = 22{,}100$$

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.

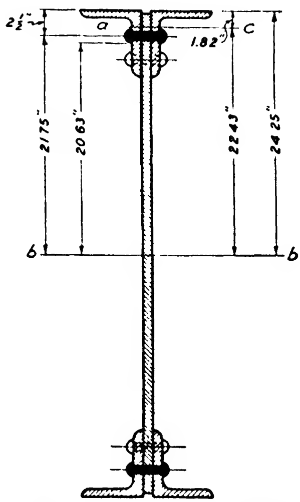


Fig. 113. Section of Plate Girder Without Flange Plates

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 22,100 leaves 19,220 as the net value of I to be supplied by the flanges. The general formula for moment of inertia is

$$I = Ar^2$$

In this case r is about 22.5 inches, then $r^2 = 506$, and $A = \dfrac{19{,}220}{506}$ or

38 square inches. This is the gross area of the two flanges, or 19 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 2 L^s $6''\times6''\times\frac{7}{8}''$, having an area of 2×9.73 or 19.46 square inches.

Checking Fig. 113 from tables found in the steel handbooks,

$$I=\begin{cases}1 \text{ Web plate } 48''\times\frac{5}{16}''= & 2{,}880\\ 4 \text{ L}^s\ 6''\times6''\times\frac{7}{8}'' = & \underline{19{,}710}\end{cases}$$

$$\text{Total value of } I \quad 22{,}590$$

The moment of inertia of the section is somewhat larger than the required amount; therefore the section is satisfactory.

The A.I.S.C. Specification requires no deduction be made for rivet holes in either flange except where the reduction exceeds 15 per cent of the gross flange area.

$$\text{*Percentage of rivet holes}=\frac{2\times\frac{7}{8}\times\frac{7}{8}}{19.46}\times100=7.9<15\%$$

No increase in gross area is necessary.

If it is required to design this girder by net area rules, the gross area of the flange will usually have to be made larger to allow for the rivet holes.

$$A=\frac{I}{r^2}=\frac{19{,}220}{506}=38 \text{ sq. in.}$$

This A is then the net area of the two flanges. The gross section must be larger to allow for the rivet holes.

$$4\times\tfrac{7}{8}\times\tfrac{7}{8}=3.06 \text{ sq. in.}$$

The gross area required is then $38+3=41$ sq. in. or 20.5 sq. in. for each flange.

Use $\quad 2 \text{ L}^s\ 6\times6\times\frac{15}{16}$

$$A=2\times10.37=20.74 \text{ sq. in.}$$

Checking the value of I and making the necessary corrections for rivet holes,

1 Web plate $48''\times\frac{5}{16}''$	$=$ 2,880	
Deduct for holes, $2\times\frac{7}{8}\times\frac{5}{16}\times21.75\times21.75=$	260	2,620
4 L^s $6''\times6''\times\frac{15}{16}''$	$=$20,970	
Deduct for holes, $4\times\frac{7}{8}\times\frac{15}{16}\times21.75\times21.75=$	1,540	19,430
Total value of I		22,050

*To simplify the working of equations, inch signs are sometimes omitted.

The moment of inertia of the section is within 2% (the usual allowed variation); therefore the section is satisfactory.

In deducting for rivet holes, the diameter deducted is $\frac{1}{8}$ inch larger than the diameter of rivet used; therefore $\frac{7}{8}$ inch is deducted for a $\frac{3}{4}$-inch rivet. The distance to the holes is taken at the outer of the two rows of holes.

Case B—With Flange Plates. When cover plates are required, it is specified that the area in the flange angles shall not be less than

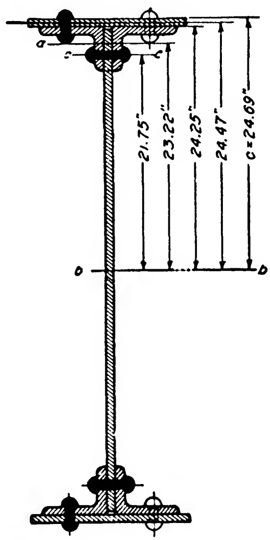


Fig. 114. Section of Plate Girder with Flange Plates

30 per cent of the flange area. Also, no cover plate shall be thicker than the angles, and the thicknesses of the plates shall diminish from the flange angles outward. For a trial section, use 2 Ls $6''\times4''\times\frac{5}{8}''$ and 1 Pl. $14''\times\frac{7}{16}''$. See Fig. 114. Then the gross area of one flange equals

2 Ls $6''\times4''\times\frac{5}{8}''$		$2\times5.86=11.72$
1 Pl. $14''\times\frac{7}{16}''$		$=\ 6.12$
	Total area	17.84 sq. in.

The section is shown in Fig. 114. 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.

$$I=\frac{18{,}225{,}000\times 24.69}{20{,}000}=22{,}500$$

The value of I for the assumed section is as follows. These values of I can be obtained from the steel handbooks.

$$\begin{array}{l}
1 \text{ Pl. } 48''\times\tfrac{5}{16}'' \;=\; 2{,}880\\
4\ \text{L}^s\ 6''\times 4''\times\tfrac{5}{8}''=12{,}667\\
2 \text{ Pl. } 14''\times\tfrac{7}{16}'' \;=\; \underline{\ 7{,}336}\\
\text{Total value of } I=22{,}883
\end{array}$$

Check for the percentage of the rivet holes in the flange.

$$\text{Percentage of rivet holes, }\frac{(1\times 1\tfrac{1}{16}\times\tfrac{7}{8})+(2\times\tfrac{5}{8}\times\tfrac{7}{8})}{17.84}\times 100=11.3<15\%$$

No increase in gross area is required.

As the moment of inertia of this section is slightly larger than the required amount, the section is satisfactory.

If it is required to design by net area rules, the gross area will have to be made larger to allow for the rivet holes. For trial use, 2 L^s $6''\times 4''\times\frac{5}{8}''$ and 1 Pl. $14''\times\frac{9}{16}''$. Then the gross area of one flange equals

2 L^s $6''\times 4''\times\frac{5}{8}''$	$2\times 5.86=11.72$
1 Pl. $14''\times\frac{9}{16}''$	$=7.88$
Total area	19.60 sq. in.

For this section the value of c is $24.25+.56=24.81$. The required I must be corrected to correspond.

$$I=\frac{18{,}225{,}000\times 24.81}{20{,}000}=22{,}600$$

Making the necessary corrections for rivet holes, the value of I for the assumed section is

1 Pl. $48''\times\frac{5}{16}''$	$=2{,}880$	
Deduct for holes, $2\times\frac{7}{8}\times\frac{5}{8}\times 21.75\times 21.75$	$=260$	2,620
4 L^s $6''\times 4''\times\frac{5}{8}''$	$=12{,}667$	
Deduct for holes, $4\times\frac{7}{8}\times\frac{5}{8}\times 21.75\times 21.75$	$=1{,}036$	11,631
2 Pl. $14''\times\frac{9}{16}''$ less 1 rivet hole		
$2\times 13\frac{1}{8}''\times\frac{9}{16}''\times 24.53\times 24.53$		$=8{,}885$
Total net value of I		$=23{,}136$

This value of I is somewhat in excess of the required amount; therefore, a section using $14''\times\frac{1}{2}''$ cover plates should be investigated.

Check should also be made of the percentage of rivet holes in the flange.

Problem. Compute the net moment of inertia for a section as used in the above illustrated example but with 14″ x $\frac{1}{2}$″ cover plates. Check the percentage of rivet holes in the flange.

The sectional areas of the two designs are

Case A
$$1 \text{ Pl. } 48\times\tfrac{5}{16}=15.00$$
$$4 \text{ L}^s\ 6\times6\times\tfrac{7}{8}=38.92$$
$$53.92 \text{ sq. in.}$$

Case B
$$1 \text{ Pl. } 48\times\tfrac{5}{16}=15.00$$
$$4 \text{ L}^s\ 6\times4\times\tfrac{5}{8}=23.44$$
$$2 \text{ Pl. } 14\times\tfrac{7}{16}=12.24$$
$$50.68 \text{ sq. in.}$$

This showing is slightly in favor of Case *B*. Also, for this case the cover plates do not extend the full length of the girder. Case *B* has another advantage in greater lateral stiffness due to wider flanges. On the other hand, the cost of the additional riveting may amount to more than the saving in weight. Also, the use of 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 cover plates is preferred.

Lateral Support. Girders, like beams, must have their compression flanges supported laterally to prevent buckling. The specification requires a reduction of the unit stress unless the $\frac{l}{b}$ ratio is 15 or less. For the preceding illustrated problems, Case *A* would have to have the top flange laterally supported about every $15\frac{1}{4}$ feet or less; in Case *B*, about $17\frac{1}{2}$ feet or less.

Case *A*
$$\frac{l}{b}=\frac{15\frac{1}{4}\times12}{12\frac{5}{16}}=14.9$$

Case *B*
$$\frac{l}{b}=\frac{17\frac{1}{2}\times12}{14}=15$$

In most cases the lateral support is provided by the joists or floor construction. Where this is not the case, wide compression-flange plates can be used or a channel placed on the compression flanges.

If it is not possible to support the compression flange so that the

$\frac{l}{b}$ ratio is 15 or less, a reduced unit stress as required in Section 12 of the specification should be used instead of 20,000 lbs. per sq. in.

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 twelve times the thickness of the thinnest plate, or a maximum of six inches. In the above example this limit is $12 \times \frac{7}{16}''$ or $5\frac{1}{4}''$. This permits a distance of $3\frac{1}{2}$ inches between the gage lines, which is satisfactory, as it is less than will be used.

The customary widths of flange plates vary by 2 inches, thus, 10-inch, 12-inch, 14-inch, etc. For 6-inch flange angles the maximum 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 the same or 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. 112, 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 graphical methods with sufficient accuracy.

Solution for Uniformly Distributed Loads. Let Fig. 115 at *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 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

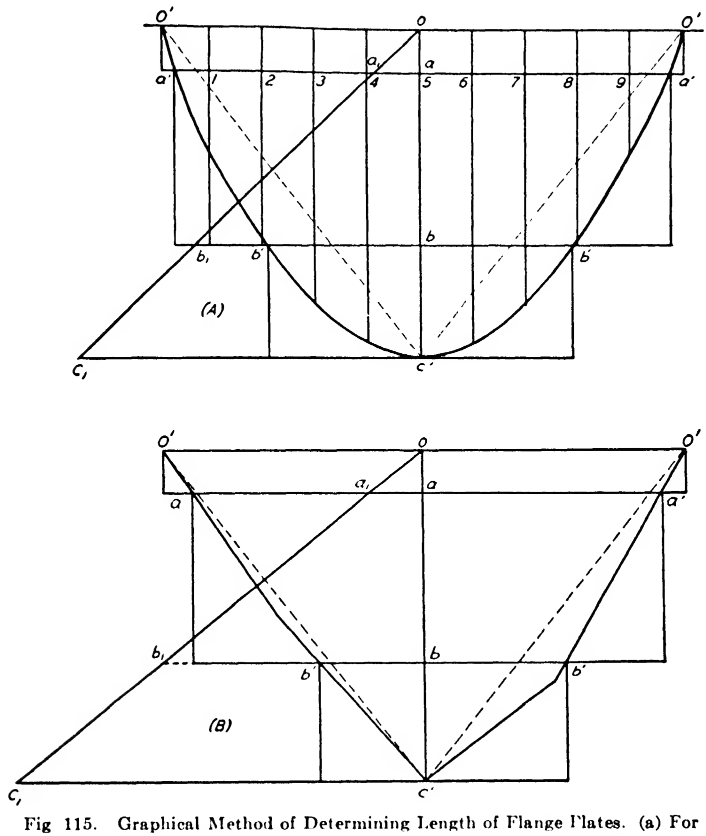


Fig 115. Graphical Method of Determining Length of Flange Plates. (a) For Uniformly Distributed Loads; (b) For Concentrated Loads

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.

This method is based on the formula $I = Mc/S$. Since c is practically constant throughout the length of the girder and the allowable unit stress S is also constant, I is proportional to M.

Sometimes the moment of resistance of the section is greater than the maximum bending moment. In that case the horizontal line C_1C^1, Fig. 115, representing the resisting moment would extend below the point c', which represents the bending moment.

As the moment diagram for a uniform load is a parabola, the lengths of the cover plates can be determined by the following equations:

$$L_1=L\sqrt{\frac{a_1}{A}}\;;\; L_2=L\sqrt{\frac{a_1+a_2}{A}}\;;\text{ etc.}$$

L = length of girder; L_1, L_2, etc. = lengths of cover plates, outer first; a_1, a_2, etc. = areas of the cover plates, from outer to inner; A = total area of flange.

Solution for Concentrated Loads. Fig. 115 at B represents a moment diagram for concentrated loads. The lengths of the cover plates can be determined graphically just as described for uniform loads.

The theoretical length of cover plate is increased 12 inches to 18 inches at each end to provide for a few extra rivets at each end.

Web Stiffeners. Specifications require that stiffeners be placed over bearing points and under concentrated loads. These stiffeners should not be crimped. The A.I.S.C. Specification requires intermediate stiffeners if $\frac{h}{t}$ is equal to or greater than 70.

Stiffeners at Loaded Points. The chief purpose of stiffeners at points of concentrated load is to transfer 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 the flange, it is 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. The size of these angles is determined by the area required for bearing. Only those portions of the outstanding legs outside the fillets of the flange angles are considered effective in bearing. These stiffener angles must be attached to the web with enough rivets to transmit the load. Generally, the bearing value of rivets in the web plate will govern.

To illustrate, the end stiffeners for the girder in Case A will be determined. End reaction, 135,000 lbs. Allowed bearing on outstanding leg of stiffener angles, 30,000 lbs. per sq. in.

An angle with a 5-inch outstanding leg will be used.

$$\text{Length available for bearing} = 5 - \tfrac{1}{2} = 4\tfrac{1}{2}''$$

$$\frac{135{,}000}{4.5 \times 30{,}000} = 1.0'' = \text{thickness required.}$$

As an angle will be used on each side of the web, the thickness of the angles required is $\frac{1}{2}$ inch. Therefore, two angles $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ can be used. As stiffeners at bearing and under concentrated loads cannot be crimped, it is necessary to place $3\frac{1}{2}'' \times \frac{5}{8}''$ fills under these angles. As the web is $\frac{5}{16}$ inch thick, the value of one rivet in bearing is 9380 lbs.

$$\text{Number of rivets required} \frac{135{,}000}{9380} = 14.4$$

Therefore, 15 rivets should be used for connecting the stiffeners.

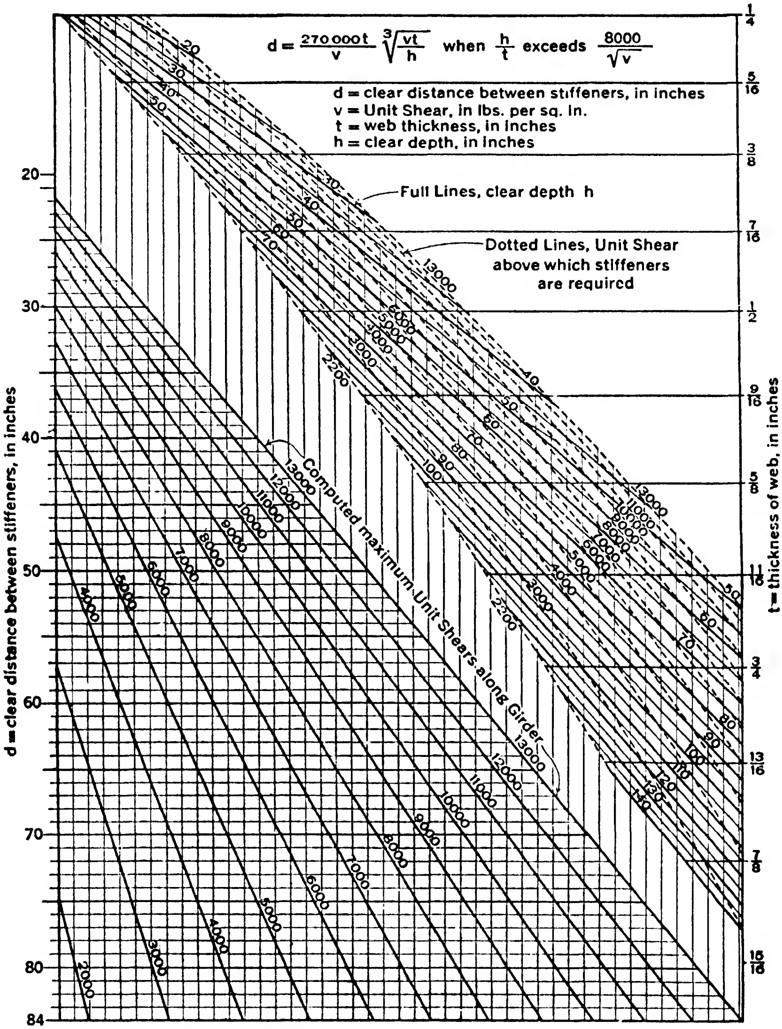
The condition at concentrated loads is analogous to that at end bearings, and is treated in the same manner. The use of stiffeners under a concentrated load is shown in Fig. 118.

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 ratio of the unsupported depth to the web thickness $\left(\frac{h}{t}\right)$ is greater than 70. Such stiffeners are required at all points where $\frac{h}{t}$ exceeds $\frac{8000}{\sqrt{v}}$. The value v = the greatest unit shear in the panel. The clear distance between stiffeners shall not exceed 84 inches or that given by the formula

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

To illustrate, the intermediate stiffeners for the girder in Case A will be determined.

$$\frac{h}{t} = \frac{36.5}{\frac{5}{16}} = 117$$



84″ = Maximum d permitted by Specification if any intermediate stiffeners are required by the Unit Shear.

Fig. 116. Stiffener Spacing Chart

Courtesy of American Institute of Steel Construction

$$\frac{8000}{\sqrt{v}}=117 \qquad \sqrt{v}=\frac{8000}{117}=68.3$$

$$v=4665$$

When the unit shear is less than 4665 the maximum spacing of 84 inches can be used. See Section 19(e) of the specification.

$$\text{Unit shear at end of girder}=\frac{135{,}000}{48\times\frac{5}{16}}=9000 \text{ lb. per sq. in.}$$

The required distances between stiffeners can then be computed from the formula

$$d=\frac{270{,}000\times t}{v}\sqrt[3]{\frac{vt}{h}}$$

The solution of this equation for d can be facilitated by the use of the chart in Fig. 116. Enter the chart at the right with $t=\frac{5}{16}$. Proceed horizontally left to intersection with Clear Depth, $h=36\frac{1}{2}$ inches. Position is estimated between full-line curves marked "30" and "40." Note that this intersection lies between Unit-Shear Curves (dotted) marked "3000" and "5000," or about 4700. From the same intersection proceed vertically downward to the Unit Shear marked "9000." The Spacing, d, shown at the left of the chart reads "40 inches."

The unit shear is then computed or scaled from a shear diagram 40 inches out from the end of the girder. In this case the unit shear is 7750 lbs. per sq. in. Proceeding down vertically in the chart to the intersection with the Shear "7750." The Spacing, d for this shear is 44 inches. The unit shear 84 inches out from the end is 6200 lbs. per sq. in. and the spacing for this shear is 51 inches as read from the chart. The unit shear 135 inches from the end is 4400 lbs. and as this is less than 4665 (read "4700" on the chart) the maximum spacing of 84 inches may be used.

Plate girder stiffeners should be in pairs, one on each side of the web, and should be connected to the web by rivets spaced not more than 8 times their nominal diameter. Intermediate stiffeners may be crimped. Fig. 117 shows some crimped stiffeners.

Rivets Connecting Flange Angles to Web. In order to make the several pieces of a plate girder act as a unit, they must be rigidly fastened together by riveting or welding. It is evident that if the angles and plates were not effectively held together, the parts would tend to act independently.

The formulas for the rivet pitch (p) of the rivets connecting the flange angles to the web are:

Using gross I and no loads on top flange,

$$p=\frac{RI}{VA\frac{d}{2}}$$

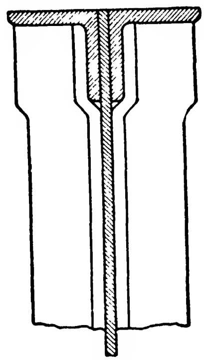


Fig. 117 Crimped Stiffeners

With gross I and loads on top flange,

$$p=\frac{R}{\sqrt{\left(\frac{VA\frac{d}{2}}{I}\right)^2+w^2}}$$

where R = the allowable resistance of one rivet; V = vertical shear at section; A = gross area of one flange; d = distance between centers of gravity of the flanges; I = gross moment of inertia; w = vertical load on flange per linear inch.

For a uniform load, each rivet space would be different because the shear varies from a maximum at the end to zero at the center. Of course it is not practical to space the rivets strictly in accordance with 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.

To illustrate, the rivet pitch will be computed for the girder in Case A, Fig. 113. The following data applies. $V=135{,}000$ lbs. Load $=6000$ lbs. per linear foot. $A=19.46$. $d=44.86$. $I=22{,}590$. Web, $48''\times\frac{5}{16}''$. Rivets, $\frac{3}{4}''\phi$.

The allowable resistance of one rivet will be governed by bearing on the web and will be 9380 lbs.

Assuming no loads on top flange.

$$p=\frac{9380\times 22{,}590}{135{,}000\times 19.46\times 22.43}=3.6\text{ in.}$$

If the load is applied directly to the top flange,

$$w=\frac{6000}{12}=500\#\text{ per linear inch}$$

$$p=\frac{9380}{\sqrt{\left(\frac{135{,}000\times 19.46\times 22.43}{22{,}590}\right)^2+(500)^2}}$$

$$p=\frac{9380}{\sqrt{(2600)^2+(500)^2}}$$

$$p=\frac{9380}{2650}=3.5\text{ in.}$$

It is evident that unless a heavy, concentrated load is placed on the top flange, the rivet spacing does not differ appreciably from that where none is considered.

In figuring the rivet spacing between the various sections or stiffener divisions, the maximum vertical shear in that panel is used.

Problem. Compute the rivet pitch both with and without load on top flange for the section selected in Case B, Fig. 114.

Rivets Connecting Cover Plates. Where cover plates are used, there must be determined the necessary riveting for attaching them to the flange angles. The procedure is similar to that just given

$$p=\frac{nIR}{VA_c\frac{dc}{2}}$$

where $n=$number of rivets on one transverse line. $Ac=$total gross area of all cover plates in one flange. $dc=$distance between centroids of all cover plates in tension flange and all cover plates in compression flange. Remainder of nomenclature or meanings for the letters, same as in preceding problem.

To illustrate the rivet pitch in the cover plates at the end of the girder, shown in Fig. 114, Case B will be computed.

$$p=\frac{2\times 22{,}883\times 6630}{135{,}000\times 6.12\times 24.47}=15 \text{ in.}$$

The rivet value used is that for single shear, as the bearing value on $\frac{7}{16}$-inch plate was larger. Due to the thin cover plate used, the resulting rivet spacing is large. When heavier cover plates and more than one are used, the allowed rivet spacing is small at the ends of the girder and increases towards the center.

Specification requires that the maximum spacing be not more than 16 times the thickness of the thinnest plate, or a maximum of 12 inches. This rule makes it necessary to use 7-inch maximum spacing, and, at the ends, 3 inches for a distance of $1\frac{1}{2}\times 14$ or 21 inches. See Section 18 A.I.S.C. Specifications.

Problem. Compute the rivet pitch in the cover plates for the following girder.

$V=500{,}000$ lb. $n=2$. $\frac{dc}{2}=32$ in. $I=95{,}000$. $R=11{,}780$ lb. $A=58$ sq. in.

Welded Plate Girders. Welded girders shown in Fig. 110 at *f* and *g* are designed in a manner similar to riveted girders. The moment of inertia is computed for the component parts, the same as for a riveted girder. The welding required between the flange and web is also figured in a similar manner.

v = unit horizontal shear per inch

$$\text{Then } v=\frac{VA\frac{d}{2}}{I}$$

After v is determined, the size and length of weld, if staggered, can be selected.

Fig. 118 shows the design drawing of a plate girder over the auditorium in the 13-story A. T. & S. F. Ry. Office Building in Amarillo, Texas.

Problem. Compute the resisting moment for the girder shown in Fig. 118, using A. I. S. C. Specifications.

OTHER FORMS OF BUILT-UP 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.

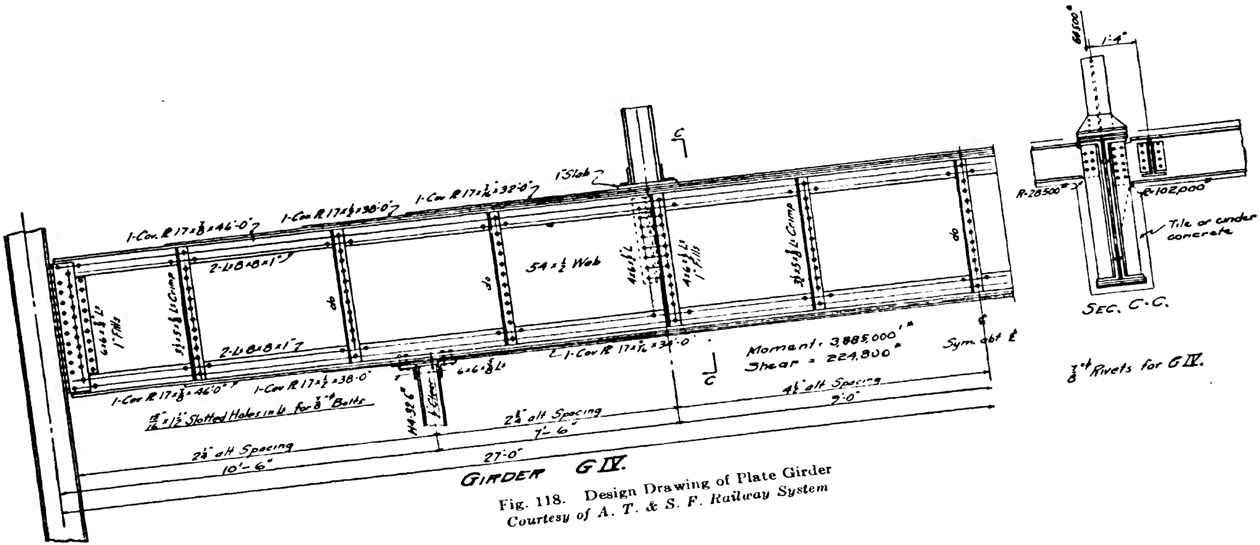


Fig. 118. Design Drawing of Plate Girder
Courtesy of A. T. & S. F. Railway System

Beams with Flange Plates. A form of girder, Fig. 119, is used when shallow girders are required and the 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 add to this value for the beam, the value of I for the section of the flange plates.

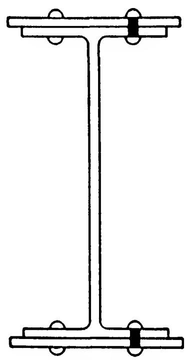


Fig. 119.
Shallow Girder

For example, compute the moment of inertia for 16 WF 114# and 2 Pl. $12'' \times \frac{1}{2}''$.

$$\begin{array}{ll} \text{16 WF 114\#} & =1642.6 \\ \text{2 Pl. } 12'' \times \frac{1}{2}'' & \\ 2 \times 6 \times 8.57 \times 8.57 = & \underline{881.3} \\ \text{Total value of } I & =2523.9 \end{array}$$

Check should be made to ascertain if it is necessary to reduce effective gross area due to rivet holes. Rivets, $\frac{3}{4}''\phi$.

$$\text{Percentage of rivet holes} = \frac{1 \times 1.535 \times \frac{7}{8}}{12+6} \times 100 = 7.5 < 15\%$$

As the rivet holes are less than 15% of the flange areas, no reduction is necessary. If the rivets in the flange were not staggered, two holes out would have to be considered.

Problem. Compute the I and the resisting moment of an 18″ WF 124# with a $12'' \times \frac{3}{4}''$ plate riveted to the top and bottom flanges. Check for percentage of rivet holes.

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. 44, and then the moment of inertia about the neutral axis (through the center of gravity), p. 45.

The common examples of unsymmetrical sections are crane girders, beam lintels with one flange plate, and girders requiring extra lateral stiffness on account of unsupported top flange.

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 a beam. With the exception noted above, viz, locating the neutral axis, the procedure in designing is the same as for symmetrical girders.

Problem. A lintel is made of a 12″ **I** $31\frac{1}{2}$# and a plate on the top flange 12″$\times\frac{5}{16}$″. What is the moment of inertia of the section?

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 Fig. 118. Usually 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. 120 shows the shop detail drawing of the plate girder of which the design drawing is shown in Fig. 118. Sometimes the vertical shear is so large that a single web plate would have greater thickness than is desirable. This leads to the adoption of a double web box girder.

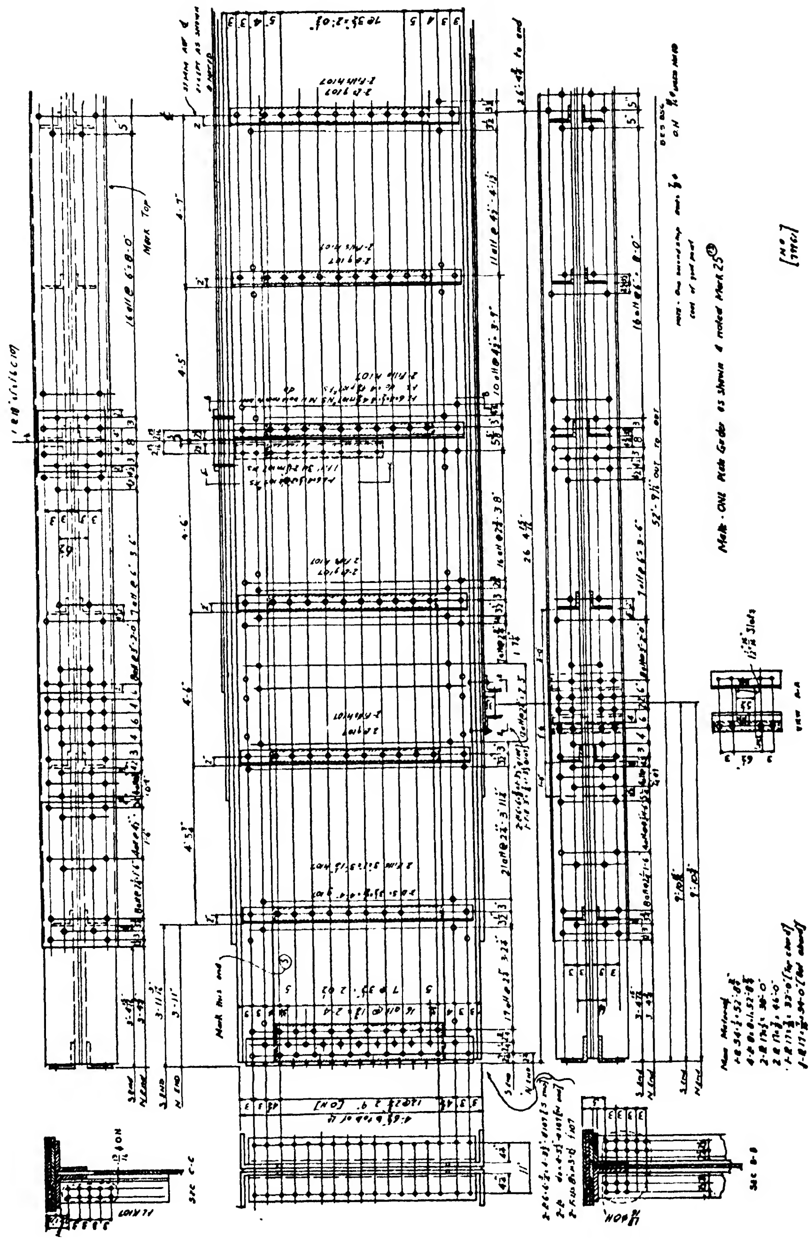


Fig. 120. Shop Detail Drawing of Girder Shown in Fig. 118.
Courtesy of A. T. & S. F. Railway System

DETAILS OF CONSTRUCTION

End Bearings. When the end of a girder bears on masonry, Fig. 121, the bearing plate is designed in the same manner as for beams. With riveted girders it is sometimes necessary to replace the plain bearing plate by I beams to spread the bearing along walls.

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 take two

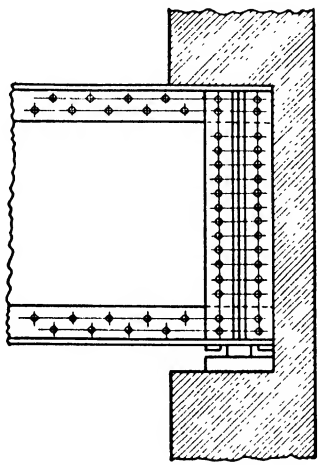


Fig. 121. Plate Girder Bearing on Masonry

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. 122. The end angles must be set accurately to the correct length and at right angles to the axis of the girder.

Bracket Connection. The bracket connection, Fig. 123, 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.

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. A girder of any length likely to occur in building construction can be shipped by rail, so that the matter involves only the com-

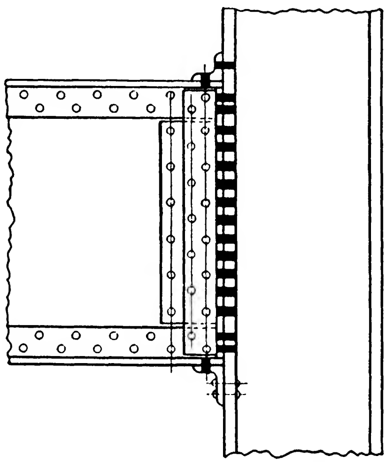


Fig. 122. Web Connection of Girder to Column

parison 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 long girders 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. 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

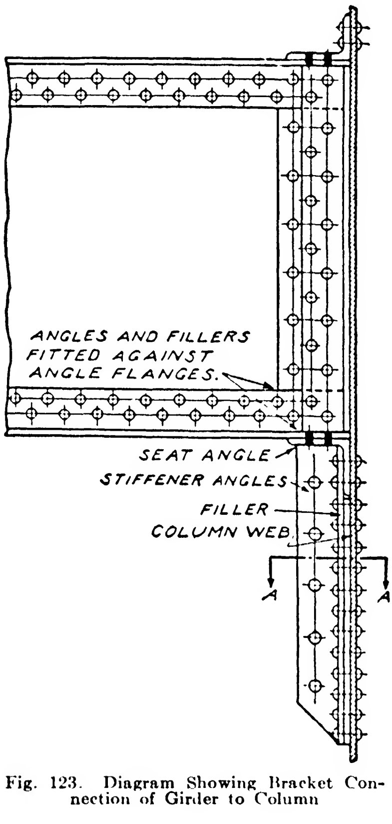


Fig. 123. Diagram Showing Bracket Connection of Girder to Column

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.

Fig. 124 at *a*, *b*, *c* and *d* show, respectively, splices in a flange plate, in flange angles, and in web plate.

Splice for Flange Plate. Fig. 124 at *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 $6.125 \times 20{,}000$ or 122,500 pounds. The splice rivets are in single shear, hence

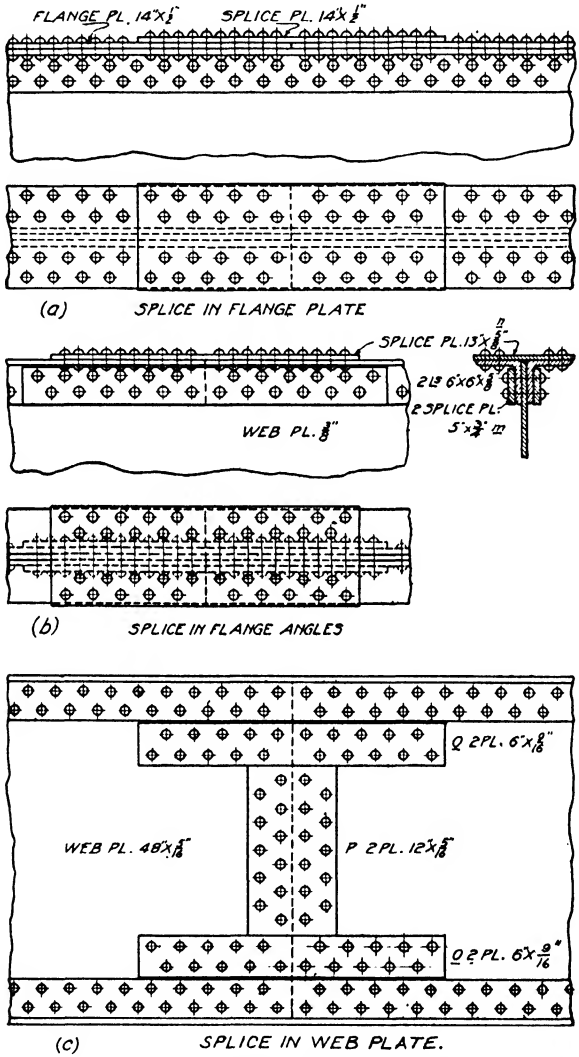


Fig. 124 (*a to c*). Diagrams of Splices in the Members of a Plate Girder Section

the number required on each side of the joint is $\frac{122{,}500}{6630}$ or 19. Use 20 rivets.

Splice for Flange Angles. Fig. 124 at *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.

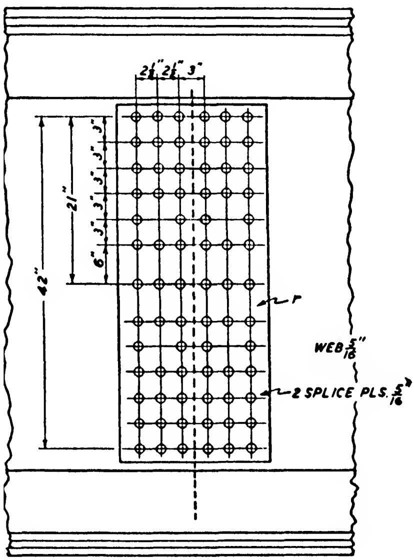


Fig. 124(*d*). Splice in Web Plate

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. 124 at *c* and Fig. 124 (*d*). 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. 124 at a; by splice plates on the angles, as plates m in Fig. 124 at b; by splice plates o in Fig. 124 at c; by splice plates r in Fig. 124 (d); 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. 124 at b, may be used to advantage for all or part of the web splice.

Taking for example the girder in Fig. 114, the web plate is $48'' \times \frac{5}{16}''$. Its net moment of inertia is 2620, p. 166. Two flange plates $14'' \times \frac{1}{4}''$, after deducting 2 rivet holes from each, have a net value of I

$$I = 2 \times 12\tfrac{1}{4} \times \tfrac{1}{4} \times 24.82 \times 24.82 = 3780$$

which is more than required.

Taking the girder shown in Fig. 113, 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. 124 at 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. 124 at c, are found to be $6'' \times \frac{9}{16}''$. The strength of the splice plate must be developed by rivet bearing in the web plate requiring 10 on each side of the joint.

Resistance to Shear. For resisting shear, the splice plates are in the form of the plates p, Fig. 124 at 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.

A common type of web splice is the one shown in Fig. 124 (*d*). The rivets on each side of the splice must be figured for the following forces.

1. The bending moment carried by the web at the section
2. The vertical shear at the section
3. The moment due to transferring the shear from the center line of rivet group to center line of splice

To illustrate, the splice shown in Fig. 124 (*d*) will be computed. Rivets, $\frac{3}{4}''$ in diameter. $V=75{,}000$ lbs. Moment carried by the web at the point of splice $=2{,}850{,}000$ inch-lbs. In figuring the moment of inertia and the section modulus of the rivet group the vertical distances to the rivets are used. For average size girders the results differ only very slightly from the polar moment of inertia and section modulus.

Moment of Inertia

$$\begin{aligned} 3\times6^2 &= 108 \\ 2\times9^2 &= 162 \\ 3\times12^2 &= 432 \\ 3\times15^2 &= 675 \\ 3\times18^2 &= 972 \\ 3\times21^2 &= \underline{1323} \\ \text{Total } I &= 3672\times2=7344 \end{aligned}$$

$$\text{Section modulus}=\frac{I}{h}=\frac{7344}{21}=350$$

$$\text{Rivet stress due to bending}=\frac{2{,}850{,}000}{350}=8140 \text{ lb.}$$

$$\text{Rivet stress due to shear}=\frac{75{,}000}{37}=2030 \text{ lb.}$$

Rivet stress due to moment caused by eccentricity of shear

$$=\frac{75{,}000\times(2\tfrac{1}{2}+1\tfrac{1}{2})}{350}=860 \text{ lb.}$$

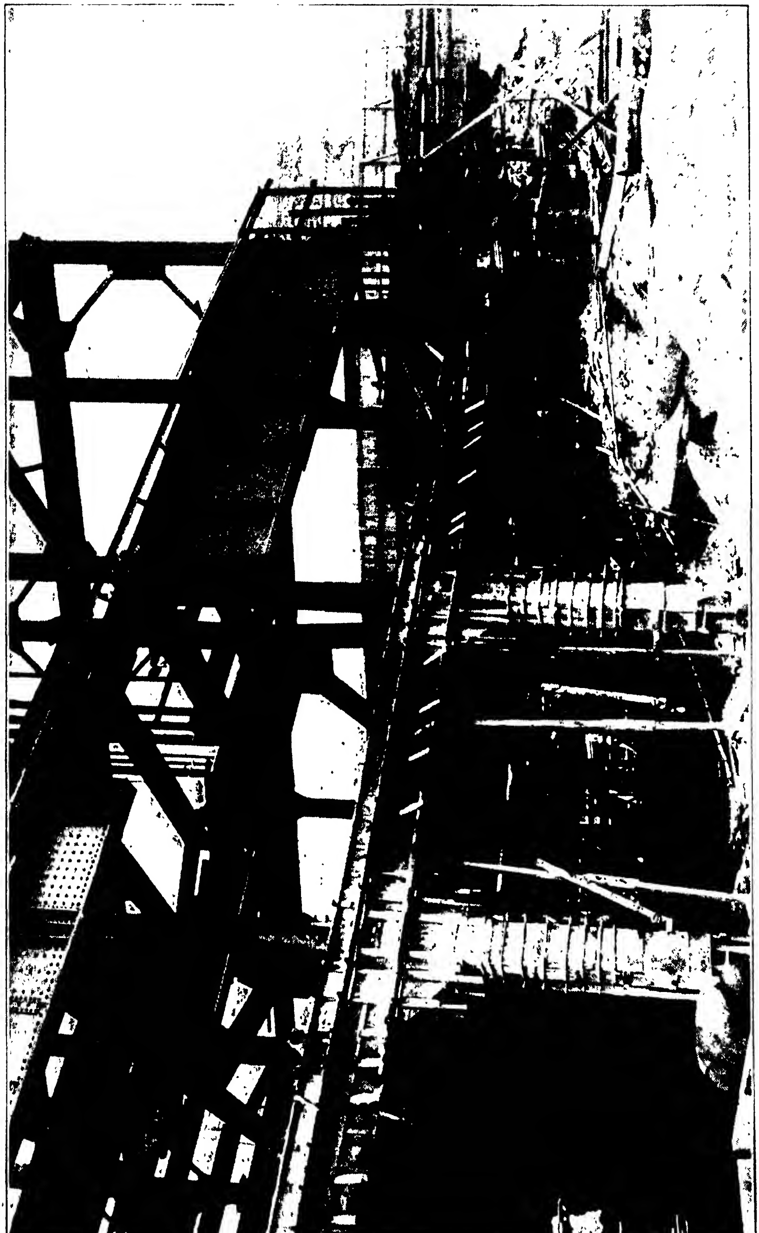
$$\text{Resultant stress on rivets}=\sqrt{2030^2+9000^2}=9200 \text{ lb.}$$

The allowed stress on rivets equals 9380 lbs., which is governed by bearing on the $\frac{5}{16}$-inch web.

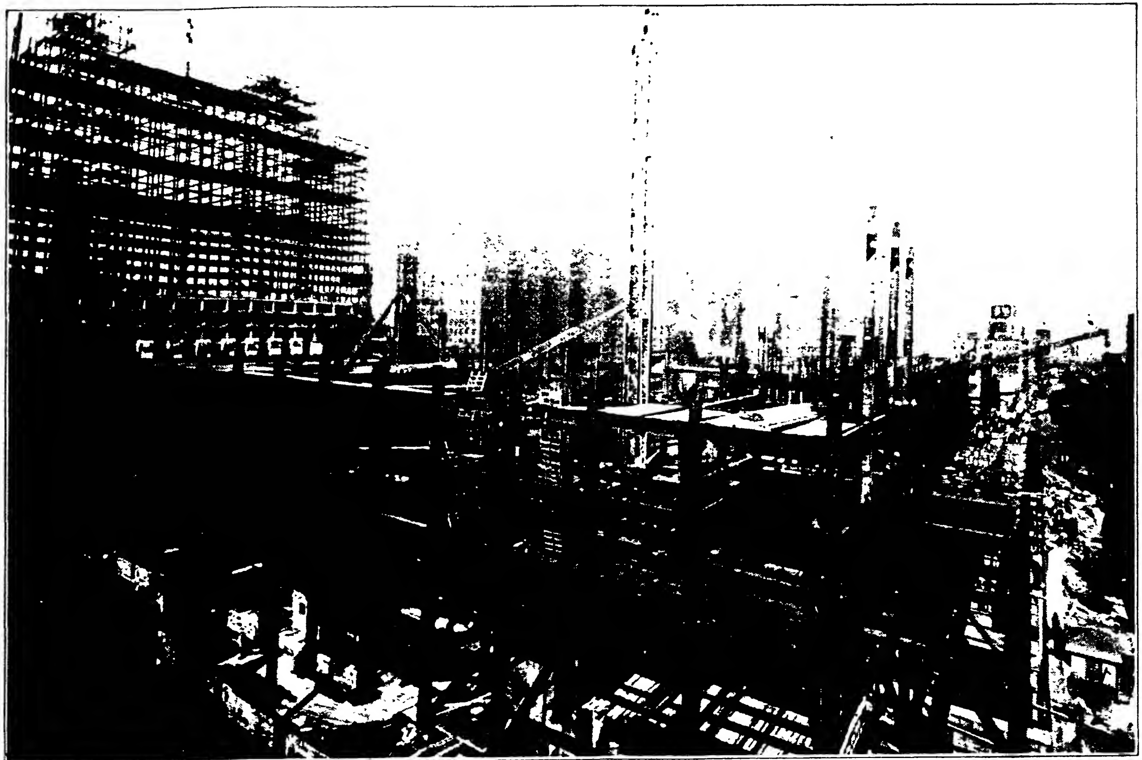
The moment of inertia of the splice plates should always equal or exceed the moment of inertia of the web.

Problem. Compute the rivets required in a splice similar to that shown in Fig. 124(d). The depth of plates equals 48″. Moment carried by splice plates, 3,000,000 ″ #. $V=100{,}000$#. Web is $\frac{5}{16}$″ thick. Rivets, $\frac{3}{4}$″ diameter.

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.



MEDICAL ARTS GARAGE AND TRANSPORTATION BUILDING OF CLEVELAND UNDER CONSTRUCTION
Note Cantilever Girders and Offset Columns
Courtesy of Graham, Anderson, Probst and White, Architects



THE 20 WACKER DRIVE BUILDING OF CHICAGO DURING CONSTRUCTION

Note Lengths and Splices of Column Sections

Courtesy of Graham, Anderson, Probst and White, Architects

CHAPTER IX

COLUMNS

Definitions. A column or strut is a member subjected to compression in the direction of its longitudinal axis; in other words, subjected to axial compression. The term "column" is usually applied to a vertical member subjected directly to a gravity load. Smaller compression members and those not in a 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 in a multistory building 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. 125 at *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. 125 at *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.

Concentric Loads. Concentric loads, Fig. 126, 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.

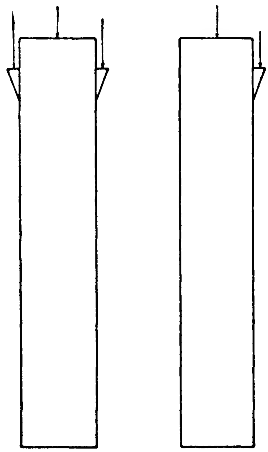


Fig. 125. Diagram Showing Examples of (a) Concentric Loads and (b) Eccentric Loads

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:

$$W = pA$$

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. 127, 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. 103.

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 maxi-

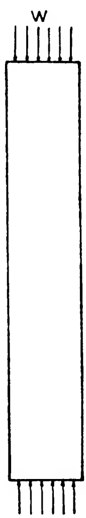


Fig. 126. Diagram of Stresses from Concentric Loads

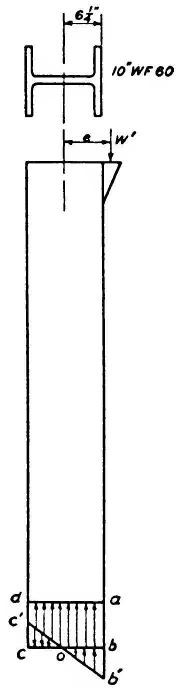


Fig. 127. Diagram of Stresses from Eccentric Loads in Compression

mum compression fiber stress per square inch is the maximum combined stress resulting from the eccentric load. This is illustrated in Fig. 127. 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.

The total stress produced by all the loads equals the sum of the stresses produced by the loads separately. This is not exactly correct, but represents usual practice. Due to the continuity of the columns, 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. 126 and 127, letting them represent the same column. Assume W a concentric load of 100,000 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 a 10″ WF 60# section from which c, the distance from the neutral axis to the extreme fiber, is 5.12 inches; r, the radius of gyration about the same axis as the bending moment, is 4.41 inches; $\frac{I}{c}$, the section modulus, is 67.1 inches^3; and A, the area, is 17.66 square inches. The average stress resulting from the total load is

$$\frac{150{,}000}{17.66} = 8490\# \text{ per sq. in.}$$

This is represented by $a\,b$ in Fig. 127.

The maximum fiber stress resulting from the bending moment, taking three-fourths of the computed moment, is

$$\frac{\frac{3}{4} \times 500{,}000}{67.1} = 5580\# \text{ per sq. in.}$$

This is represented in Fig. 127 by $b\,b'$ and $c\,c'$ in compression and tension, respectively.

Then the total maximum fiber stress in the column is

$$8490+5580=14{,}070\# \text{ per sq. in.}$$

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.

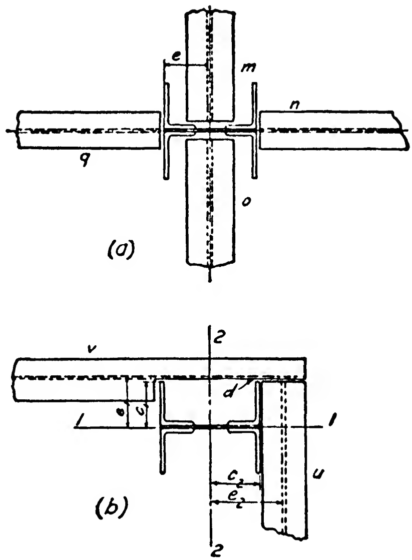


Fig. 128. Diagrams Showing (a) Concentric Load and (b) Eccentric Loads on Columns

(b) Fig. 128 illustrates cases of concentric and eccentric loading. In each of them there may be a concentric load from the column section above. In Fig. 128 at *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, *q* becomes eccentric with a moment arm *e* and a bending moment $q \times e$. If *n* is less than *q*, the difference is the eccentric load and the bending moment is $(q-n) \times e$. In Fig. 128 at *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.

As an example, take Fig. 128 at 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.

$$\begin{array}{lr} \text{Concentric load from column section above} & =150{,}000 \\ v & = 30{,}000 \\ u & = \underline{45{,}000} \\ \text{Total load} & =225{,}000\# \end{array}$$

$$\text{Unit stress from total load} = \frac{225{,}000}{29.7} = 7575\# \text{ per sq. in.}$$

$$M_1 = 30{,}000 \times 7 = 210{,}000 \text{ in.-lb.}$$

$$\tfrac{3}{4} \text{ of this} = 157{,}500 \text{ in.-lb.}$$

$$\text{Unit fiber stress due to } v = \frac{157{,}500 \times 6\frac{3}{8}}{213} = 4720\# \text{ per sq. in.}$$

$$M_2 = 45{,}000 \times 9\tfrac{1}{4} = 416{,}000 \text{ in.-lb.}$$

$$\tfrac{3}{4} \text{ of this} = 312{,}000 \text{ in.-lb.}$$

$$\text{Unit fiber stress due to } u = \frac{312{,}000 \times 6\frac{1}{8}}{721} = 2650\# \text{ per sq. in.}$$

Total fiber stress at $d = 7575 + 4720 + 2650 = 14{,}945\#$ per sq. in.

Problem. Assume a 12″ WF 85# 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

$$W_e' = W' \frac{ec}{r^2}$$

In this formula 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.

Referring now to the example illustrated in Figs. 126 and 127 and explained on p. 190, the eccentric effect is

$$W_e' = \tfrac{3}{4} \times 50{,}000 \times \frac{10 \times 5.12}{4.41 \times 4.41} = 98{,}600\#$$

Then the total equivalent concentric load on the column is

$$\begin{aligned} W &= 100{,}000 \\ W' &= 50{,}000 \\ W_e' &= \underline{98{,}600} \\ & \ 248{,}600\# \end{aligned}$$

The above method of computing eccentric loads in terms of concentric loads should not be used for final design, but as a convenient method for selection of a trial section from the load tables.

In the column load tables given in the steel handbooks, the bending factors $\left(\frac{c}{r^2}\right)$ are usually tabulated at the bottom of the page.

Problem. Compute the equivalent concentric load for the eccentric loads u and v in Fig. 128 at b, from the data given on page 192.

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 cooperate 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.

Formulas for Allowed Unit Stress. The A.I.S.C. Specifications allow the following unit stresses in columns axially loaded.

For columns with values of $\frac{l}{r}$ not greater than 120,

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

For columns with values of $\frac{l}{r}$ greater than 120,

$$p=\frac{18{,}000}{1+\dfrac{l^2}{18{,}000r^2}}$$

For convenience in using these formulas, the allowable unit stresses for various ratios of $\frac{l}{r}$ are given in Table VI.

For columns subjected to both direct loads and eccentric loads causing bending, the A.I.S.C. Specifications require that the quantity $\frac{f_a}{F_a}+\frac{f_b}{F_b}$ shall not exceed unity.

In the preceding formulas, l = the unsupported length of column in inches; r = the least radius of gyration in inches; F_a = axial unit stress that would be permitted by this specification if axial stress alone existed; F_b = bending unit stress that would be permitted by this specification if bending stress alone existed; f_a = axial unit stress (actual) = axial stress divided by area of member; f_b = bending unit stress (actual) = bending moment divided by section modulus of member.

Illustrative Examples. 1. Assume that a column with an unbraced length of 12 feet carries a concentric load of 500,000 lb.

Try a 14″ WF 111#.

$$l=12\times12=144''$$

$$\text{(least) } r=3.73$$

$$\frac{l}{r}=\frac{144}{3.73}=38.6$$

$$A=32.65 \text{ sq. in.}$$

$$p=17{,}000-0.485\times(38.6)^2$$

$$p=16{,}280 \text{ lb. per sq. in.}$$

It is not necessary to solve this formula for the allowed unit stress, as it can be picked from the table when the $\frac{l}{r}$ is known.

TABLE VI

ALLOWABLE STRESSES PER SQUARE INCH
FOR
COMPRESSION MEMBERS

Main Members, l/r up to 120·
$f = 17000 - 0.485 \frac{l^2}{r^2}$

l = Unbraced length in inches
r = radius of gyration in inches

Secondary Members, l/r 120 to 200
$f = \frac{18000}{1 + \frac{l^2}{18000r^2}}$

$\frac{l}{r}$	Unit Stress f (Kips)	$\frac{l}{r}$	Unit Stress f (Kips)	$\frac{l}{r}$	Unit Stress f (Kips)	$\frac{l}{r}$	Unit Stress f (Kips)	$\frac{l}{r}$	Unit Stress f (Kips)
1	17.00	41	16.19	81	13.82	121	9.93	161	7.38
2	17.00	42	16.14	82	13.74	122	9.85	162	7.32
3	17.00	43	16.10	83	13.66	123	9.78	163	7.27
4	16.99	44	16.06	84	13.58	124	9.71	164	7.22
5	16.99	45	16.02	85	13.50	125	9.64	165	7.16
6	16.98	46	15.97	86	13.41	126	9.56	166	7.11
7	16.98	47	15.93	87	13.33	127	9.49	167	7.06
8	16.97	48	15.88	88	13.24	128	9.42	168	7.01
9	16.96	49	15.84	89	13.16	129	9.35	169	6.96
10	16.95	50	15.79	90	13.07	130	9.28	170	6.91
11	16.94	51	15.74	91	12.98	131	9.22	171	6.86
12	16.93	52	15.69	92	12.90	132	9.15	172	6.81
13	16.92	53	15.64	93	12.81	133	9.08	173	6.76
14	16.91	54	15.59	94	12.72	134	9.01	174	6.71
15	16.89	55	15.53	95	12.62	135	8.94	175	6.66
16	16.88	56	15.48	96	12.53	136	8.88	176	6.62
17	16.86	57	15.42	97	12.48	137	8.81	177	6.57
18	16.84	58	15.37	98	12.34	138	8.75	178	6.52
19	16.83	59	15.31	99	12.25	139	8.68	179	6.48
20	16.81	60	15.25	100	12.15	140	8.62	180	6.43
21	16.79	61	15.20	101	12.05	141	8.55	181	6.38
22	16.77	62	15.14	102	11.95	142	8.49	182	6.34
23	16.74	63	15.08	103	11.86	143	8.43	183	6.29
24	16.72	64	15.01	104	11.75	144	8.36	184	6.25
25	16.70	65	14.95	105	11.65	145	8.30	185	6.20
26	16.67	66	14.89	106	11.55	146	8.24	186	6.16
27	16.65	67	14.82	107	11.45	147	8.18	187	6.12
28	16.62	68	14.76	108	11.34	148	8.12	188	6.07
29	16.59	69	14.69	109	11.24	149	8.06	189	6.03
30	16.56	70	14.62	110	11.13	150	8.00	190	5.99
31	16.53	71	14.56	111	11.02	151	7.94	191	5.95
32	16.50	72	14.49	112	10.92	152	7.88	192	5.91
33	16.47	73	14.42	113	10.81	153	7.82	193	5.86
34	16.44	74	14.34	114	10.70	154	7.77	194	5.82
35	16.41	75	14.27	115	10.59	155	7.71	195	5.78
36	16.37	76	14.20	116	10.47	156	7.65	196	5.74
37	16.34	77	14.12	117	10.36	157	7.60	197	5.70
38	16.30	78	14.05	118	10.25	158	7.54	198	5.66
39	16.26	79	13.97	119	10.13	159	7.49	199	5.62
40	16.22	80	13.90	120	10.02	160	7.43	200	5.59

Courtesy of American Institute of Steel Construction

Actual unit stress,

$$p=\frac{W}{A}$$

$$p=\frac{500,000}{32.65}=15,310 \text{ lb. per sq. in.}$$

2. Assume that a column with an unbraced length of 12 feet carries a concentric load of 265,000 lb., and an eccentric load of 100,000 lb. applied 10 inches from the major or strong axis.

A 14″ WF section will be used for trial. The average bending factor $\left(\frac{c}{r^2}\right)$ for this group is 0.185.

Bending moment $=M=100,000\times 10=1,000,000$ in.-lb. Equivalent direct load $=1,000,000\times 0.185=185,000$ lb. Approximate column load $=265,000+100,000+185,000=550,000$ lb.

A 14″ WF 111# is selected from the tables in the handbooks, giving concentric loads. See Table VII. The table shows that this section has a capacity of 531,000 lb. for a 12-foot unbraced length. It will now be necessary to investigate this selected column section as to its compliance with Section 6 (a) of the Specifications.

$$\frac{l}{r}=38.6 \qquad A=32.65 \qquad \frac{I}{c}=S=176.3$$

Allowed unit stress $=16,280$ lb. per sq. in.

$$F_a=16,280$$

$$F_b=20,000$$

$$f_a=\frac{W}{A}=\frac{365,000}{32.65}=11,200$$

$$f_b=\frac{M}{S}=\frac{1,000,000}{176.3}=5,670$$

$$\frac{f_a}{F_a}+\frac{f_b}{F_b}=\frac{11,200}{16,280}+\frac{5,670}{20,000}=.69+.28=.97$$

As the result is less than unity, the section is adequate. The next lighter section, which is a 14″ WF 103#, will be investigated.

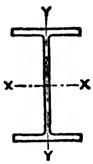
$$\frac{l}{r}=38.8 \qquad A=30.26 \qquad S=163.6$$

$$F_a=16,270 \qquad F_b=20,000$$

$$f_a=12,100 \qquad f_b=6,120$$

$$\frac{f_a}{F_a}+\frac{f_b}{F_b}=\frac{12,100}{16,270}+\frac{6,120}{20,000}=.74+.31=1.05$$

TABLE VII



COLUMNS 14

WF SECTIONS

ALLOWABLE CONCENTRIC LOADS IN KIPS

Unbraced Length Feet (Unbraced length with respect to least radius of gyration)	Nominal Depth and Width—Weight per Foot								
	14 x 14½							14 x 12	
	136	127	119	111	103	95	87	84	78
6	673	628	589	549	509	470	430	413	384
7	670	626	587	547	507	468	428	411	382
8	667	623	584	545	505	466	426	408	379
9	664	620	581	542	502	464	424	405	376
10	660	616	578	539	499	461	422	401	372
11	656	612	574	535	496	458	419	397	368
12	651	608	570	531	493	455	416	393	364
13	646	603	566	527	489	451	413	388	360
14	641	598	561	523	485	447	409	383	355
15	635	593	556	518	480	443	405	378	350
16	629	587	551	513	475	439	401	372	344
17	623	581	545	508	470	434	397	366	338
18	616	575	539	502	465	429	392	359	332
19	609	568	532	496	459	424	387	352	326
20	601	561	525	490	453	418	382	345	319
21	593	553	518	483	447	412	377	337	312
22	585	545	511	476	441	406	372	329	304
23	576	537	503	469	434	400	366	320	296
24	567	528	495	461	427	393	360	311	288
25	557	519	486	453	419	386	353	302	279
26	547	510	477	444	411	379	346	292	270
27	537	500	468	435	403	372	339	282	260
28	526	490	459	426	395	364	332	272	250
29	515	480	449	417	386	356	325	261	240
30	503	469	438	408	377	347	317	250	230
32	478	446	417	387	358	330	301	234	216
34	453	421	394	366	338	311	284	221	204
36	425	396	370	343	317	291	265	208	192
38	397	370	346	321	297	273	250	196	181
40	379	353	330	306	283	261	238	185	170

PROPERTIES

Depth	14¾	14⅝	14½	14⅜	14¼	14⅛	14	14⅛	14
Width	14¾	14¾	14⅝	14⅝	14⅝	14½	14½	12	12
Ratio r_x/r_y	1.67	1.67	1.67	1.67	1.67	1.66	1.66	2.03	2.03
Bending Factors B_x	.185	.185	.185	.185	.185	.186	.185	.189	.189
Bending Factors B_y	.519	.520	.521	.525	.525	.529	.530	.659	.665

Loads below heavy line are for l/r ratios between 120 and 200.

Courtesy of American Institute of Steel Construction

As the result is more than unity, this section is not satisfactory.

In using the formula it should be remembered that when designing long columns, which are to be subjected to bending about the major axis, and where the ratio of length to flange width $\left(\frac{l}{b}\right)$ falls between 15 and 40, the allowable bending stress F_b must be reduced according to Section 10 of the Specifications.

In cases where the bending moment on a column is considerable, the column should be turned so that bending occurs about the major axis rather than about the minor axis.

Problems. 1. Select a 10″ WF section capable of carrying a concentric load of 300,000#, the unbraced length being 12 feet.

2. Make the necessary computations for determining the most economical WF column section for carrying the following loads: 295,000# concentric load; 55,000# eccentric load applied 17 inches from the major axis.

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, 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.

The wide-flange sections provide economical shapes for columns, with a wide range of carrying capacities. The heaviest 14″ WF section has a depth of 18.69 inches and a flange width of 16.695 inches. The moment of inertia of this section is 6610.3 about the major axis and 2359.5 about the minor axis. If the rolled WF sections do not provide sufficient capacity, cover plates can be riveted or welded to the flanges. Columns can also be made up out of shapes such as shown in Fig. 129. When columns are made up out of shapes, the properties such as I, r, and c must be determined.

Radius of Gyration. The radius of gyration is computed about each axis by the method explained and illustrated on pp. 50 and 51. 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. 128 at 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 direc-

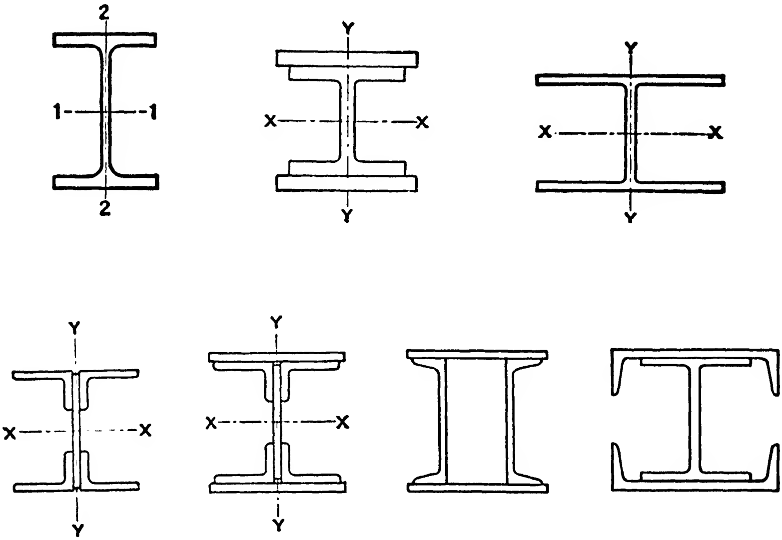


Fig. 129. Typical Column Sections

tion, Fig. 130. 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.

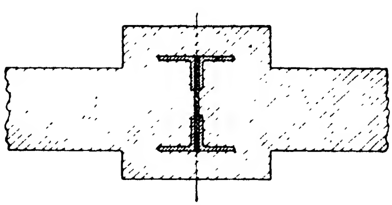


Fig. 130 Section Showing Column Supported by Masonry in Its Weaker Direction

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.

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.

The splice is placed above the floor line a sufficient distance so that the splice plates will not interfere with the beam connections and so that the riveting can be done readily; usually 3 feet 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.

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. 131 shows typical column splices. Note that the top rivets in all lower column sections are field rivets to facilitate erection. Fig. 131 at *c* shows a column splice in which the upper section is of such a small size that it would not bear directly on the bottom section. In

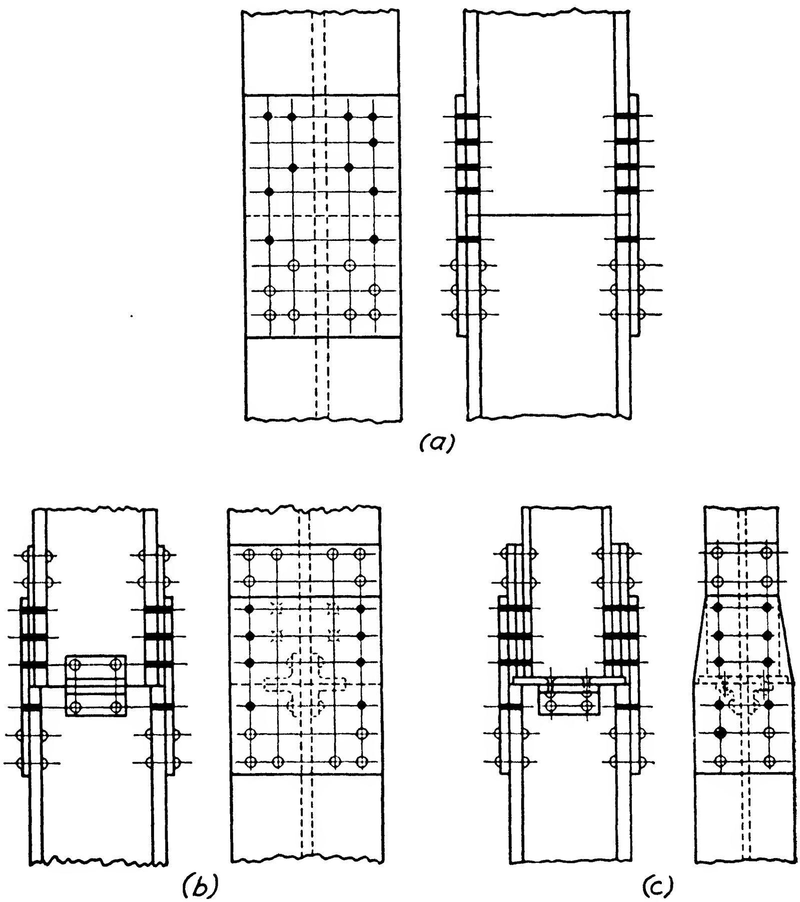


Fig. 131. Typical Column Splices

such a case a bearing plate is used. Filler plates, of course, are necessary to make up the difference in the column widths.

The thickness of the column splice plates should be made consistent with the column section; usually $\frac{3}{8}$-inch for smaller sizes and $\frac{1}{2}$-inch for the larger columns.

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

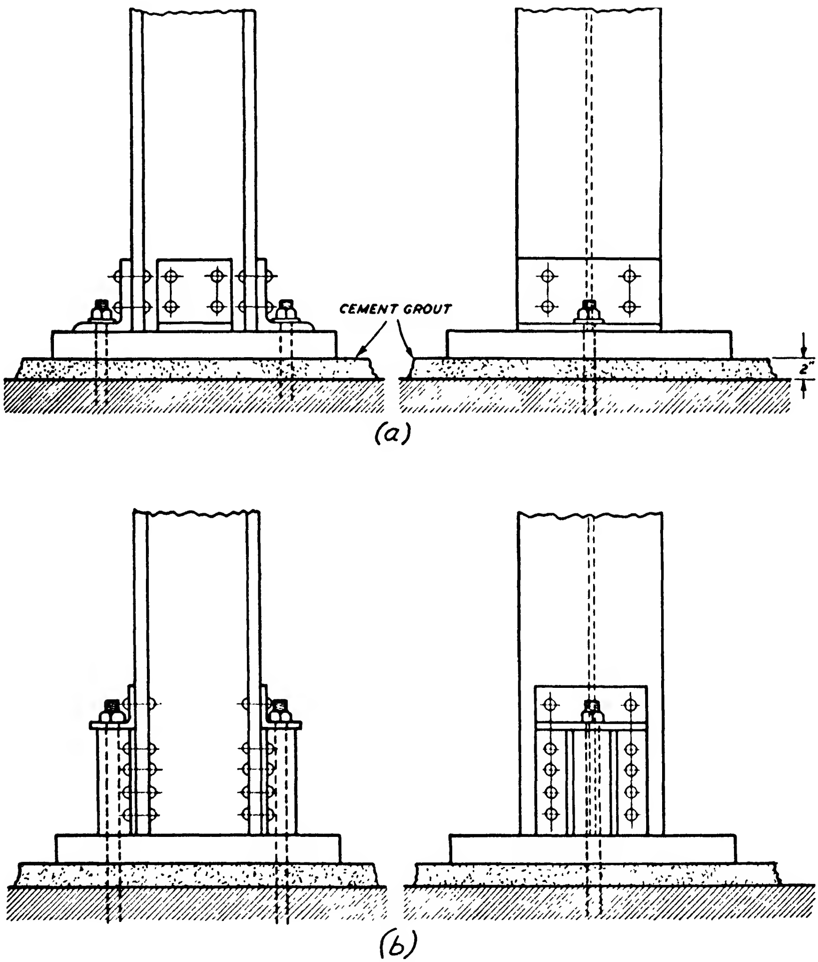


Fig. 132. Typical Column Bases

to provide a base plate to spread the load over the required area of the masonry. Steel base plates, as shown in Fig. 132, are generally used for distributing the column load over the necessary area of the concrete footing. For extremely heavy column loads, beam grillages are used for bases as shown in Fig. 133.

Cast-iron bases are used with cast-iron pipe columns. Whatever type of base is used, the bottom of the column should be milled. The portion of the base on which the column rests should also be milled. The milling of steel base plates can be omitted if the thickness is 4 inches or less, as they can be flattened by pressing.

For column bases that do not take moment or uplift, two bolts are usually provided. These bolts are used to attach the column to the base and their chief purpose is to hold the columns in line during erection.

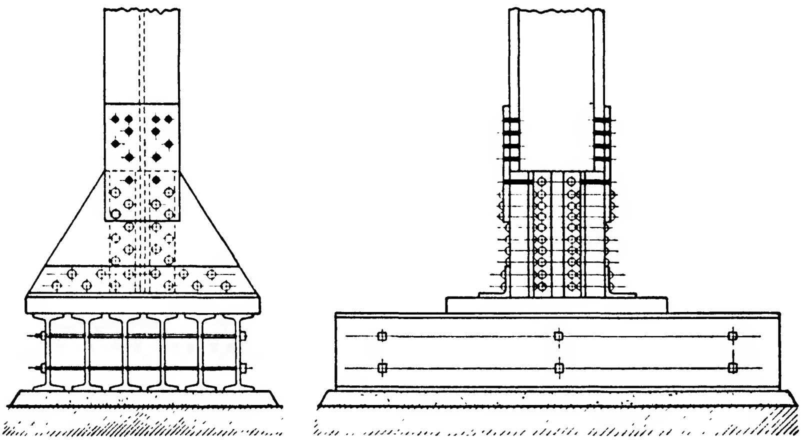


Fig. 133. Beam Grillage Column Base

Column bases are set to the correct elevation and alignment before the columns are erected. Large bases are leveled up by leveling bolts as shown in Fig. 134. The space under them is then filled with cement grout. For smaller bases, wedges are used under the edges to support and level them or they are leveled by hand on top of a layer of grout. Standard sizes of base plates are given in the handbooks; and thicknesses up to eight inches can be secured.

Design. The thickness of a base slab such as shown in Fig. 132 at a is calculated by using equations for beam design. The most common method used is the one proposed by the A.I.S.C.

The column load, F, is assumed to be uniformly distributed within a rectangle whose dimensions are .95 d and .80 b, shown in Fig. 135, and the base plate is assumed to have a uniform bearing pressure, p, on the foundation.

F = Total column load, in kips.
$A = B \times C$ = Area of plate, in square inches.
t = Thickness of plate, in inches.
p = Bearing pressure on foundation, in kips per square inch.

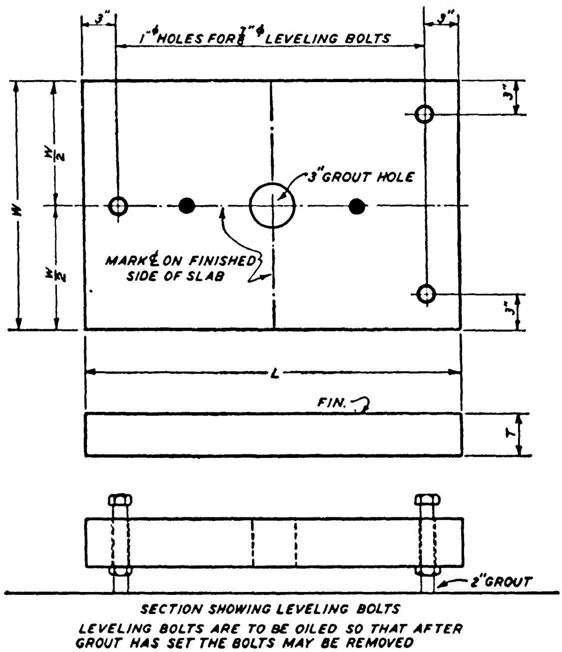


Fig. 134. Method of Leveling Heavy Base Slab

The first step is to determine the required area $A = \frac{F}{p}$. The dimensions B and C should then be picked so that the projections m and n will be approximately equal. After the dimensions m and n are determined, the thickness, t, can be found using the larger of the two m and n values and one of the following formulas.

$$t = m\sqrt{.15p} \quad \text{or} \quad t = n\sqrt{.15p}$$

Illustrative Examples. 1. Design a steel base slab for a 14″ WF 95# column carrying a load of 500,000 lbs. The allowable bearing pressure on the concrete foundation is 600 lbs. per sq. in.

$$A = \frac{500{,}000}{600} = 833 \text{ sq. in.}$$

Assume $\qquad C = 29''$

Then
$$B=\frac{833}{29}=28.7.\quad \text{Use } 29''$$
$$.80\,b=.80\times 14.545=11.6$$
$$.95\,d=.95\times 14.12=13.4$$

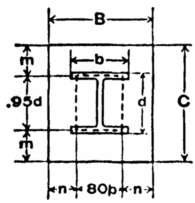


Fig. 135. Base Slab

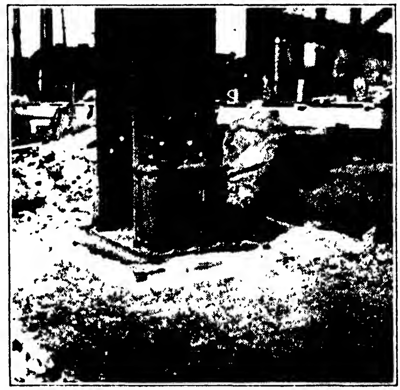


Fig. 136. Fixed Type of Column Base
Courtesy of A. T. & S. F. Railway System

$$p=\frac{500{,}000}{29\times 29}=595 \text{ lb. or } .595 \text{ kip per sq. in.}$$

$$n=\frac{29-11.6}{2}=8.7$$

$$m=\frac{29-13.4}{2}=7.8$$

$$t=n\sqrt{.15\times .595}$$
$$=8.7\times .299=2.6 \text{ in.}$$

Use 2′-5″×2$\frac{3}{4}$″×2′-5″ Base plate.

2. Design a column base similar to the one shown in Fig. 132 at *b* for a 14 WF 95# column with a concentric load of 300,000# and a moment at the base of 1,000,000 in.-lb. One condition of loading requires a base 29″×29″.

When a column has bending at the base in addition to the vertical load, the pressure under the base may be assumed to vary uniformly from a minimum value, p_1, at one end to a maximum value, p_2 at the other end. The maximum pressure, p_2, is the sum of the pressure p, due to direct load, and the extreme pressure due to bending.

As the size of base required due to vertical loads only is 29″×29″,

$$p=\frac{300{,}000}{29\times 29}=356 \text{ lb. per sq. in. due to direct load only.}$$

$$p^1=\frac{Mc}{I}=\frac{1{,}000{,}000\times 14.5}{29\times\frac{(29)^3}{12}}$$

$=245$ lb. per sq. in. due to bending moment.

$p_1=356-245=111$ lb. per sq. in.

$p_2=356+245=601$ lb. per sq. in.

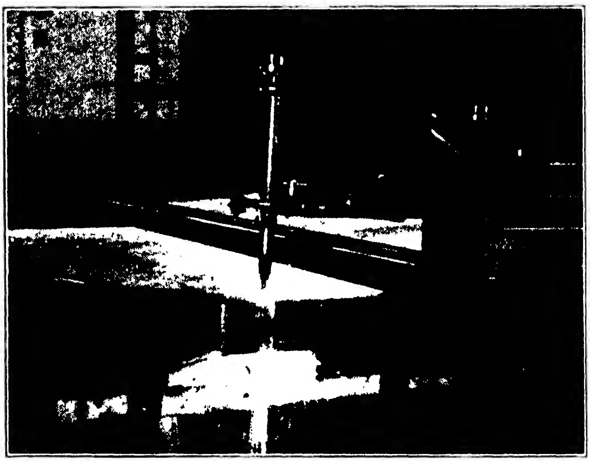


Fig. 137. Gas-cutting a Steel Slab Such as Is Used for Column Bases
Courtesy of The Linde Air Products Company

This is the maximum pressure at one end.

The method of computing the required size of anchor bolts is illustrated in the next chapter on Industrial Buildings.

In Fig. 136 is shown a column base capable of taking bending.

The heavy rolled-steel base slabs, such as those used for column bases, are readily gas-cut in the shop. Fig. 137 shows a thick slab being cut.

Typical shop details of multistory building columns are shown in Fig. 138.

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 one or two stories and in non-fireproof construction.

The allowed unit stress and column formula for cast-iron col-

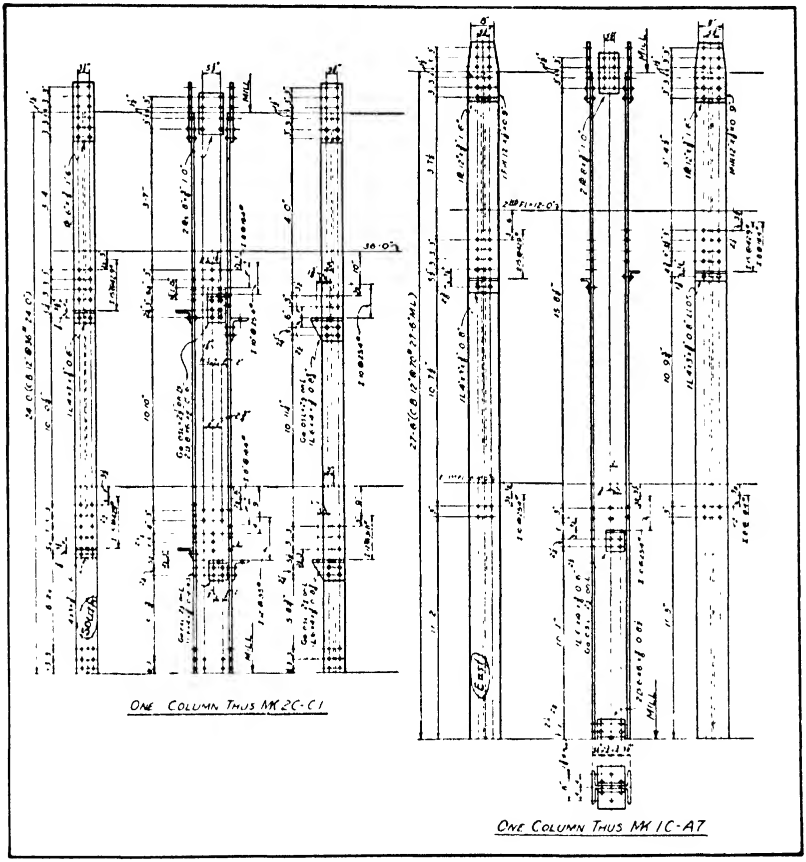
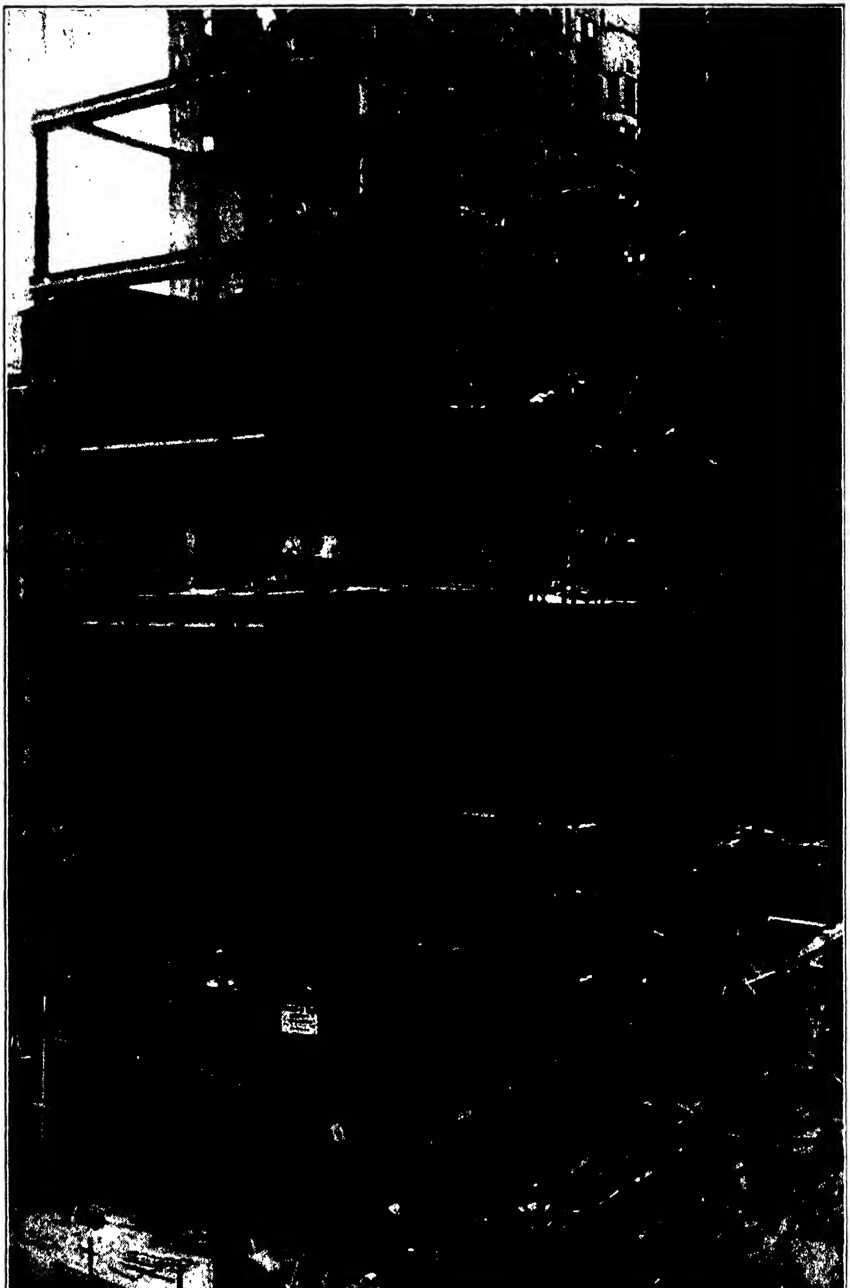


Fig. 138 Typical Column Details
Courtesy of A. T. & S. F. Railway System

umns are given in the chapter on Specifications. The various manufacturers of cast-iron columns have booklets giving the carrying capacities and showing bases, splices, and beam connections for their columns.



BENSON & RIXON BUILDING, CHICAGO
Note That First-Story Columns Are Offset to Give Unobstructed Display Windows
Courtesy of A. S. Alschuler, Architect

CHAPTER X

INDUSTRIAL BUILDINGS

General. Industrial buildings are built in both single and multiple-story heights. As the multistory building is covered in another chapter, the single story or mill building will be considered here.

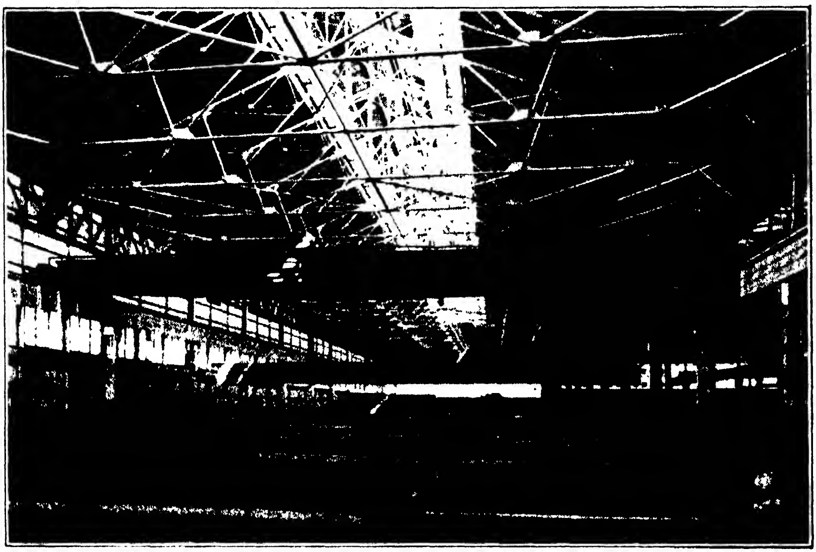


Fig. 139. Typical Mill Building with Fink Roof Trusses and Crane
Courtesy of Whiting Corporation, Harvey, Illinois

There are several types of mill buildings in common use. The type used will depend somewhat on the industrial requirements. A typical mill building with Fink roof trusses and crane is shown in Fig. 139. In Fig. 140 are shown the transverse sections of some common types of mill buildings.

LOADS

The loads to be provided for are: dead load, snow load, wind load, impact, and miscellaneous concentrated loads. The miscellaneous loads result from jib cranes, girder cranes, pipes, and so forth.

The wind and snow loads should be according to the geographical location where the building is to be built. If there is no building code to cover, the wind load should be taken as 15 pounds per vertical square foot and the snow load as 25 pounds per horizontal square foot. For all roof slopes up to about 20°, the full snow load should be used; how-

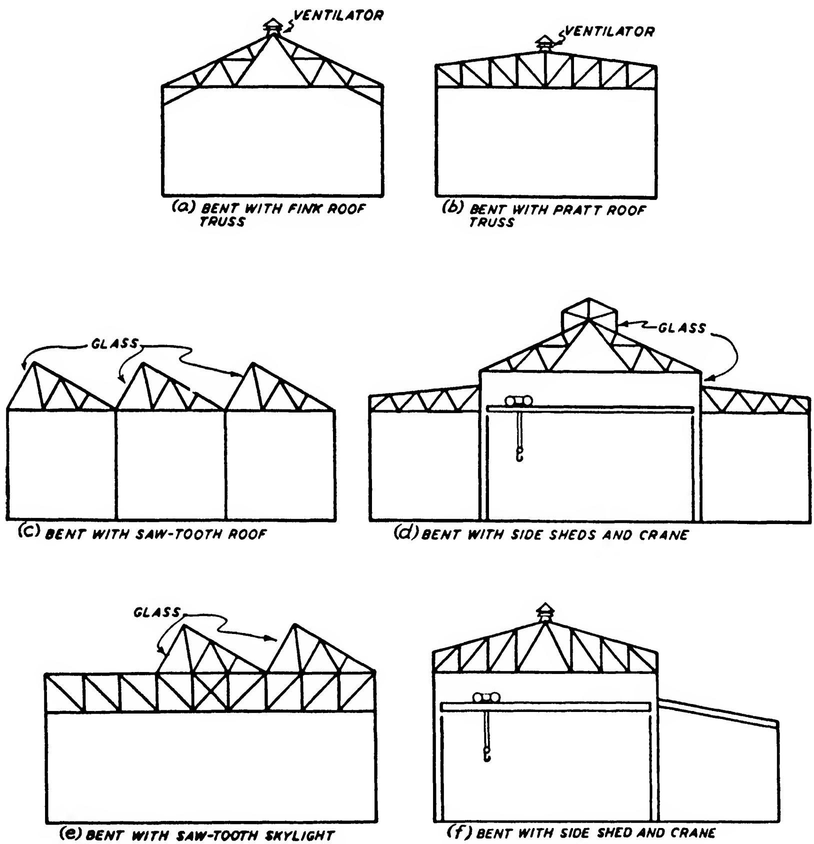


Fig. 140 Bents of Typical Mill Buildings

ever, for over 20° slopes, the load can be decreased one pound for each degree of increase in slope. For roof slopes over 45°, the snow load can be omitted.

The wind loads are taken as acting horizontally; however, on the inclined roof, only the normal component is considered. The normal pressures are computed from Duchemin's formula:

$$P = P_1 \frac{2 \sin a}{1 + \sin^2 a}$$

Where P_1 is the direct wind pressure on a vertical surface and P is the normal pressure on a unit of surface sloping at an angle, a, with the horizontal. The following table gives the normal wind pressures based on a wind pressure of 15 pounds per vertical square foot.

Normal Wind Pressure, in Pounds per Square Foot

Slope a°	Pressure per Sq. Foot Pounds	Slope a°	Pressure per Sq. Foot Pounds	Slope a°	Pressure per Sq. Foot Pounds	Slope a°	Pressure per Sq. Foot Pounds
5	2.59	20	9.18	35	12.95	50	14.48
10	5.05	25	10.75	40	13.64	55	14.70
15	7.27	30	12.00	45	14.14	60	14.84

For wind pressures other than 15 pounds per square foot, the values given will change proportionally. For roof slopes over 60 degrees, the wind pressures on vertical surfaces are used.

ROOF CONSTRUCTION

The most common types of roof construction used on mill buildings are: timber sheathing with tar and gravel roofing, corrugated asbestos or corrugated metal sheets, pressed steel interlocking plates and channel shapes, and light pre-cast slabs.

Roofs on mill buildings should be adequately anchored due to their light weight. In Fig. 141 are shown methods of fastening different types of roofs. For fastening corrugated roofing, the use of straps is preferable to clinch rivets or bent clips.

WALLS

Steel frame mill buildings may have walls of corrugated metal, corrugated asbestos, flat pressed steel panel plates, glass, brick, concrete, stone, or tile. Figs. 142 and 143 show a railroad repair shop, the side walls of which are all glass.

When a light side wall (such as one made with corrugated or pressed steel material) is used, it is supported by horizontal steel members called girts. These girts consist either of angles, channels, or small rolled beams. Girts are usually designed for the wind pressure only. The dead load of the side-wall material and girts is carried by sag rods. In Fig. 144 are shown some typical fastenings for corrugated walls.

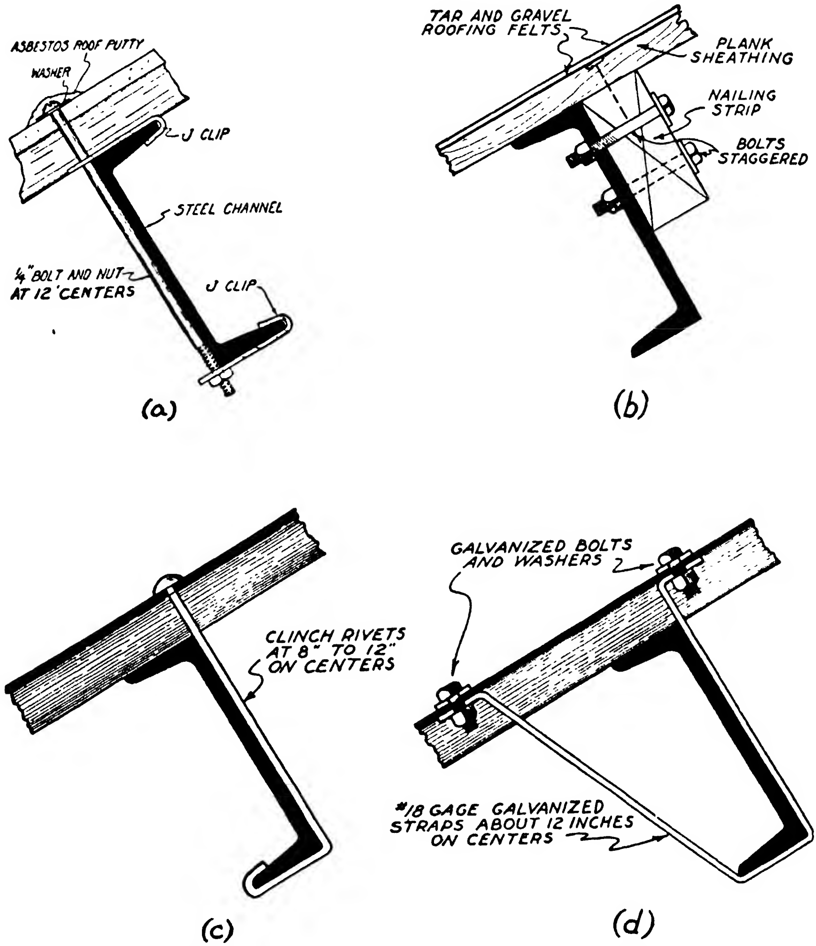


Fig. 141. Fastenings for Various Types of Roofs (*a*) For Corrugated Asbestos Roofing; (*b*) for Tar and Gravel Roof; (*c*) and (*d*) for Corrugated Metal Roofing

FLOORS

Various types of floors are in common use. Some of these are: reinforced concrete, brick, asphalt concrete, creosoted wood blocks, or the following laid on a concrete base: asphalt plank, asphalt mastic, or wood. Floors in some types of mill buildings are only dirt or cinders.

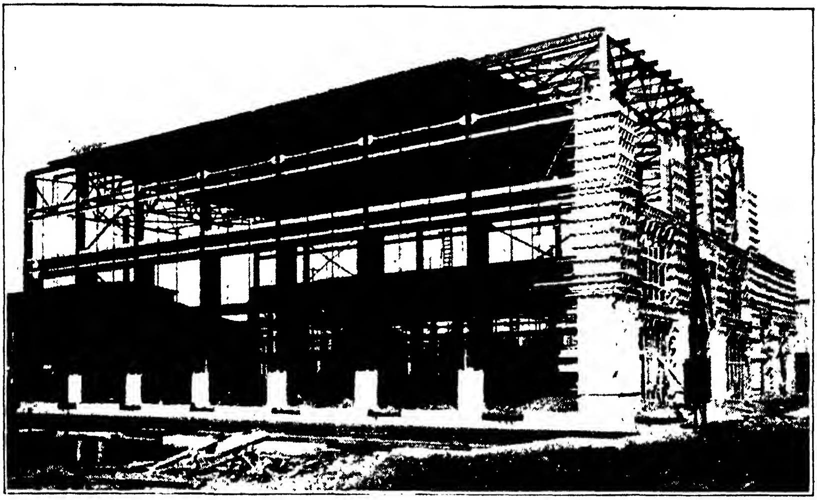


Fig. 142. Exterior View of Railroad Repair Shop at Cleburne, Texas
Courtesy of A. T. & S. F. Railway System

Fig. 143. Interior View of Railroad Repair Shop at Cleburne, Texas
Courtesy of A. T. & S. F. Railway System

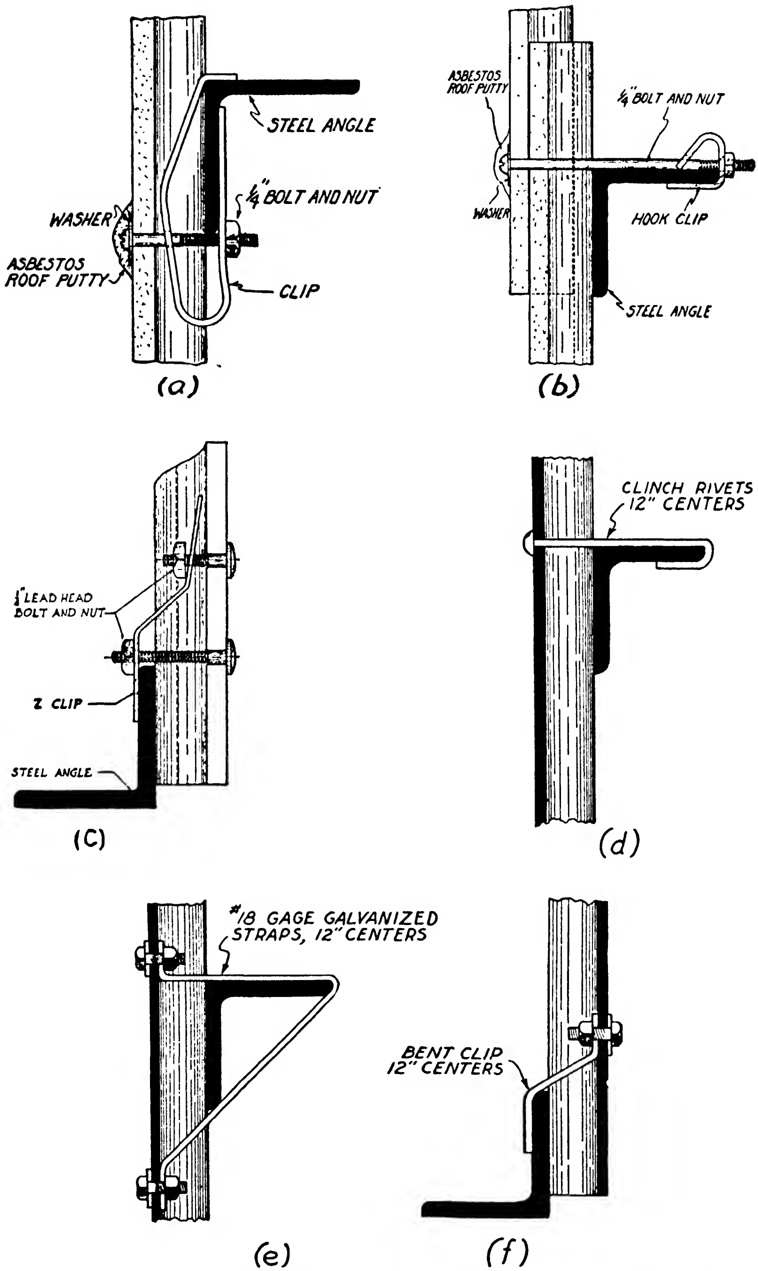


Fig. 144. Fastenings for Corrugated Walls. (*a*), (*b*) and (*c*) Are for Corrugated Asbestos Walls; (*d*), (*e*) and (*f*) Are for Corrugated Metal Walls

The type used most is undoubtedly a reinforced concrete slab poured on the natural ground or sand fill. In a floor slab like this, a reinforcement consisting of rods or wire mesh is placed in the middle of the slab.

FOUNDATIONS

The foundations for the columns, walls, and machinery, are usually made of either plain or reinforced concrete. The bottom of out-

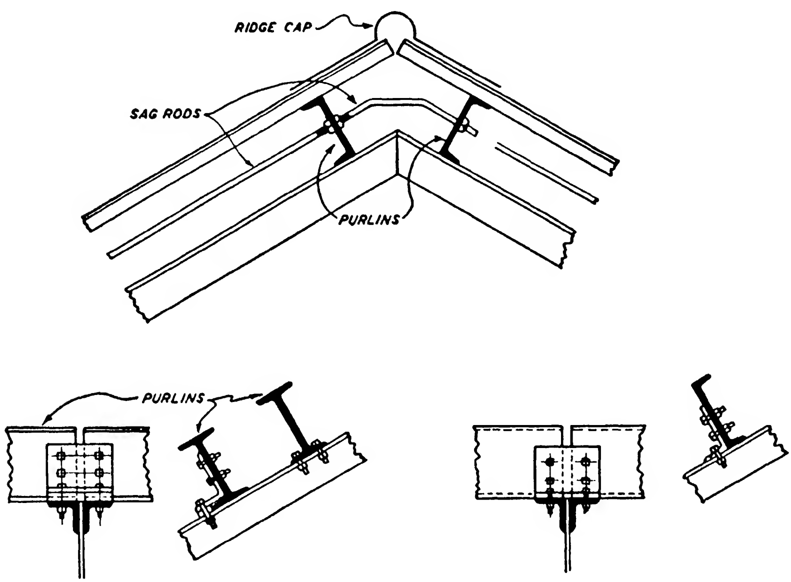


Fig. 115. Sag Rods and Purlin Connections

side wall and column footings should be placed at least below frost line—which is approximately three or four feet below the surface.

Machinery foundations should be divorced from the concrete floor slab by placing a $\frac{1}{2}$-inch expansion joint between the foundation and floor slab. This is done so that any vibration from the machinery will not be transmitted into the building frame.

PURLINS

Channels are usually the most economical section and type of purlin. Other types of purlins are: I beams, timber beams, and open or truss web steel joists.

When purlins rest on top of the sloping upper chords of roof trusses, the wind, snow, and dead loads act at an angle with the main axes of the purlins. Therefore, sag rods are used when the slope of the roof is more than 10 degrees or 2 in 12. In some cases where the slope of the roof is less than 2 in 12, it is desirable to use sag rods in order to support the top flanges of the purlins.,

These sag rods are designed to carry the components of the loads parallel with the roof. In designing the purlins the components normal to the roof (of the wind, snow, and dead loads) are used. In Fig. 145 are shown sag rods and purlin connections.

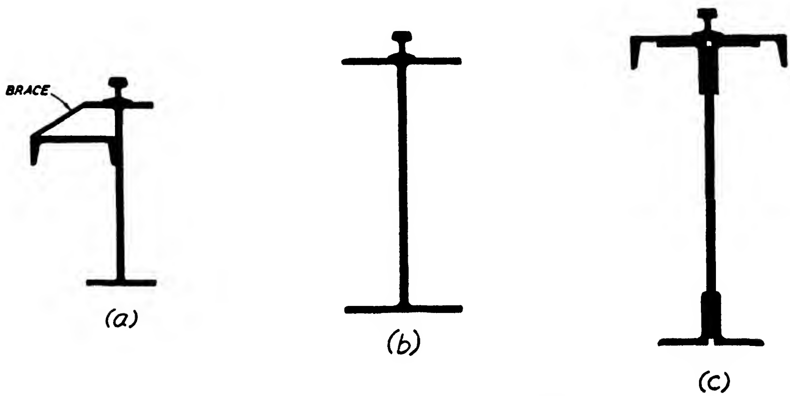


Fig. 146. Typical Crane Runway Girders

When timber sheathing is used and it is nailed securely to nailing strip bolted to the purlins, sag rods can be dispensed with. However, tie rods or diaphragms should be placed between the two purlins at the ridge.

CRANE GIRDERS

Girders for crane runways are designed in a manner similar to that illustrated in Chapter VIII.

The live load (load carried plus weight of crane) is transmitted to the girder from a two- or four-wheel truck. The crane manufacturers supply wheel loads and dimensions covering their cranes. In addition to the live load it is customary to add 25 per cent to provide for impact. The lateral force on crane runways should be taken as 20 per cent of the capacity of the crane and the weight of the trans-

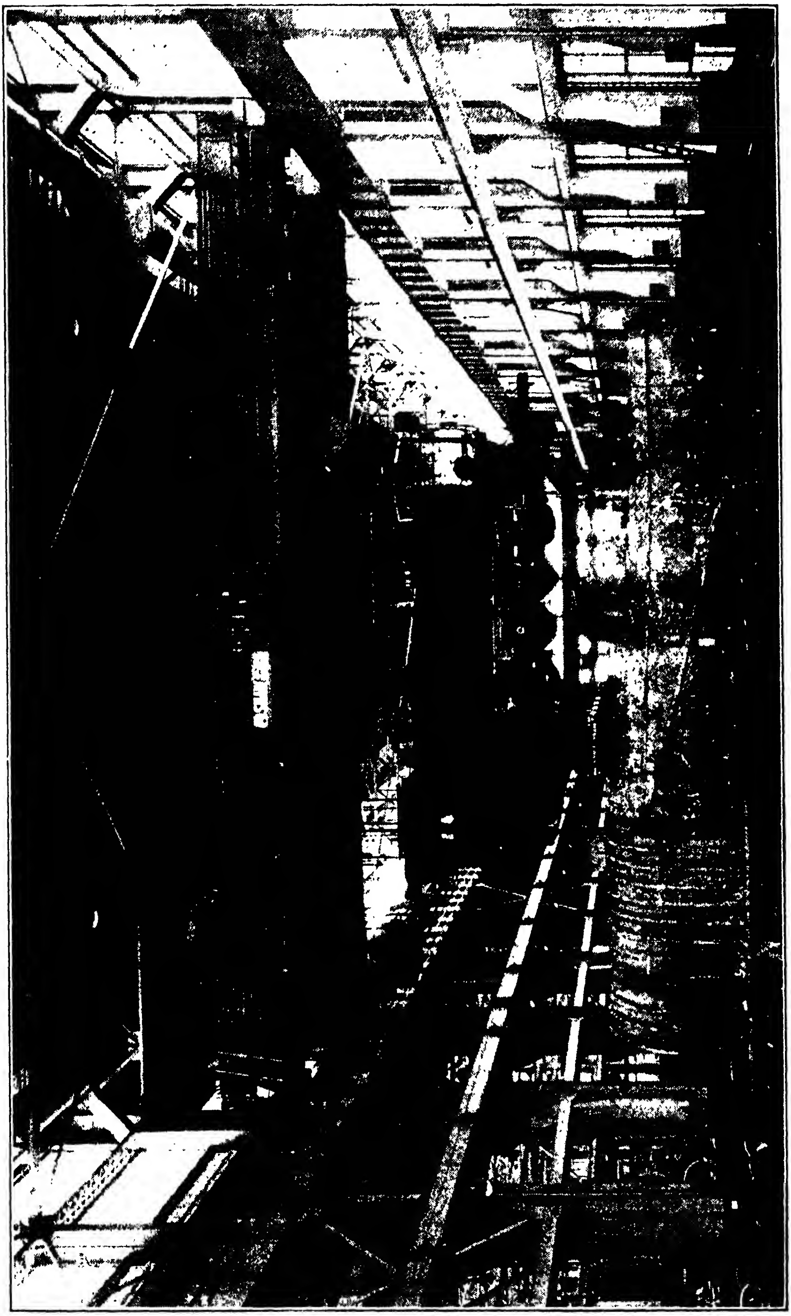


Fig. 147. Locomotive Repair Shop
Courtesy of A. T. & S. F. Railway System

verse trolley. This load is applied at the top of the crane rail, one half of the load to each side of the runway, and is considered as acting in either direction transverse to the runway rail.

Provision should also be made for a longitudinal force per rail equal to 10 per cent of the wheel loads of the crane applied at the top of the rail on the runway.

In Fig. 146 are shown some typical crane girder sections. When the type of crane girder shown in Fig. 146 at *a* is used, the channel

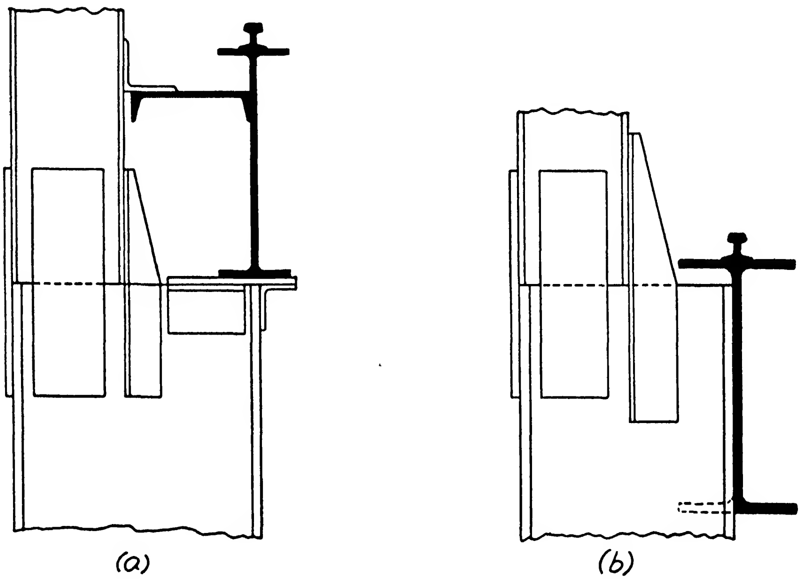


Fig. 148. Connections of Crane Girders to Columns

should be braced to the top flange or else there will be a tendency for the web to crack just above where the channel is connected to the web. Fig. 147 shows two crane runways of different size and capacity in one building.

Crane girders can be connected to the columns in two ways as shown in Fig. 148. The connection illustrated in Fig. 148 at *a* is the one most commonly used. On a connection such as Fig. 148 at *b*, the rivets have a tendency to work loose when the crane is used a great deal.

The rail fastenings are usually one of the three types shown in

Fig. 149. All of these types incorporate adjustable features for aligning the rail. It is customary to use one-bolt fastenings for runways for cranes under 25 tons; above 25 tons, two-bolt connections. Hook

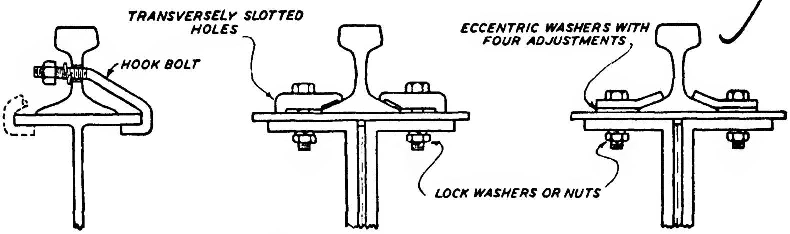


Fig. 149. Crane Rail Fastenings

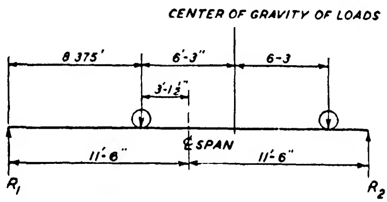


Fig. 150. Location of Crane Wheels for Maximum Moment

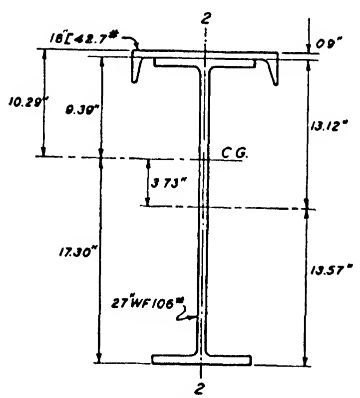


Fig. 151. Section of Assumed Crane Girder

bolts should be used only where the crane girder has such a narrow flange that clips could not be used. The spacing between pairs of clips is usually 3 feet, and for hook bolts, 2 feet. The space between pairs of bolts is made 3 inches.

Design of a Crane Girder. The span will be taken as 23 feet; crane capacity as 40 tons. The crane used has two-wheel trucks with the wheels spaced 12′-6″ on centers. Wheel loads equal 64,000#. A.I.S.C. Specifications will be used.

Fig. 150 shows the location of the crane wheels to produce the maximum moment in the crane girder. In order to get the maximum bending moment under a wheel, the wheel must be placed just as far on one side of the center line of span as the center of gravity of the loads is on the other side of the center line.

Dead load per foot of girder

100# rail	= 33# per ft.
27″ WF 106# girder	= 106
18″ [42.7#	= 43
	182# per ft.

Fig. 151 shows a section of the assumed girder.

Live load reaction R_1 equals

$$R_1 = \frac{128{,}000 \times 8.375}{23} = 46{,}600\#$$

Moment in girder equals

Live load $= 46{,}600 \times 8.375$	$= 390{,}000$ ft.-lb.
Impact, 25%	$= 97{,}500$
Dead load $\dfrac{182 \times (23)^2}{8}$	$= 12{,}000$
Total	499,500 ft.-lb.

The moment that will govern the design of the bottom flange will be

$$499{,}500 \times 12 = 5{,}999{,}000 \text{ in.-lb.}$$

The top flange of each girder has to resist a lateral force of 10% of the lifted load plus the weight of the trolley.

Lifted load	= 80,000#
Weight of trolley	= 31,000#
	111,000#

$$10\% \times 111{,}000 = 11{,}100\#$$

$$\text{Lateral reaction} = \frac{11{,}100 \times 8.375}{23} = 4{,}040\#$$

$$\text{Lateral moment} = 4{,}040 \times 8.375 = 33{,}850 \text{ ft.-lb.} = 406{,}000 \text{ in.-lb.}$$

To locate center of gravity of girder section, refer to Fig. 151. Moments will be taken about the base.

$$
\begin{array}{ll}
31.17 \times 13.57 & = 423 \\
\underline{12.48} \times 26.69 & = \underline{332} \\
43.65 & \quad 755
\end{array}
$$

Distance from base to $C.\ G. = 755 \div 43.65 = 17.3''$

Moment of Inertia of section equals

$$
\begin{array}{lr}
\text{WF Beam} & = 3{,}761 \\
ad^2 = 31.17 \times (3.73)^2 = & 433 \\
\text{Channel} & = \quad 15 \\
ad^2 = 12.48 \times (9.39)^2 = & \underline{1{,}100} \\
 & 5{,}309
\end{array}
$$

The bottom flange stress due to the vertical loads will then be

$$f_s = \frac{Mc}{I}$$

$$f_s = \frac{5{,}999{,}000 \times 17.3}{5{,}309} = 19{,}500\# \text{ per sq. in.}$$

This is satisfactory for the tension flange as the allowed stress is 20,000# per sq. in.

The lateral force from the crane is assumed as taken by the top flange. The moment of inertia of the top flange about axis *2-2* equals

$$
\begin{array}{lr}
\text{Channel} & = 549 \\
\text{Top flange WF beam same as } 10'' \times \frac{7}{8}'' \text{ plate} = & \underline{\ 73} \\
 & 622
\end{array}
$$

The compressive stress in the top flange will equal:

Due to lateral loads

$$f_s = \frac{Mc}{I} = \frac{406{,}000 \times 9}{622} = 5{,}880\# \text{ per sq. in.}$$

Due to vertical loads

$$f_s = \frac{Mc}{I} = \frac{5{,}999{,}000 \times 10.29}{5{,}309} = 11{,}600\# \text{ per sq. in.}$$

Total compressive stress $= 5{,}880 + 11{,}600 = 17{,}480\#$ per sq. in.

The $\frac{l}{b}$ ratio may be such as to require the unit stress to be reduced; therefore this will be checked.

$$\frac{l}{b} = \frac{23 \times 12}{18} = 15.3$$

From the table shown in Section 12 of the Specifications, it is noted that for an $\frac{l}{b}$ of 15.3 the allowable unit stress is approximately 19,900#. The section selected is satisfactory both in the tension and compression flanges.

Crane Girder Columns. Crane runway girders are either supported on brackets connected to the columns, on independent columns, or on one of the column flanges. Due to the crane loads and the manner of connecting the girders, the crane columns must be designed for axial, eccentric, and transverse forces. There is also a longitudinal force acting, due to the starting and stopping of the traveling crane; however, this force is usually taken to the column bases by longitudinal bracing in the plane of the columns.

The method of computing the stresses in crane girder columns varies among structural engineers. The stresses are not statically determinate; furthermore, the degree of fixedness at the top and bottom of the columns is unknown.

The loads for which these columns should be investigated are as follows: (1) Dead load, live load, impact, and wind load. (2) Dead load, live load, impact, and lateral force due to the thrust of the crane. It is not customary to include wind load when the lateral thrust of the crane is used. Some designers also feel that it is not necessary to use any impact when the lateral force is included. When wind load is added, an increase in stress is allowed. The load combination shown in (*2*) usually governs.

In addition to the loads mentioned, there is another one that should be included, and that is the eccentric load effect due to the deflection of the crane girder when the crane is in one panel. This eccentric effect is assumed as equal to the crane girder reaction applied three inches from the end of the girder. The distance from the end of the girder, of course, depends on the type of end bearing or connection.

In determining the stresses in columns where the girders are supported on brackets, the ends of the columns can be considered as hinged; then the stresses are statically determinate. If the ends of the columns are well secured, a reduction in the moment can be made. A reduction of 20 or 25 per cent is customary.

In Fig. 152 at *a* is shown a column with a crane bracket. Fig. 152 at *b* shows the bending moment diagram with the ends hinged.

The moments are computed as follows:

$$W \times e = H \times h$$

$$H = \frac{W \times e}{h}$$

$$\text{Moment at } C = M_c = \frac{W \times e}{h} \times h_1$$

$$\text{Moment at } D = M_d = \frac{W \times e}{h} \times h_2$$

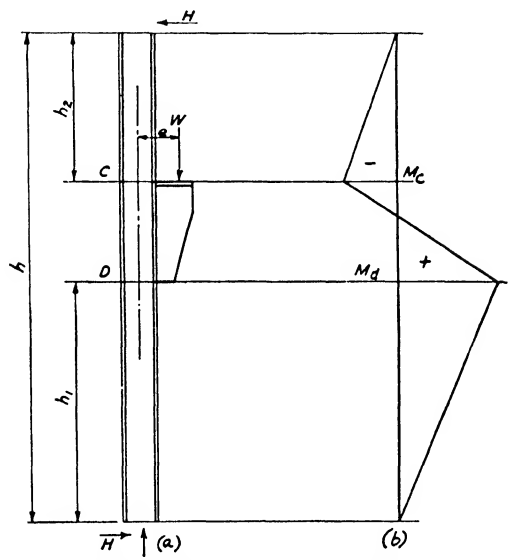


Fig. 152. Bending Moment Diagram for Column with Crane Bracket and Hinged Ends

To these moments should be added that due to the crane thrust, which will be the simple beam moment for a concentrated load.

If the ends of the columns are more fixed than hinged, a percentage of the total moment may be taken. The unit stress in the column due to these forces will then equal

$$p = \frac{W}{A} + \frac{Mc_{1-1}}{I_{1-1}}$$

In addition to this column stress, should be included that due to the deflection of the crane girder. The unit column stress will then be

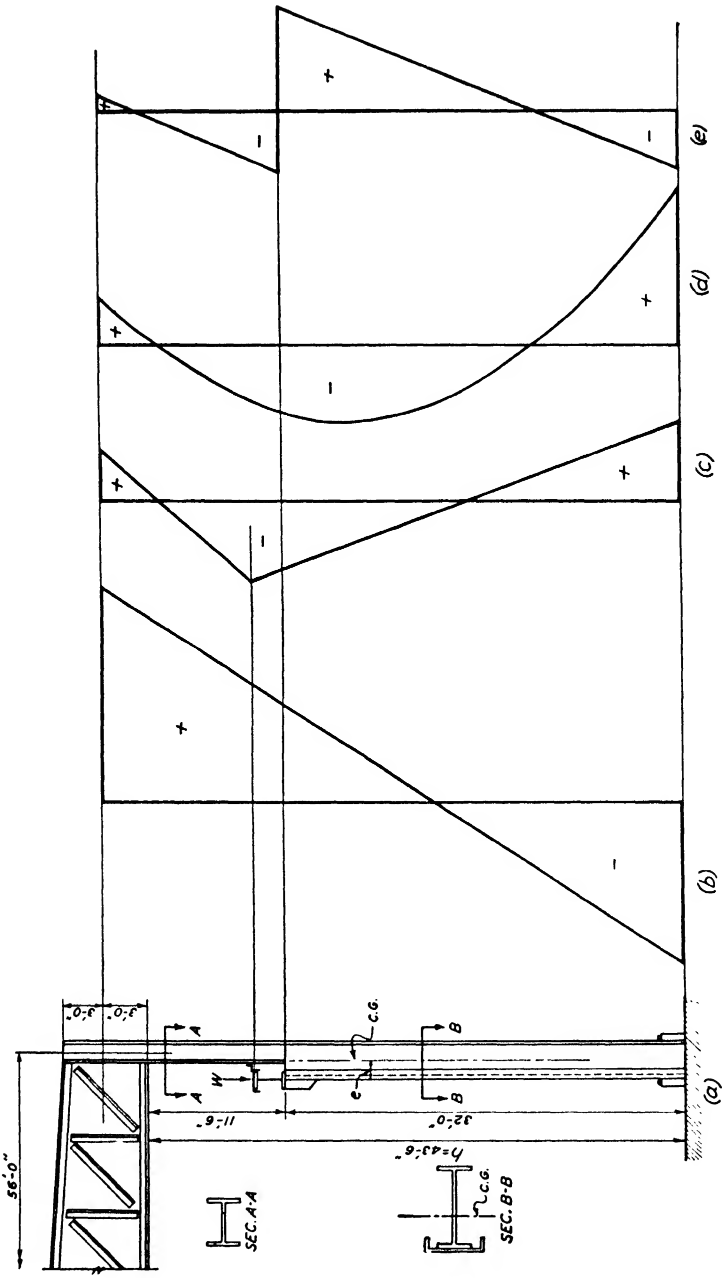


Fig. 153 Bending Moment Diagram for Heavy Crane Column with Ends Rigidly Connected

equal to
$$p=\frac{W}{A}+\frac{Mc_{1-1}}{I_{1-1}}+\frac{Mc_{2-2}}{I_{2-2}}$$

As most of the bending which is due to the deflection of the girder will be resisted by only one flange, it is better practice to use the I_{2-2} of the flange to which the bracket is connected, instead of the I_{2-2} for the whole section.

For heavier cranes a column similar to that shown in Fig. 153 is usually used. A rigid frame analysis was made of this bent, assuming the top and bottom of the columns rigidly connected, and the bending moment diagrams for the column under various types of loading are shown in Figs. 153 at *b, c, d,* and *e*. A combination of these moments should be used to secure a maximum.

Attention is called to the fact that the moments due to crane thrust can be reversed, as this lateral force can act in either direction. The moments due to roof load will be reduced if snow load is omitted.

Fig. 153 at *b, c, d,* and *e* shows the bending moment diagrams for the column under the following loadings respectively: roof, lateral crane thrust, wind, and vertical crane load.

For the roof load the bending moments at the top and bottom of the column are respectively 32 and 24 per cent of the simple beam moment of the roof truss.

The moments at the top and bottom of the column due to the crane thrust are respectively 37 and 56 per cent of the simple beam moment in the column.

For the wind load the moments at the top and bottom of the column are respectively 28 and 92 per cent of the simple beam moment in the column.

Under the vertical crane load the moments at the top and bottom of the column are respectively 7 and 32 per cent of the crane moment ($W \times e$).

With the truss framed to the column as shown in Fig. 153, this joint can be considered rigidly connected. There is some question whether a column can be considered fixed at the bottom even if a substantial anchorage is used. The column may be rigidly fixed to the footing; however, the footing may rotate slightly.

If the column is not completely fixed at the bottom, the moments at the top will be increased.

This type of column is designed by some engineers by assuming that the vertical load from the crane, and the eccentric effect due to the deflection of the crane girder are taken by the inner flange, which consists of the column flange, the cover (whether plate, channel, or I beam), and about 10 inches of the web.

In selecting the sizes for the top and bottom portions of a crane girder column similar to the one shown in Fig. 153, sufficient clearance should be allowed so that the ends of the crane girders will clear the top portion of the column by 2 inches and preferably a little more. The top portion of the column should also be made relatively stiff so the lateral forces from the crane will be transferred to the lower lateral bracing system.

Crane brackets should not be used for supporting crane runway girders where the crane capacity is larger than 10 or 15 tons. In Fig. 154 is shown a crane runway supported on brackets.

Independent columns support the crane girders shown in Fig. 155.

BRACING

In mill buildings without cranes, the lateral bracing in the plane of the lower chords of the roof trusses is customarily placed in every third or fourth bay. This is usually satisfactory bracing to resist the wind forces; however, when buildings have cranes, the bracing should be more substantial.

A satisfactory system of lateral bracing for buildings with cranes is illustrated in Fig. 156 at *a*. The advantage of this type of bracing is that it forms a complete truss throughout the length of the building. Where cranes are used, it is necessary to have an effective system of lateral bracing or else difficulty will be experienced in maintaining the alignment of the crane runways.

Lateral bracing in the plane of the top chords of the trusses is provided in every third or fourth bay. The purlins act as struts to transmit the forces to the braced bays. The stresses in the lateral bracing are small, therefore the sections are determined according to the requirements for allowable ratio l to r.

Bracing is also required longitudinally in the plane of the columns. This bracing carries to the column bases the wind force acting on the end of the building, also the longitudinal force from traveling cranes. The type of longitudinal bracing to use depends on the height

Fig. 154. Crane Girder Runways Supported on Brackets
Courtesy of Whiting Corporation, Harvey, Illinois

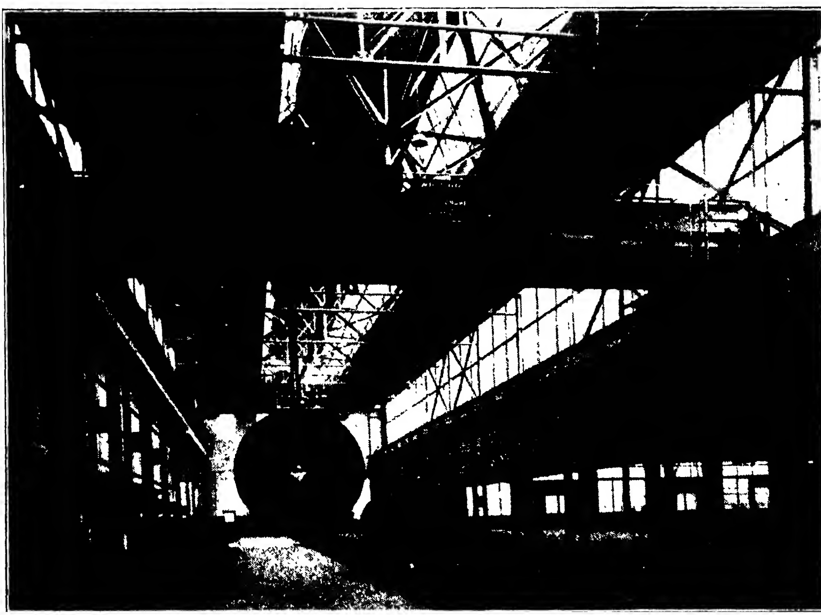


Fig. 155. Crane Girder Runways Supported on Independent Columns
Courtesy of Whiting Corporation, Harvey, Illinois

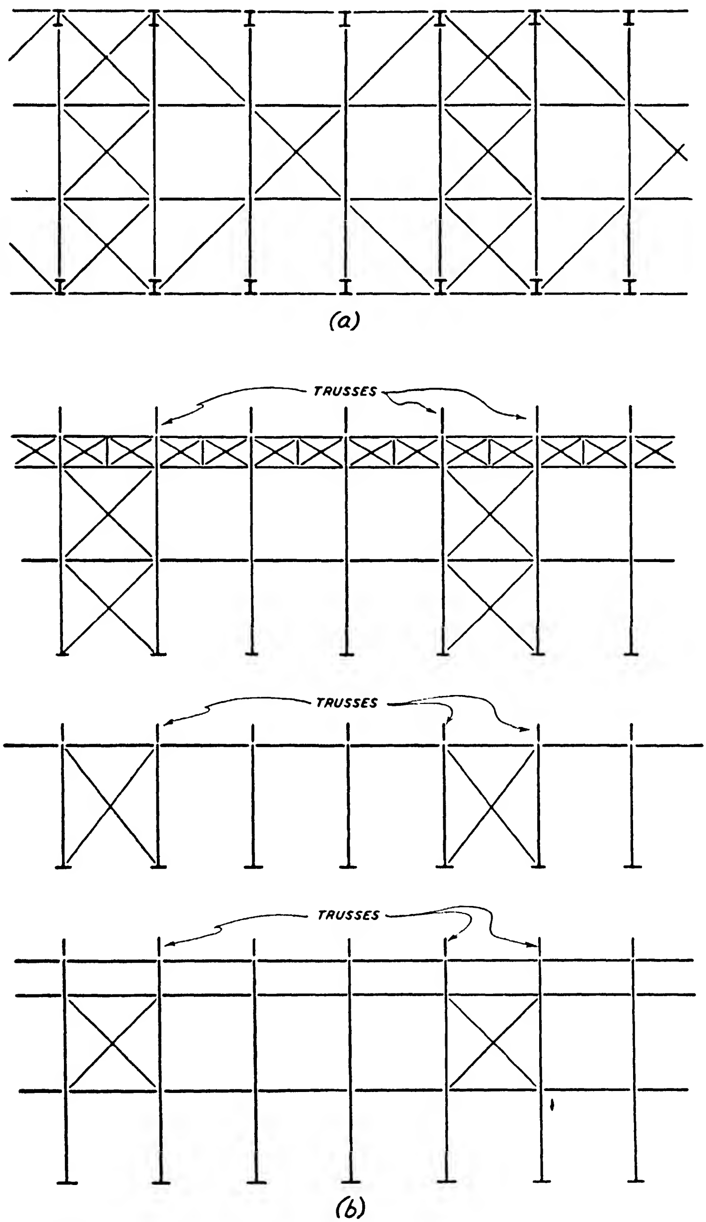


Fig. 156. Lateral and Longitudinal Bracing. (*a*) Lateral Bracing for Mill Building with Crane; (*b*) Longitudinal Bracing in Plane of Columns

of the building. Three systems of longitudinal bracing are shown in Fig. 156 at *b*.

ERECTION

A large number of mill buildings are erected using unfinished bolts for the field connections. A bolted structure has the advantage that it can readily be taken down and moved—also it is easily salvaged.

In buildings with cranes of over 5-ton capacity, all field connections, except those for purlins and girts, should be riveted.

In Fig. 157 is shown an impact wrench which is a very efficient tool used to turn bolts up tightly and rapidly.

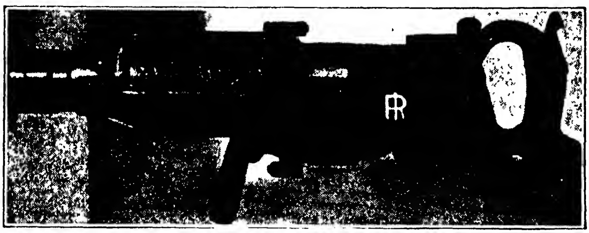


Fig. 157. Impact Wrench
Courtesy of Ingersoll-Rand Company

DESIGN OF A MILL BUILDING

As an illustrative example, the design of the typical members in a Power House will be made. The complete structural plans for this building are shown in Figs. 158 to 163. The side and end elevations, also a typical section, are shown in Figs. 164 and 165.

This building is located in the city of Chicago, therefore it is necessary that the design comply with the city code. This municipal code is in accordance with the A.I.S.C. Specifications except as otherwise provided in the ordinance. The main exception is that an 18,000-pound basic stress is used instead of 20,000 pounds as allowed by the A.I.S.C.

The design of roof trusses is not covered in this book, therefore no design will be made of the trusses for this building. The stresses and sections of the members are shown in Fig. 161.

Purlins. The first step in the design of the purlins is to determine the load to be carried. A tar and gravel roof with sheathing $1\frac{5}{8}$

(*Continued on page 244*)

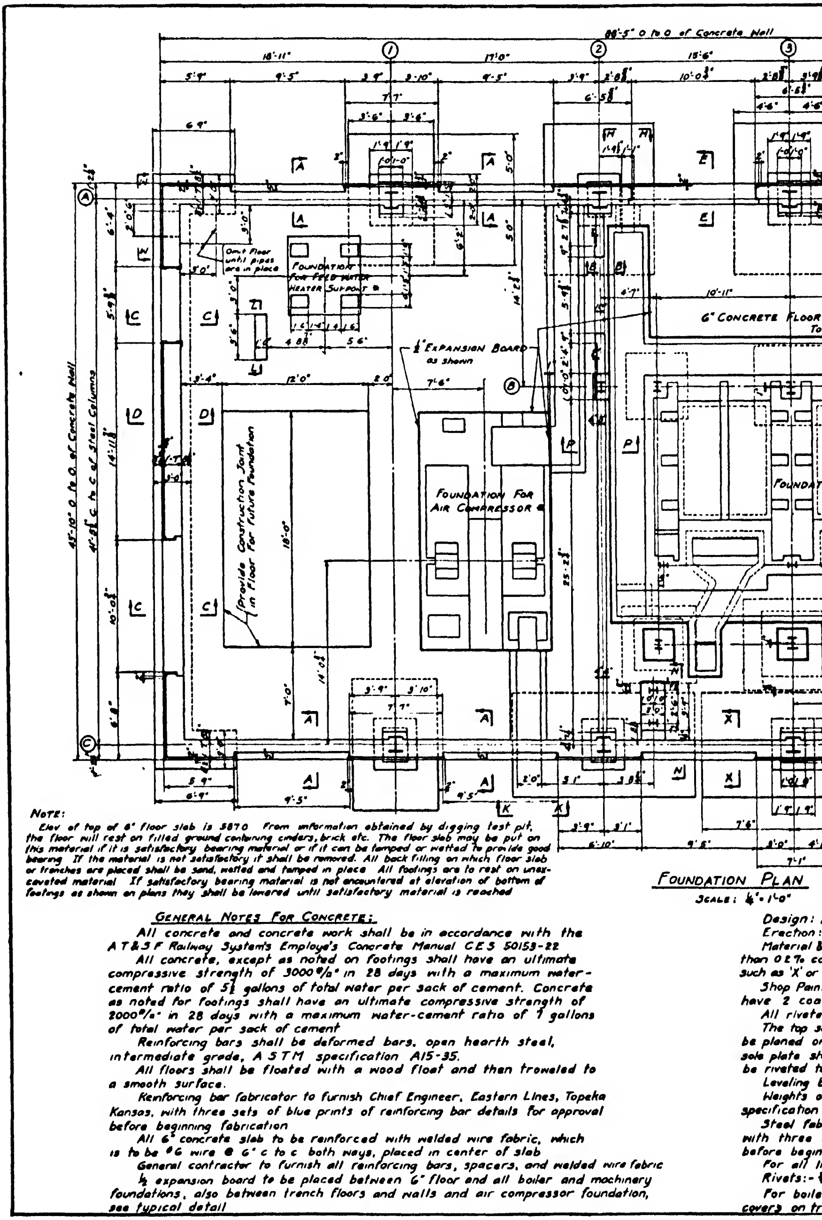


Fig. 158A

Courtesy of A. T. & S. F. Railway System

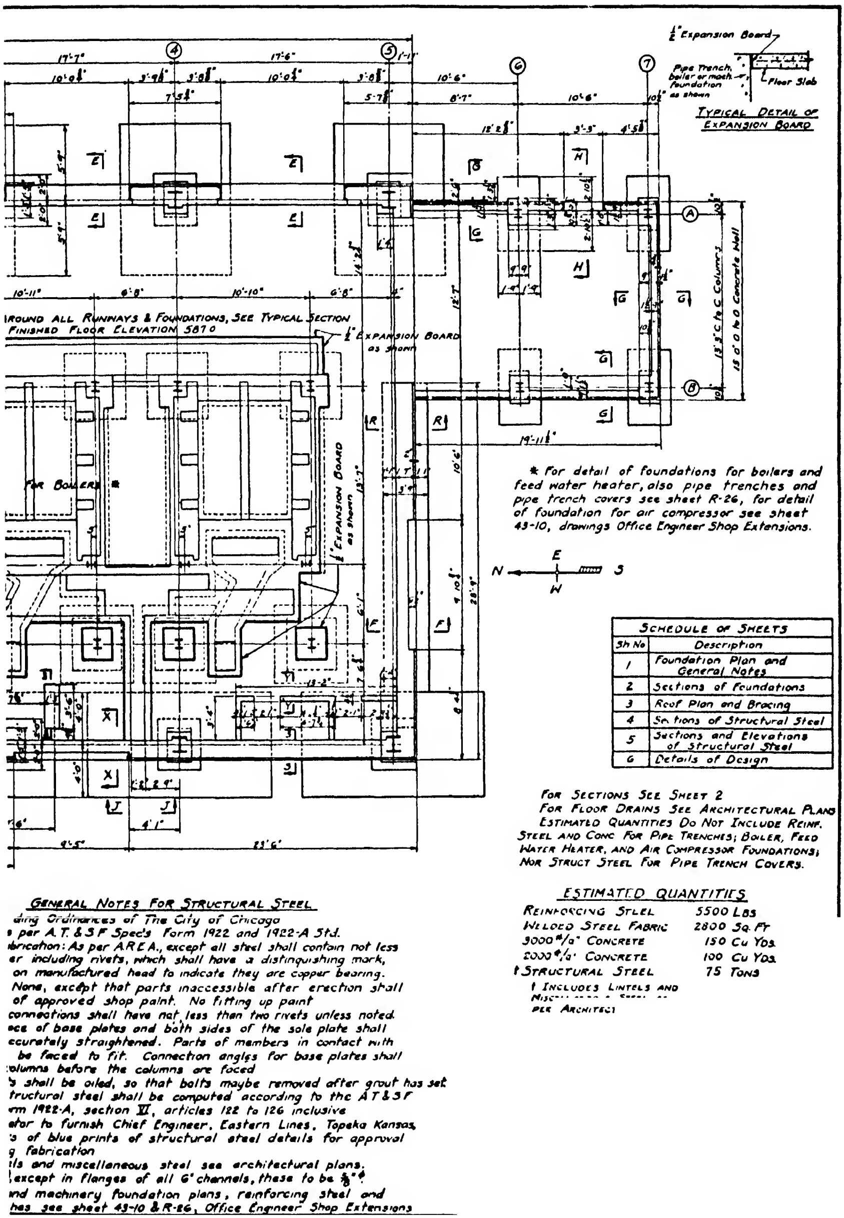


Fig. 158B

Courtesy of A. T. & S. F. Railway System

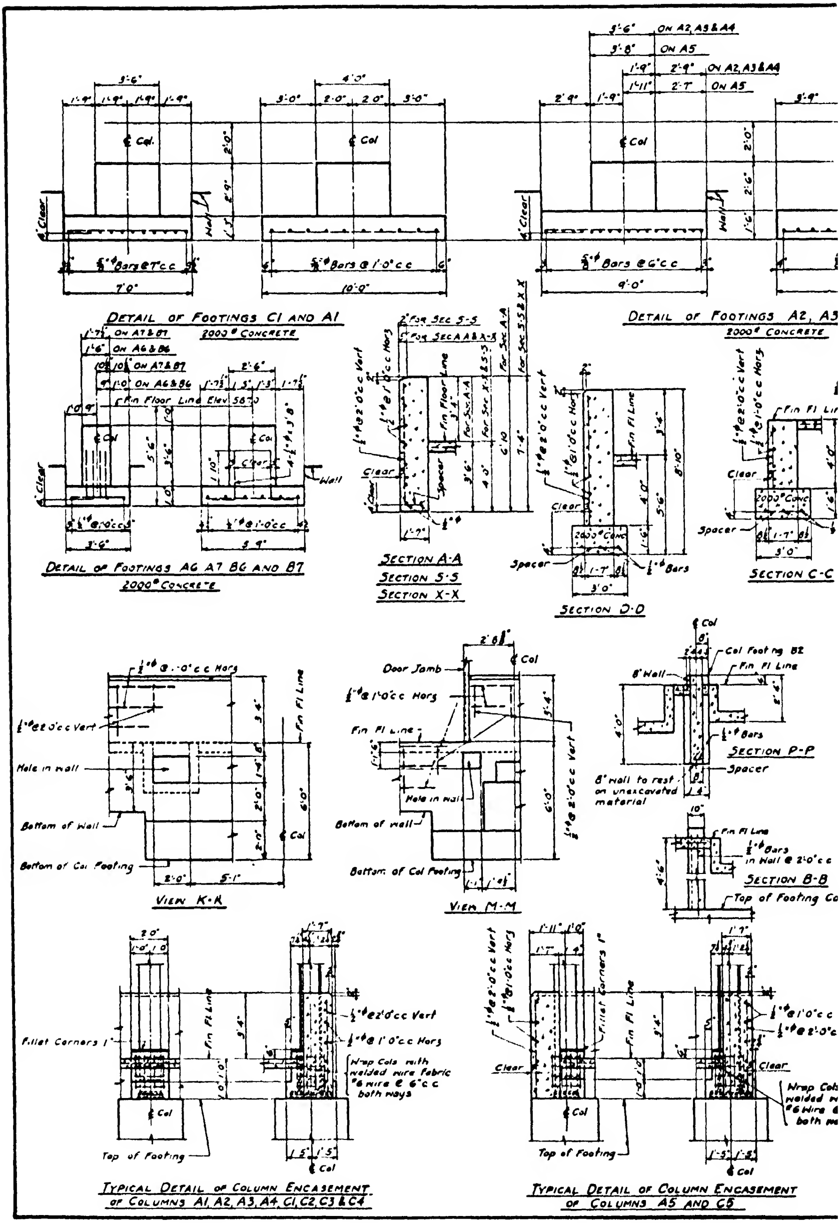


Fig. 159A

Courtesy of A. T. & S. F. Railway System

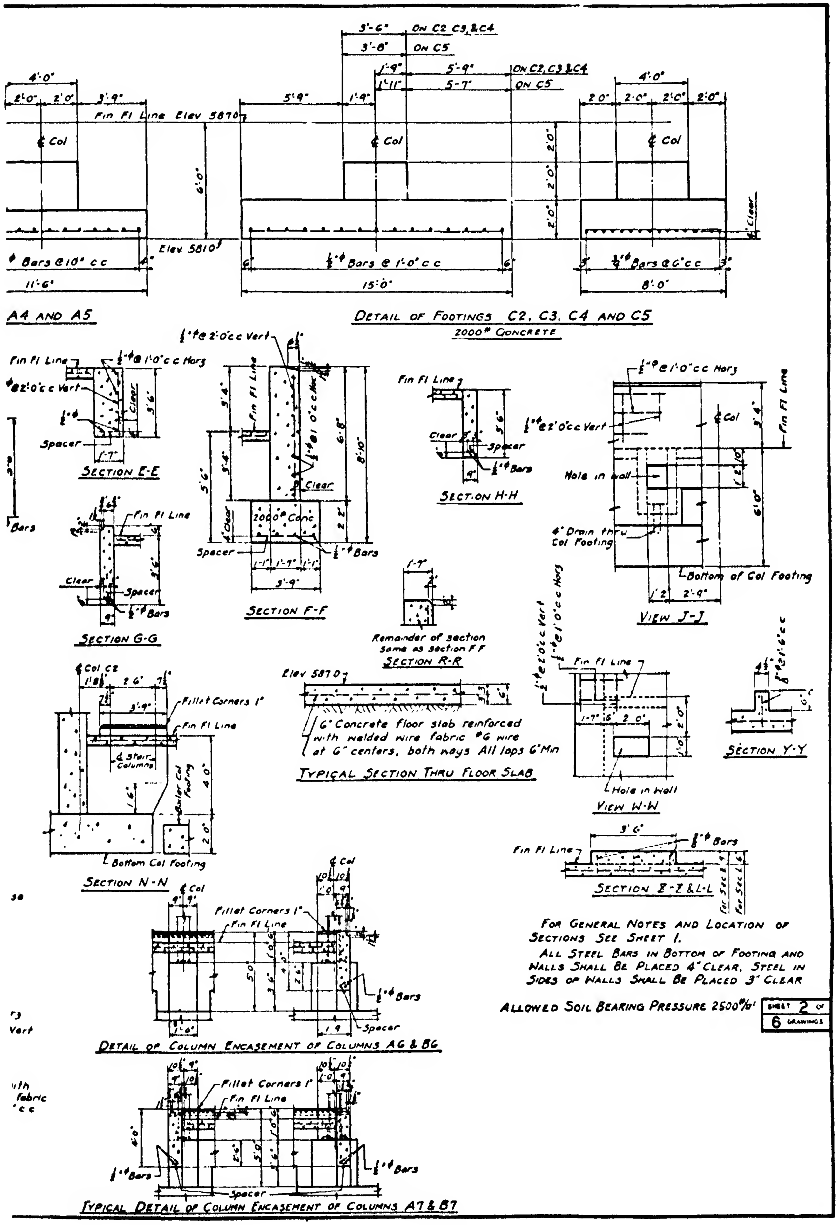


Fig. 159B

Courtesy of A. T. & S. F. Railway System

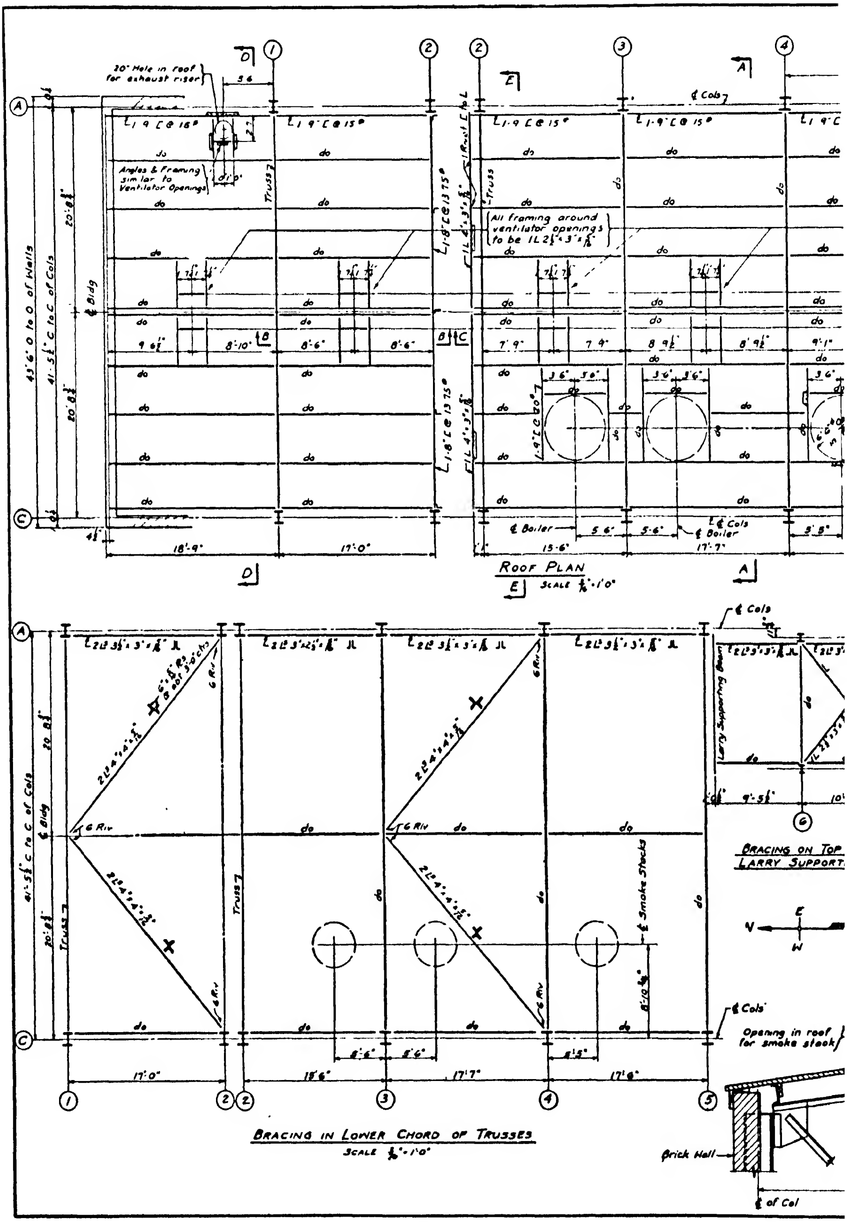


Fig. 160A

Courtesy of A. T. & S. F. Railway System

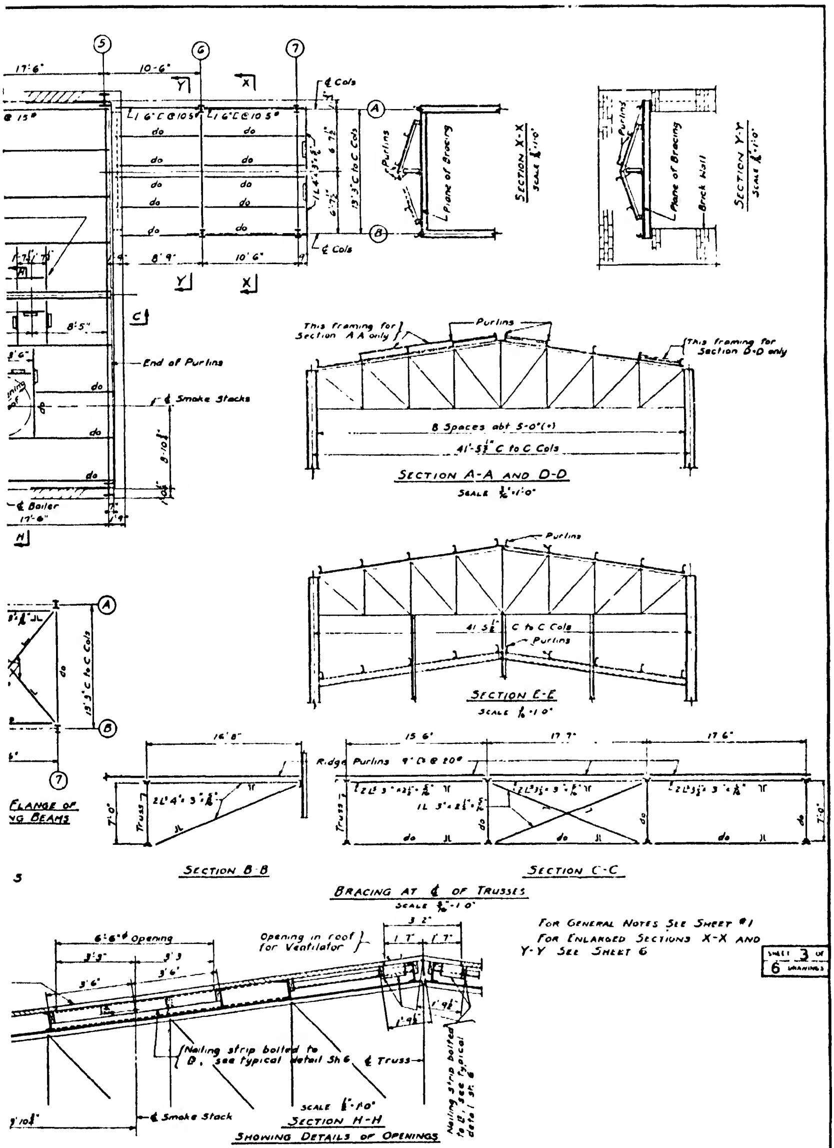


Fig. 160B

Courtesy of A. T. & S. F. Railway System

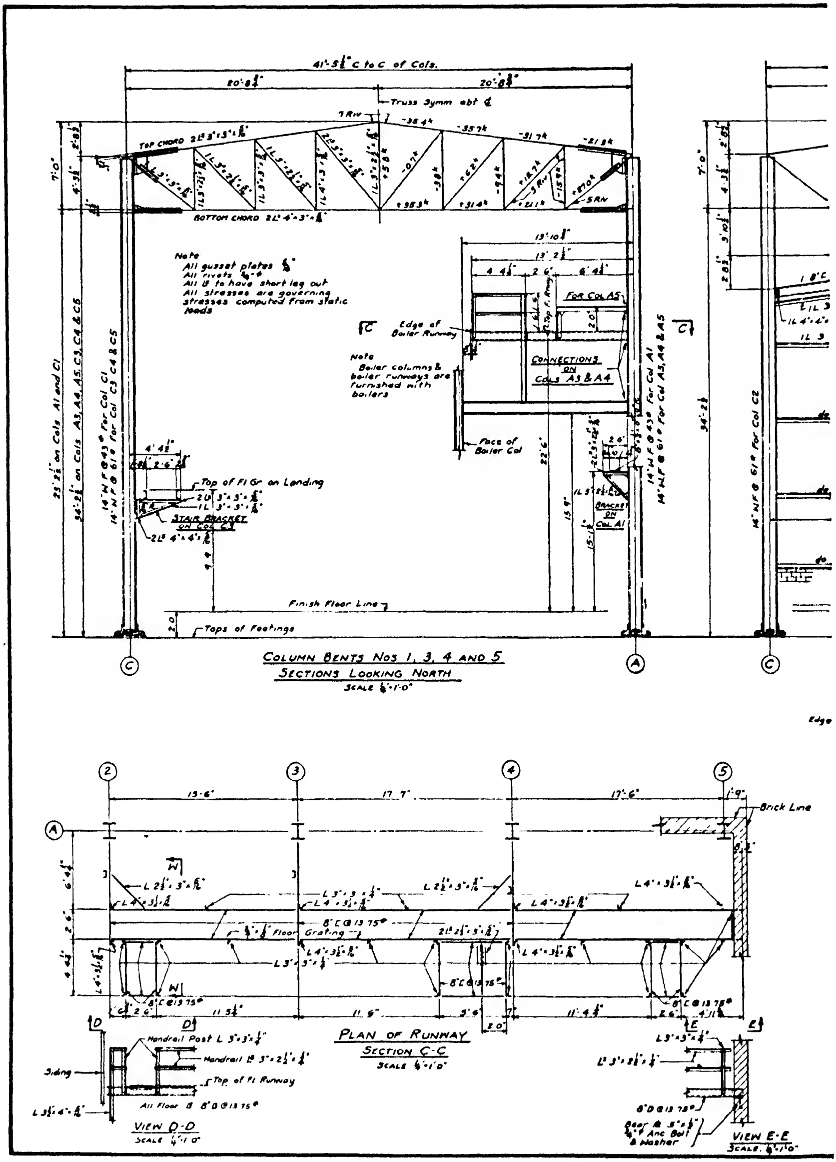


Fig. 161A
Courtesy of A. T. & S. F. Railway System

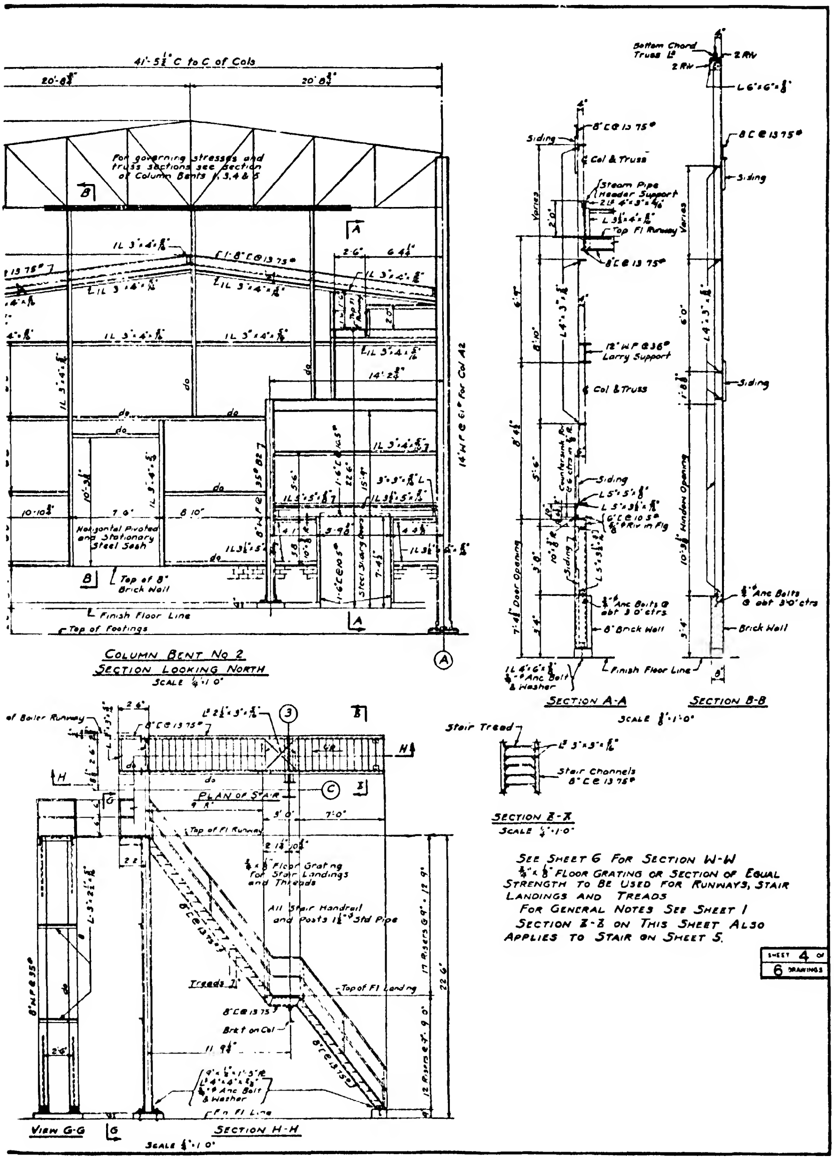


Fig. 161B

Courtesy of A. T. & S. F. Railway System

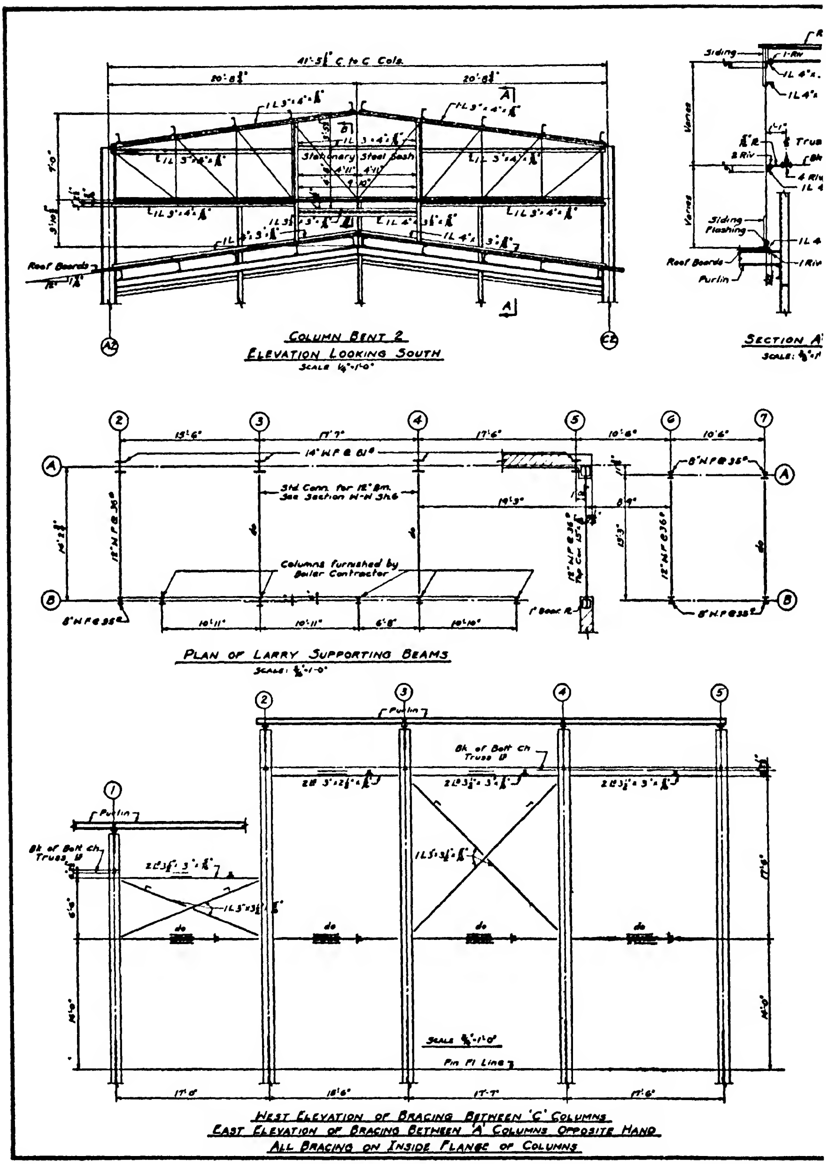


Fig. 162A
Courtesy of A. T. & S. F. Railway System

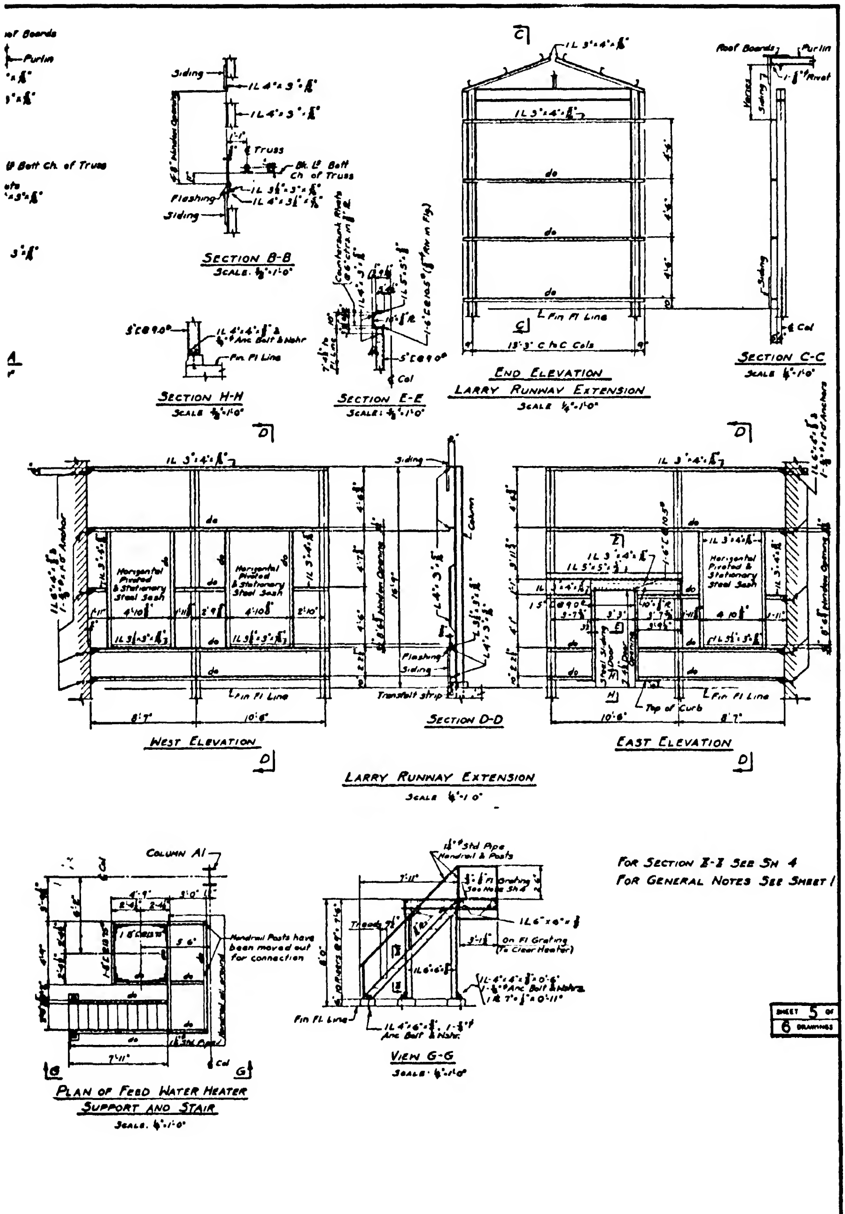


Fig. 162B
Courtesy of A. T. & S. F. Railway System

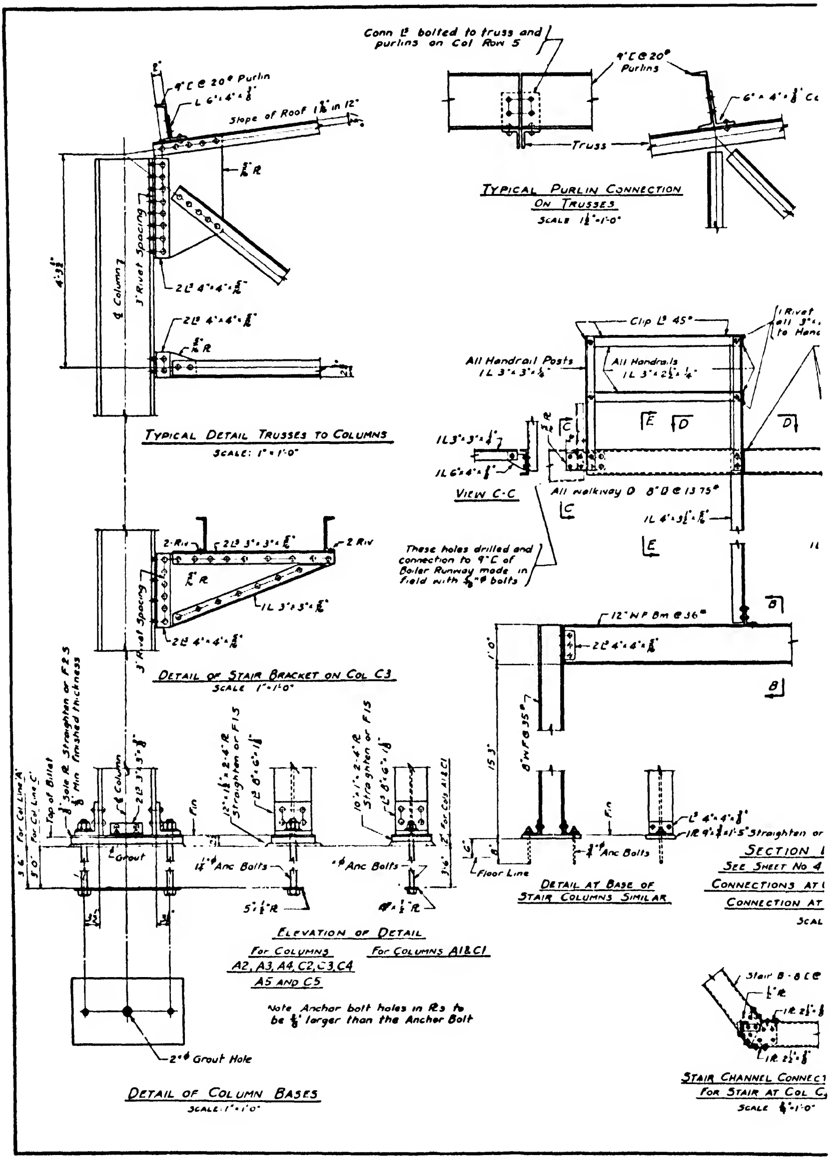


Fig. 163A
Courtesy of A. T. & S. F. Railway System

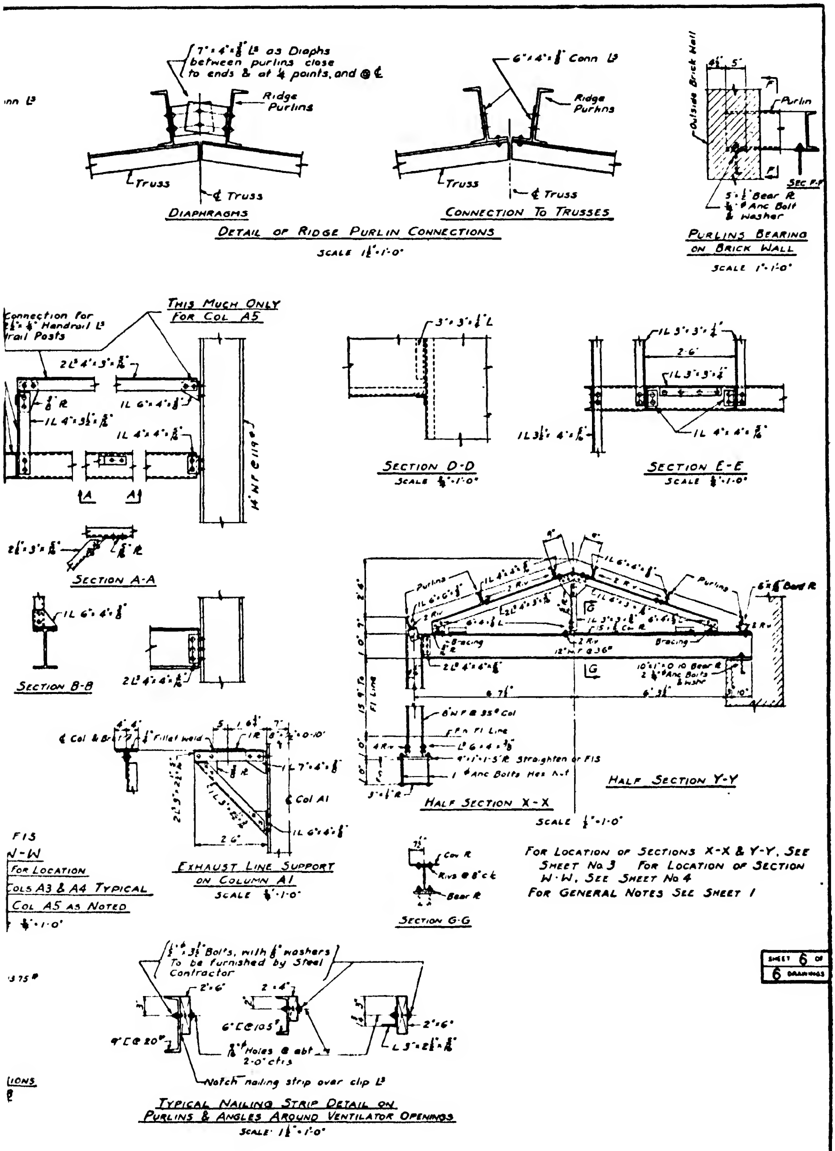
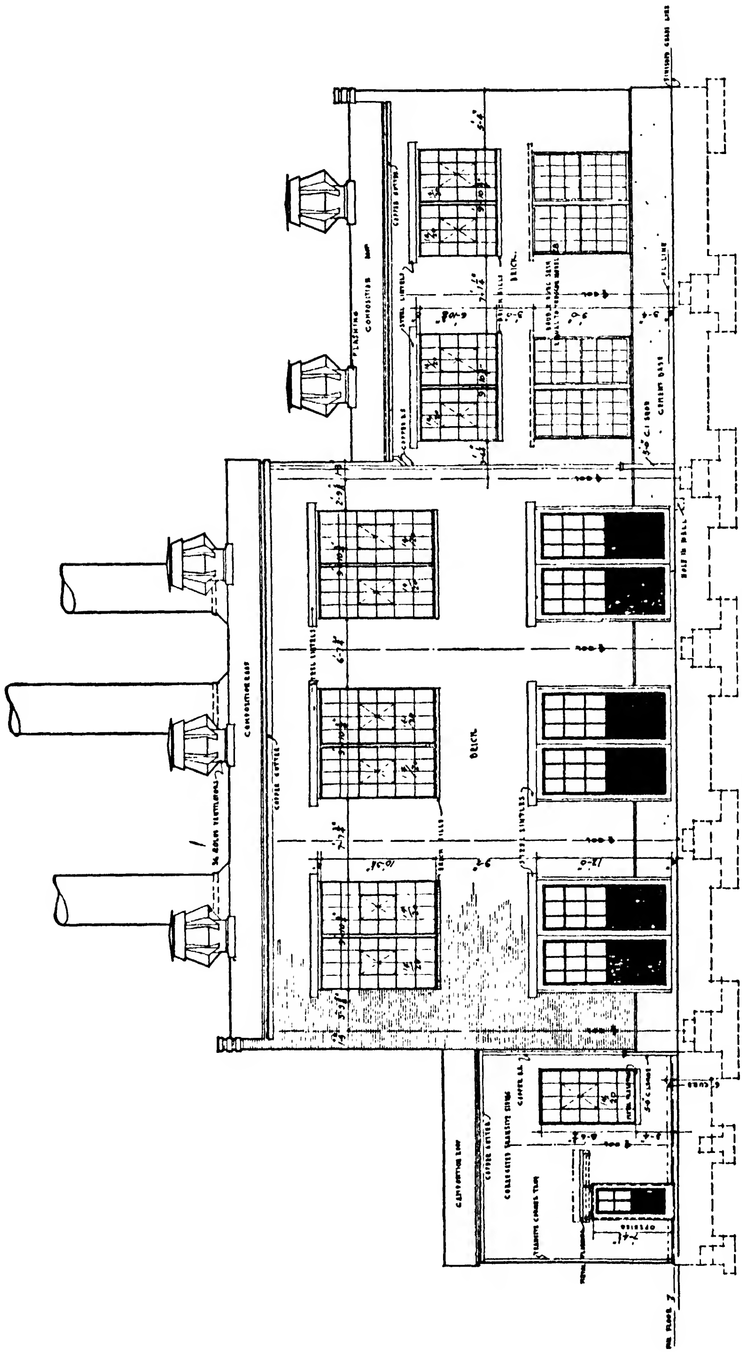


Fig. 163B

Courtesy of A. T. & S. F. Railway System



EAST ELEVATION

SCALE $\frac{1}{8}'' = 1'\text{-}0''$

Fig. 164

Courtesy of A. T. & S. F. Railway System

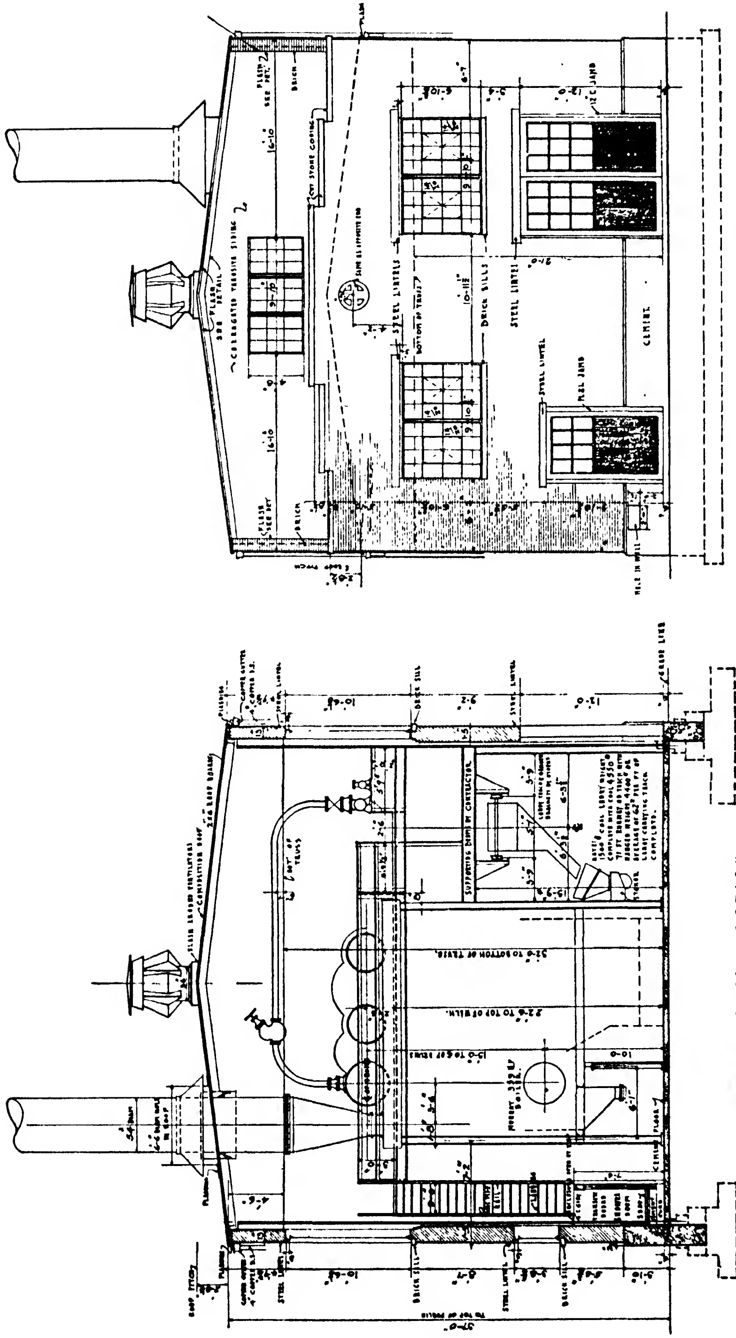


Fig. 165
Courtesy of A. T. & S. F. Railway System

inches thick will be used. The purlins will be placed at panel points which are about 5 feet on centers.

Load per square foot of roof equals

Required live load	25#
Assumed weight of purlins	3
$1\frac{5}{8}''$ timber sheathing	7
Tar and gravel roofing	10
Total	45#

The depth of purlins should be made between $\frac{1}{24}$ and $\frac{1}{30}$ of the span.

As this building is a power house it is felt that the minimum thickness of metal should be $\frac{5}{16}$-inch to provide for possible corrosion.

Referring to the roof plan, Fig. 160, it is noted that the maximum span length is approximately 18′-3″.

$$\frac{1}{24}\times 18.25\times 12 = 9.12 \text{ in.}$$
$$\frac{1}{30}\times 18.25\times 12 = 7.3 \text{ in.}$$

A purlin either 8 or 9 inches deep will be satisfactory; however, a 9-inch channel will be investigated first.

A 9″ **[** 15# has a $\frac{5}{16}$-inch web, therefore this section will be checked for strength. The loads will be assumed as acting normal to the roof. No sag rods are required, as timber sheathing is used; furthermore, the slope of the roof is less than 2 in 12. The sheathing will also serve as lateral support for the top flanges of the purlins.

$$\text{Bending moment} = M = \tfrac{1}{8}\times 5\times 45\times (18.25)^2\times 12$$
$$= 112{,}500 \text{ in.-lb.}$$

$$f_s = \frac{Mc}{I} = \frac{112{,}500}{11.3} = 9{,}950\# \text{ per sq. in.}$$

An 8″ **[** 13.75# has a section modulus of 9 and a web of $\frac{5}{16}$-inch thickness; therefore, this section will also be satisfactory. Due to the small difference in weight between the two sections, the stiffer 9-inch section will be used.

In computing the stresses in the purlins the wind load was not considered, as the normal component is very small due to the flat slope of the roof. For roofs having greater slopes, the wind load should be taken into account, and when this is done the unit stresses can be increased.

Columns. The columns will be assumed partially fixed at the top and bottom. The inflection point will be taken at a point up one third the distance from the bottom of the column to the lower chord of the truss. The columns can be assumed hinged at the inflection points, therefore only the loads above these points need to be considered. Refer to the diagram in Fig. 166. The vertical reaction on the columns, due to the live load and dead load from roof, truss, and ventilator, is 36,000#.

The Chicago code requires a wind load of 25# per sq. foot for this height of building.

The wind force per bent equals $2H$.

$$2H = 17.5' \times 29.67' \times 25\# = 13,000\#$$
$$H = 6,500\#$$

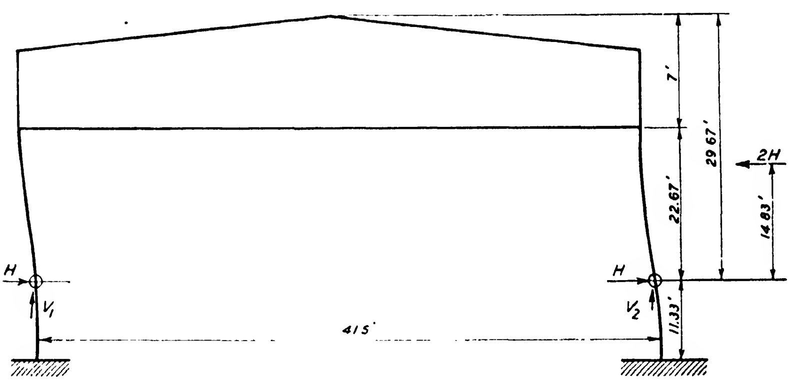


Fig. 166 Transverse Bent with Columns Fixed at Top and Bottom

The vertical reaction on the leeward column due to this wind force equals

$$\frac{13,000\# \times 14.83'}{41.5'} = 4,650\#$$

The total reaction on the leeward column equals

$$4,650 + 36,000 = 40,650\#$$

The moment on the column equals

$$6,500\# \times 22.67' = 147,400'\# = 1,770,000''\#$$

The maximum unit stress in the column will be that due to the direct load plus that due to bending

$$p = \frac{W}{A} + \frac{Mc}{I}$$

A 14″ WF 43# section will be selected for investigation.

$$A=12.65 \text{ sq. in.}$$
$$I_{1-1}=429$$
$$I_{2-2}=45.1$$

$$\frac{W}{A}=\frac{40{,}650}{12.65}=3{,}210\# \text{ per sq. in.}$$

$$\frac{Mc}{I}=\frac{1{,}770{,}000\times 6.84}{429}=28{,}200\# \text{ per sq. in.}$$

$$p=3210+28{,}200=31{,}410\# \text{ per sq. in.}$$

The Chicago code uses the following column formula, allowing a 50 per cent increase in stress when wind loads are included.

$$p=\frac{18{,}000}{1+\dfrac{l^2}{18{,}000\ r^2}}$$

$$l=34'\times 12=408''$$
$$r_{1-1}=5.82$$

$$p=\frac{18{,}000}{1+\dfrac{(408)^2}{18{,}000\times(5.82)^2}}=14{,}100\# \text{ per sq. in.}$$

Allowed stress $=14{,}100+50\%=21{,}200\#$ per sq. in.

The 14″ WF 43# section is overstressed, therefore a larger section will be tried.

Using 14″ WF 61# section

$$A=17.94,\ I_{1-1}=641.5,\ I_{2-2}=107.3,\ r_{1-1}=5.98$$

$$\frac{W}{A}=\frac{40{,}650}{17.94}=2{,}260\# \text{ per sq. in.}$$

$$\frac{Mc}{I}=\frac{1{,}770{,}000\times 6.95}{641.5}=19{,}200\# \text{ per sq. in.}$$

$$p=2{,}260+19{,}200=21{,}460\# \text{ per sq. in.}$$

The allowed unit stress will be slightly larger for this section as the value for r is larger. This section is satisfactory and will be used.

The column was not investigated in regard to the strength in the weak axis, as the longitudinal force due to wind on the end of the building is resisted by bracing in the plane of the columns and the vertical load is light.

Column Bases. The columns should be fixed at the bases. In order to do this, the anchor bolts will have to be designed to take uplift to resist the bending moment.

The anchor bolts will be placed $3\frac{1}{2}$ inches from the face of the column, therefore the distance between anchor bolts will be

$$13.91''+7''=20.91''$$

Moments will be taken about the right-hand anchor bolt, using the forces shown in Fig. 167.

$$\frac{6{,}500\times 11.33\times 12-40{,}650\times 10.45}{20.91}=19{,}100\#$$

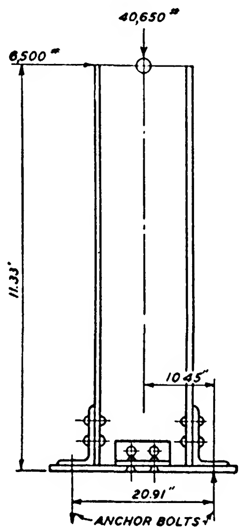


Fig. 167. Forces to Be Resisted by Anchor Bolts

Allowed unit stress in anchor bolts equals

$$18{,}000+50\%=27{,}000\# \text{ per sq. in.}$$

The required area through the threads of the bolts will then be

$$\frac{19{,}100}{27{,}000}=0.71 \text{ sq. in.}$$

Bolts $1\frac{1}{4}$ inches in diameter will be required. Heavy angles and a plate are used at the base of the column; however, a base similar to that shown in Fig. 132 at *b* is considered more effective.

The allowed bearing on the concrete is 600# per sq. in. The required size of the base slab will then be

$$\frac{36{,}000}{600}=60 \text{ sq. in.}$$

The size of the slab will have to be made larger than required. A slab $1\frac{1}{2}$ inches thick will be used. To help anchor the bolts and also to hold them in position while the concrete is poured, a plate is placed at the bottom between them.

Bracing. Due to interference from the smokestacks, a cross-frame type of lateral bracing cannot be used. Single type bracing taking both tension and compression has to be resorted to.

The wind forces on the ends of the building, which the lateral bracing transmits to the columns, is relatively small; therefore, the bracing is usually designed according to allowable l to r ratio. The

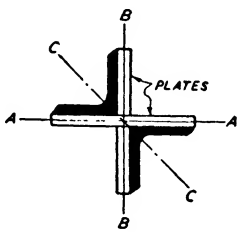


Fig. 168 Angle Compression Member

maximum $\frac{l}{r}$ ratio for bracing members taking compression is usually 200, and for tension, 300. A two-angle member, as shown in Fig. 168, will be used. From the handbooks, the least radius of gyration about axis *c-c* can be found.

The length of the lateral member is

$$\sqrt{20^2+(17.58)^2}=26.6$$

Try 2 angles $4''\times4''\times\frac{5}{16}''$

$$r_{c-c}=1.57$$

$$\frac{l}{r}=\frac{26.6\times12}{1.57}=203$$

Therefore the 2-angle $4''\times4''\times\frac{5}{16}''$ lateral members are satisfactory. Lateral bracing will be placed in the plane of the lower chords of the trusses in the bays between columns 1 and 2 and columns 3 and 4.

No lateral bracing is required in the plane of the top chords of the trusses, as tongue-and-groove timber sheathing is used.

Longitudinal bracing in the plane of the columns is required to transmit the wind forces from the lateral bracing down to the column bases. For buildings without cranes these longitudinal forces are relatively small and the l to r ratio governs the design.

The wind force on the end of the building that will be carried by one line of column bracing will be as follows. It is assumed that the wind force below the inflection point will be carried to the ground.

$$F=25\# \left[\left(\frac{41.5}{2}\times 29.67\right)-\left(\tfrac{1}{2}\times 2.67\times\frac{41.5}{2}\right)\right]=14{,}700\#$$

If bracing is placed at 45 degrees, $F=1.41\times 14{,}700=20{,}700\#$

$$\text{Required net area of bracing} = \frac{20{,}700}{18{,}000} = 1.15 \text{ sq. in.}$$

A cross-frame type of bracing will be used so the members will have to take tension only. The l to r ratio will be held to 300 or less.

Try $3\frac{1}{2}''\times 3''\times\frac{5}{16}''$ angles.

$$A=1.93 \text{ sq. in.}$$

$$Net\ A=1.93-(\tfrac{7}{8}\times\tfrac{5}{16})=1.66 \text{ sq. in.}$$

$$\text{Length of bracing members}=\sqrt{(17.5)^2+(17.5)^2}=24.8'$$

$$24.8\times 12=297 \text{ in.}$$

The members will be connected where they cross, therefore the length will be 297 inches in one direction and half of this in the other direction.

$$r_{1-1}=1.10$$

$$r_{2-2}=.90$$

$$\frac{l}{r}=\frac{297}{1.10}=270$$

$$\frac{l}{r}=\frac{148.5}{.90}=165$$

The $3\frac{1}{2}''\times 3''\times\frac{5}{16}''$ angles are satisfactory and will be used.

Two angles of the same size will be used as horizontal struts throughout the length of the building at the top and bottom of the cross bracing.

Girts. The girts on the end wall of the Larry Runway Extension will be designed.

Wind load per linear foot of girt equals

$$4.5'\times 25\#=112.5\#$$

Span of girt equals 13.25′

$$\text{Bending moment} = \frac{112.5 \times (13.25)^2}{8} = 2470'\#$$

Try $3\frac{1}{2}'' \times 4'' \times \frac{5}{16}''$ angles.

$$S_{1-1} = 1.3$$

$$f_s = \frac{2470 \times 12}{1.3} = 22{,}800\# \text{ per sq. in.}$$

Allowed stress = 18,000 + 50% = 27,000# per sq. in.

As the span is short, no sag rods are used; therefore the weight of the siding must be carried by the girts. The siding weighs about 5 pounds per sq. foot. Weight per linear foot of girt equals 4.5×5 plus the weight of the girt.

$$\text{Bending moment} = \frac{30 \times (13.25)^2}{8} = 658'\#$$

$$S_{2-2} = 1.0$$

$$f_s = \frac{658 \times 12}{1.0} = 7{,}900\# \text{ per sq. in.}$$

As this stress is low, the vertical leg can be made 3 inches. The members will then be $3'' \times 4'' \times \frac{5}{16}''$.

Beams Supporting Larry Track Hangers. The span of these beams is approximately 13′-6″ and the two concentrated loads occur 3′-9″ from the beam supports. The total load at these two points is 2800#.

$$\text{Bending moment} = 2800 \times 3.75 = 10{,}500'\#$$

Due to the lateral and impact forces that can be had from the Larry, it is desirable to use beams 12 inches deep so that a three-rivet end connection can be used. The lightest 12-inch WF beam with a $\frac{5}{16}$-inch web weighs 36 pounds. This section has a section modulus of 45.9, which is sufficiently strong and will be used.

In Fig. 169 is shown this Power House under construction.

PRACTICE PROBLEMS

In computing the following problems, use the A.I.S.C. Specifications.

1. Design the purlins to support a corrugated steel roof the slope of which is 6 in 12. The wind force is 20# per sq. ft. on the vertical projection. The span of the purlins is 20 ft. and they are spaced 5 ft. on centers. Snow load = 25# per sq. ft. Dead load of roofing = 3# per sq. ft.

2. Design a crane girder for a 20-ton crane. The maximum wheel loads are 36,000# and wheels are spaced 11′-6″ on centers. The trolley weight is 16,000#. The span of the crane girder is 20 ft.

3. Design a girt the span of which is 20 ft. Assume sag rods carry the weight of the wall. The girts are spaced 10 ft. on centers. Wind pressure equals 30# per sq. ft.

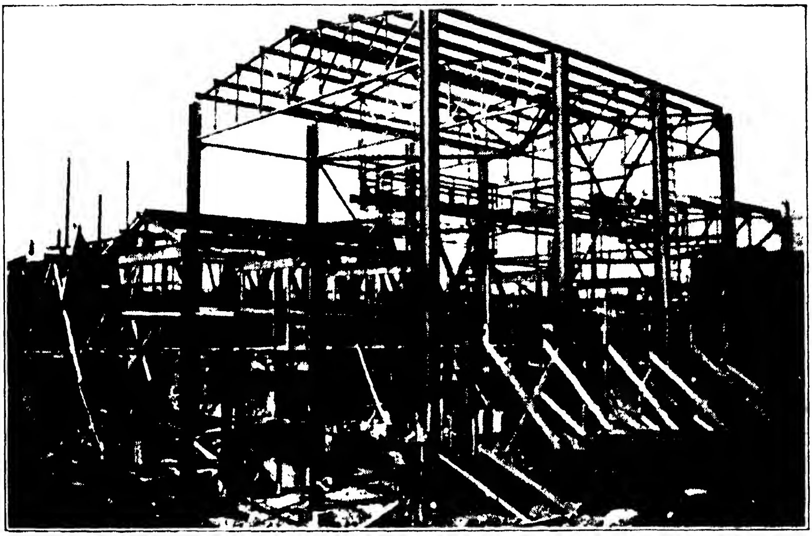
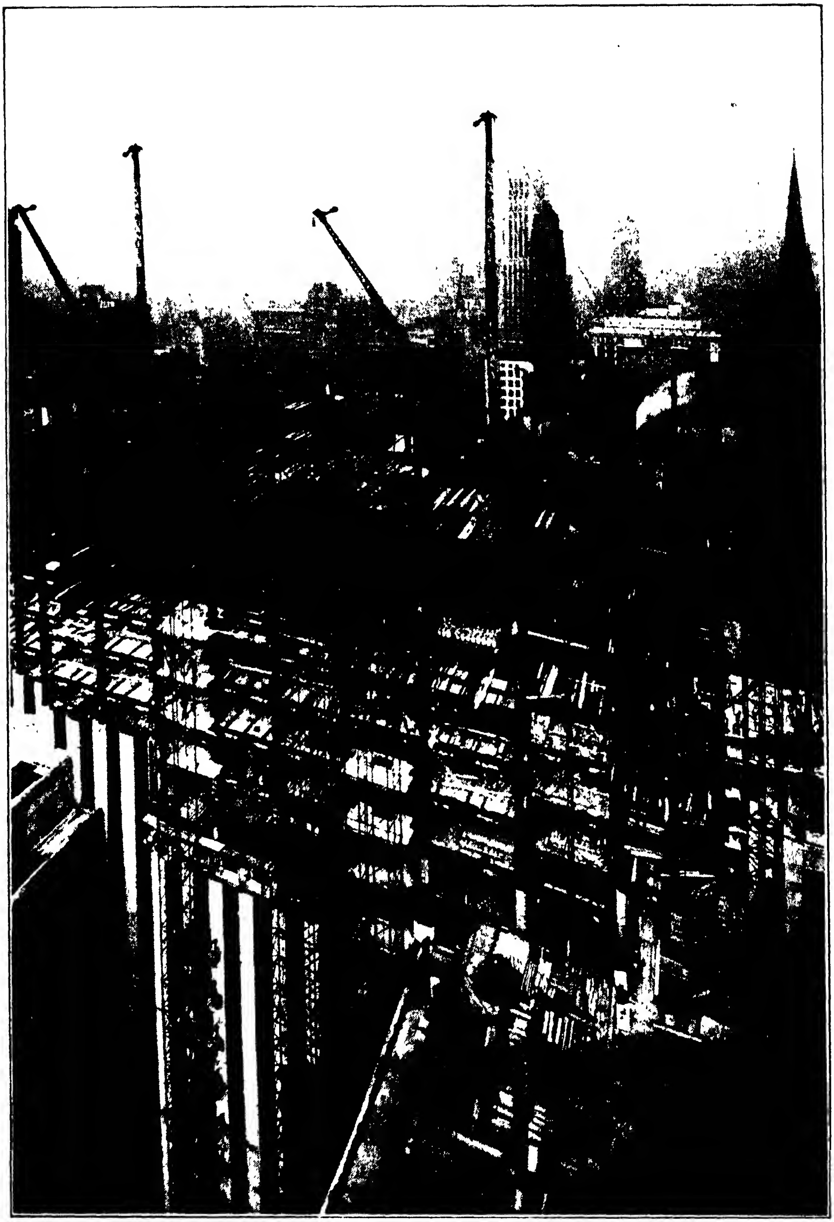


Fig. 169. Power House under Construction
Courtesy of A. T. & S. F. Railway System

4. Design the columns for a bent similar to that shown in Fig. 166. The spacing of the bents is 20 ft. Assume a span of 50 ft. instead of 41.5 ft.; also, take the height from base to underside of truss as 24 ft. Wind pressure is 20# per sq. ft. Vertical reaction from truss equals 50,000#.

5. Design the anchor bolts for the columns in Problem 4.



THE FIELD BUILDING OF CHICAGO DURING CONSTRUCTION
Courtesy of Graham, Anderson, Probst and White, Architects

CHAPTER XI

MULTISTORY BUILDINGS

A multistory steel building consists of tiers of floors supported on a steel skeleton frame composed of steel columns, girders, and beams. Whether the building is used for a hotel, hospital, office building, school, apartments, or warehouse, the method and problems of design are similar.

In this chapter various types of construction and problems of design encountered in steel skeleton buildings are treated, such as loads, floor construction, concrete encased steel design, spandrel sections, balcony construction, and wind-bracing analysis and connections. The practical problems of building design are also explained and illustrated for a steel frame school building.

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; however, machinery may cause loads in lateral directions. In addition to the gravity loads, the framework must resist wind loads and also seismic loads in earthquake territory.

Dead Loads. The dead loads in a building are the fixed or immovable loads and consist of the weight of walls, permanent partitions, framing, floors, roofs, and all other permanent construction.

Weights of Materials. 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

Material	Pounds
Book tile, 2 in. thick, per sq. ft.	12
Book tile, 3 in. thick, per sq. ft.	14
Beam tile (when not included with arch tile), per sq. ft.	12
Brick masonry, pressed or paving, per cu. ft.	140
Brick masonry, hard common, per cu. ft.	120
Brick masonry, hollow, per cu. ft.	90
Cast iron, bar 1 in. square, per lin. ft.	3 125
Cast iron, per cu. in.	26
Cinder concrete, per cu. ft.	96
Cinder fill (without sand and cement), per cu. ft.	72
Floors, marble, tutti colori, and similar, per sq. ft.	12
Floor flat arch (average of set), 8 in. thick, per sq. ft.	28
Floor flat arch (average of set), 10 in. thick, per sq. ft.	32
Floor flat arch (average of set), 12 in. thick, per sq. ft.	36
Floor flat arch (average of set), 14 in. thick, per sq. ft.	40
Floor flat arch (average of set), 16 in. thick, per sq. ft	46
Floor segmental arch tile (average of set), 6 in. thick at crown, per sq. ft.	28
Granite, per cu. ft.	160
Gypsum partition blocks, 3 in. thick, per sq. ft.	10
Gypsum partition blocks, 4 in. thick, per sq. ft.	12
Gypsum partition blocks, 5 in. thick, per sq. ft.	14
Gypsum partition blocks, 6 in. thick, per sq. ft.	16
Marble, per cu. ft.	175
Mortar and plaster, per cu. ft.	120
Mortar for tile arch floors, per sq. ft.	3
Oaks, maple, per ft. board measure	5
Ornamental terra cotta, backed and filled with common brick, per cu. ft.	120
Partition tile, 3 in. thick, per sq. ft.	14
Partition tile, 4 in. thick, per sq. ft.	15
Partition tile, 6 in. thick, per sq. ft.	22
Partition tile, 8 in. thick, per sq. ft.	28
Partition tile, 10 in. thick, per sq. ft.	32
Plaster on brick, concrete, tile or gypsum, per sq. ft.	5
Plaster on lath, per sq. ft.	7
Roofing, composition, per sq. ft.	5
Roofing gravel, per sq. ft.	10
Roofing slate, per sq. ft.	10
Roofing tile, per sq. ft.	10
Roofing, shingle, per sq. ft.	3
Sandstone or limestone rubble, per cu. ft.	140
Sandstone or limestone cut facing, per cu. ft.	150
Sheet metal roofing, cornice, etc., per sq. ft.	3
Suspended ceiling complete, per sq. ft.	10
Steel bar, 1 in. square, per lin. ft.	3 4
Steel plate, 1 in. thick, per sq. ft.	40.8
Stone concrete, per cu. ft.	144
Windows (glass, frames and sash), per sq. ft.	5
White pine, spruce, hemlock, per foot board measure	3
Yellow pine, fir, per ft. board measure	4

The following items may vary considerably in weight, but the values given may be used for preliminary computations, or when the quantities are small.

Material	Pounds
Concrete stair construction, per sq. ft.	150
Steel stair construction, per sq. ft.	50
Partition, tile plastered, per sq. ft.	25
Same in hotels, per sq. ft. of floor	35
Same in office buildings, per sq. ft. of floor	25
Reinforcement of concrete, per cu. ft.	6
Sidewalk lights in concrete, per sq. ft.	30
Steel joists, per sq. ft. of floor	6
Steel girders, per sq. ft. of floor	4
Total weight of reinforced concrete, per cu. ft.	150
Wood stair construction, per sq. ft.	20

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 Building Code Committee in its report "Minimum Live Loads Allowable for Use in Design of Buildings," published by the Bureau of Standards, 1924, recommends the following:

Sec. 3. Human Occupancy.

1. For rooms of private dwellings, hospital rooms and wards, guest rooms in hotels, lodging and tenement houses, and for similar occupancies, the minimum live load shall be taken as 40 pounds per square foot uniformly distributed, except that where floors of one and two family dwellings are of monolithic type or of solid or ribbed slabs the live load may be taken as 30 pounds per square foot.

2. For floors for office purposes and for rooms with fixed seats, as in churches, school classrooms, reading rooms, museums, art galleries, and theatres, the minimum live load shall be taken as 50 pounds per square foot uniformly distributed. Provision shall be made, however, in designing office floors for a load of 2,000 pounds placed upon any space $2\frac{1}{2}$ feet square wherever this load upon an otherwise unloaded floor would produce stresses greater than the 50-pound distributed load.

3. For aisles, corridors, lobbies, public spaces in hotels and public buildings, banquet rooms, assembly halls without fixed seats, grandstands, theater stages, gymnasiums, stairways, fire escapes or exit passageways, and other spaces where crowds of people are likely to assemble, the minimum live load shall be taken as 100 pounds per square foot uniformly distributed. This requirement shall not apply, however, to such spaces in private dwellings, for which the minimum live load shall be taken as in paragraph 1 of this section.

Sec. 4. Industrial or Commercial Occupancy.

In designing floors used for industrial or commercial purposes, or purposes other than previously mentioned, the live load shall be assumed as the maximum caused by the use which the building or part of the building is to serve. The following loads shall be taken as the minimum live loads permissible for the occupancies listed, and loads at least equal shall be assumed for uses similar in nature to those listed in this section.

Floors used for:	Minimum live load (lb./ft.)
Storage purposes (general)	250
Storage purposes (special)	100
Manufacturing (light)	75

Printing plants	100
Wholesale stores (light merchandise)	100
Retail salesrooms (light merchandise)	75
Stables	75
Garages—	
All types of vehicles	100
Passenger cars only	80

Sidewalks—250 or 8,000 pounds concentrated, whichever gives the largest moment of shear.

Sec. 5. Roof Loads.

Roofs having a rise of 4 inches or less per foot of horizontal projection shall be proportioned for a vertical live load of 30 pounds per square foot of horizontal projection applied to any or all slopes. With a rise of more than 4 inches and not more than 12 inches per foot a vertical live load of 20 pounds on the horizontal projection shall be assumed. If the rise exceeds 12 inches per foot no vertical live load need be assumed, but provision shall be made for a wind force acting normal to the roof surface (on one slope at a time) of 20 pounds per square foot of such surface.

Sec. 6. Allowance for Movable Partition Loads.

Floors in office and public buildings and in other buildings subject to shifting of partitions without reference to arrangement of floor beams or girders shall be designed to support, in addition to other loads, a single partition of the type used in the building, placed in any possible position.

Sec. 7. Reductions in Live Loads.

Except in buildings for storage purposes the following reductions in assumed total floor live loads are permissible in designing all columns, piers or walls, foundations, trusses, and girders.

	Per cent
Carrying one floor	0
Carrying two floors	10
Carrying three floors	20
Carrying four floors	30
Carrying five floors	40
Carrying six floors	45
Carrying seven or more floors	50

For determining the area of footings the full dead loads plus the live loads, with reductions figured as permitted above, shall be taken; except that in buildings for human occupancy, listed in section 3, a further reduction of one-half the live load as permitted above may be used.

Sec. 8. Wind Pressures.

For purposes of design the wind pressure upon all vertical plane surfaces of all buildings and structures shall be taken at not less than 10 pounds per square foot for those portions less than 40 feet above ground, and at not less than 20 pounds per square foot for those portions more than 40 feet above ground.

The wind pressure upon sprinkler tanks, sky signs, or upon similar exposed structures and their supports shall be taken at not less than 30 pounds per square foot of plane surface, acting in any direction. In calculating the wind pressure on circular tanks or stacks this pressure shall be assumed to act on six-tenths of the projected area.

Where it shall appear that a building or structure will be exposed to the full force of the wind throughout its entire height and width the pressure upon all vertical surfaces thus exposed shall be taken at not less than 20 pounds per square foot.

In Table VIII, are shown the recommended live loads for storage warehouses.

Most building codes allow the floor girders and columns in all buildings except warehouses to be designed for a reduced live load. For girders it is customary to reduce 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 until 50 per cent is reached; this final value is used for the remaining floors. This amount of allowed reduction varies slightly from that recommended by the Building Code Committee of the United States Department of Commerce.

Wind Loads. Specifications and building codes vary in regard to the amount of wind pressure that should be provided for. Requirements vary from 15 to 30 pounds per square foot. Many tests and experiments have been made to establish the relation between wind velocity and wind pressure. There is still a lack of information on wind velocities at higher elevations, and on the velocity and size of gusts of wind. In a preliminary report of the Sub-Committee of the Structural Division of the American Society of Civil Engineers issued in 1931, they proposed: "That the prescribed wind force for buildings for the first 500 feet of height be a pressure of 20 pounds per square foot, and that above this level it be increased at the rate of 2 pounds per square foot for each 100 feet of height." For buildings up to 100 feet in height, 15 pounds per square foot is satisfactory except under exceptional conditions. For buildings higher than 100 feet, local conditions should also be considered in deciding on the wind pressure to use.

The formula for the relation between wind pressure and velocity is

$$P = CV^2$$

TABLE VIII. Recommended Live Loads for Storage Warehouses

United States Department of Commerce, National Bureau of Standards

Material	Weight per Cubic Foot of Space Lb.	Height of Pile Feet	Weight per Square Foot of Floor Lb.	Recommended Live Load Lb. per Sq. Ft.
BUILDING MATERIALS				
Asbestos	50	6	300	
Bricks, building	45	6	270	
Bricks, fire clay	75	6	450	
Cement, natural	59	6	354	300
Cement, Portland	72 to 105	6	432 to 630	to
Gypsum	50	6	300	400
Lime and plaster	53	5	265	
Tiles	50	6	300	
Woods, bulk	45	6	270	
DRUGS, PAINTS, OIL, ETC.				
Alum, pearl, in barrels	33	6	198	
Bleaching powder, in hogsheads	31	3½	102	
Blue vitriol, in barrels	45	5	226	
Glycerine, in cases	52	6	312	
Linseed oil, in barrels	36	6	216	
Linseed oil, in iron drums	45	4	180	
Logwood extract, in boxes	70	5	350	
Rosin, in barrels	48	6	288	
Shellac, gum	38	6	228	200
Soaps	50	6	300	to
Soda ash, in hogsheads	62	2¾	167	300
Soda, caustic, in iron drums	88	3¾	294	
Soda, silicate, in barrels	53	6	318	
Sulphuric acid	60	1⅝	100	
Toilet articles	35	6	210	
Varnishes	55	6	330	
White lead paste, in cans	174	3½	610	
White lead, dry	86	4¾	408	
Red lead and litharge, dry	132	3¾	495	
DRY GOODS, COTTON, WOOL, ETC.				
Burlap, in bales	43	6	258	
Carpets and rugs	30	6	180	
Coir yarn, in bales	33	8	264	
Cotton, in bales, American	30	8	240	
Cotton, in bales, foreign	40	8	320	
Cotton bleached goods, in cases	28	8	224	
Cotton flannel, in cases	12	8	96	
Cotton sheeting, in cases	23	8	184	
Cotton yarn, in cases	25	8	200	
Excelsior, compressed	19	8	152	200
Hemp, Italian, compressed	22	8	176	to
Hemp, Manila, compressed	30	8	240	250
Jute, compressed	41	8	328	
Linen damask, in cases	50	5	250	
Linen goods, in cases	30	8	240	
Linen towels, in cases	40	6	240	
Silk and silk goods	45	8	360	
Sisal, compressed	21	8	168	
Tow, compressed	29	8	232	
Wool, in bales, compressed	48			
Wool, in bales, not compressed	13	8	104	
Wool, worsteds, in cases	27	8	216	
GROCERIES, WINES, LIQUORS, ETC.				
Beans, in bags	40	8	320	
Beverages	40	8	320	
Canned goods, in cases	58	6	348	
Cereals	45	8	360	
Cocoa	35	8	280	250
Coffee, roasted, in bags	33	8	264	to
Coffee, green, in bags	39	8	312	300
Dates, in cases	55	6	330	
Figs, in cases	74	5	370	
Flour, in barrels	40	5	200	
Fruits, fresh	35	8	280	
Meat and meat products	45	6	270	

TABLE VIII—Continued
Recommended Live Loads for Storage Warehouses

Materia	Weight per Cubic Foot of Space Lb.	Height of Pile Feet	Weight per Square Foot of Floor Lb.	Recom-mended Live Load Lb. per Sq. Ft.
GROCERIES, WINES, LIQUORS, ETC. Cont'd.				
Milk, condensed	50	6	300	
Molasses, in barrels	48	5	240	
Rice, in bags	58	6	348	
Sal soda, in barrels	46	5	230	
Salt, in bags	70	5	350	250
Soap powder, in cases	38	8	304	to
Starch, in barrels	25	6	150	300
Sugar, in barrels	43	5	215	
Sugar, in cases	51	6	306	
Tea, in chests	25	8	200	
Wines and liquors, in barrels	38	6	228	
HARDWARE, ETC.				
Automobile parts	40	8	320	
Chain	100	6	600	
Cutlery	45	8	360	
Door checks	45	6	270	
Electrical goods and machinery	40	8	320	
Hinges	64	6	384	
Locks, in cases, packed	31	6	186	
Machinery, light	20	8	160	
Plumbing, fixtures	30	8	240	300
Plumbing, supplies	55	6	330	to
Sash fasteners	48	6	288	400
Screws	101	6	606	
Shafting steel	125			
Sheet tin, in boxes	278	2	556	
Tools, small, metal	75	6	450	
Wire cables, on reels			425	
Wire, insulated copper, in coils	63	5	315	
Wire, galvanized iron, in coils	74	4½	333	
Wire, magnet, on spools	75	6	450	
MISCELLANEOUS				
Automobile tires	30	6	180	
Automobiles, uncrated	8		64	
Books (solidly packed)	65	6	390	
Furniture	20			
Glass and chinaware, in crates	40	8	320	
Hides and leather, in bales	20	8	160	
Hides, buffalo, in bundles	37	8	296	
Leather and leather goods	40	8	320	
Paper, newspaper, and strawboards	35	6	210	
Paper, writing and calendared	60	6	360	
Rope, in coils	32	6	192	
Rubber, crude	50	8	400	
Tobacco, bales	35	8	280	

P equals the wind pressure in pounds per square foot. V equals the actual wind velocity in miles per hour. C is a constant, and various investigators have found different values for it. G. Eiffel, experimenting in Paris, determined a C value of 0.0033. The Bureau of Standards, Washington, conducted tests in a wind tunnel in 1926 and found a C value of 0.00255.

Seismic Loads. Sometimes earthquake forces have been provided for by designing for an increased wind load. This is not correct, as the action is not the same under these loads; also, earthquake forces are proportional to the weight of the building and not to the exposed area.

As the result of the studies of many investigators, it has been agreed that the horizontal earthquake forces seldom exceed those produced by horizontal static forces equal to $\frac{1}{10}$ of the gravity loads. The following formula is used in determining the horizontal earthquake force.

$$F = CW$$

In this formula F equals the horizontal force in pounds. Except for warehouses, W equals the total dead load plus one half of the total vertical designed live load at and above the elevation under consideration. For warehouses, W equals the total dead load plus the total vertical live load at and above the point or elevation under consideration. Machinery or other fixed concentrated loads are considered as a part of the dead load. C in the formula is a numerical constant and varies with the type of construction, foundation conditions, and the susceptibility of the locality to seismic disturbances. For buildings as a whole, C may vary from 0.02 to 0.16, while for specific parts of building structures, it may range from 0.05 to a maximum of 1.0.

Observations of earthquake action have shown that the earth movements and resulting lateral forces may occur in any direction. Investigations also show that structures located on rock or hard ground are subject to smaller forces than those located on a softer material.

A typical building code requires that the following seismic forces be provided for. The stresses shall be computed for a force applied horizontally at the center of gravity of the dead load. The force shall be assumed to come from any possible direction at right angles to any elevation of the building. When the foundation rests upon a material for which 4 tons per square foot is allowed, the horizontal force applied at the center of gravity shall be assumed as 10 per cent of the load on the columns. When the foundations rest on material good for 3 tons per square foot, the lateral force shall be taken as 15 per cent of the load on the columns. When the foundations rest on material with a 2-ton bearing capacity, the lateral force is assumed as 20 per cent of the load on the columns.

When earthquake forces are included, it is customary to allow an increase in unit stress of 25 or $33\frac{1}{3}$ per cent.

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.

FLOOR CONSTRUCTION

There are several types of floor construction in common use in steel skeleton buildings. Some of the factors influencing the selection

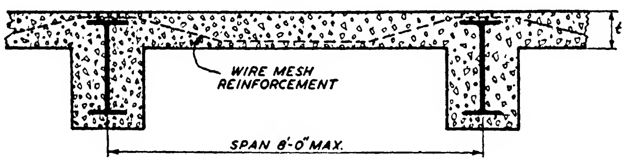


Fig. 170. Typical Construction for Cinder Concrete Arch Slab

of a type of floor are: size of job, weight of construction, availability of materials, degree to which it is fireproof, space for conduits and piping, speed of construction, weather conditions, and local labor conditions. The most commonly used types will be discussed and illustrated.

Cinder Concrete Arch or Slab Floors. A widely used type of floor construction, especially in the New York City area, is the reinforced cinder concrete arch or slab. These slabs are usually made about 4 inches in thickness and with spans not over 8 feet between structural steel framing. In Fig. 170 is shown a typical section of cinder concrete floor construction. The aggregate is hard-coal cinders. This reinforced cinder concrete weighs 108 pounds per cubic foot. The reinforcement is usually wire mesh and is laid or draped over the steel joist supports.

There are other light-weight aggregates and compounds, besides cinders, that can be used to secure light-weight concrete.

The advantages of this type of floor construction are as follows: The light weight reduces the size of the steel members, especially the columns, in a tall multistory building. The forms are hung from the steel beams, which permits speedy erection. The formwork is easy to strip and reuse. The reinforcing and concrete are easily placed, resulting in a low installation cost. Practically all the conduits and outlet boxes can be placed on the forms and embedded in the slab, or, when cinder fill is used on top of the slab, the conduits can be placed in the fill.

Flat Arch Clay Tile Floors. The hollow tile arch floor was one of the first types of floor construction used for steel skeleton buildings. The segmental or curved arch floor is no longer used a great deal, as the flat arch can be made of sufficient strength, is more easily erected, and gives a flat ceiling.

All forms of tile arches produce side thrusts on the steel joists. It is necessary to counterbalance this thrust by the use of tie rods.

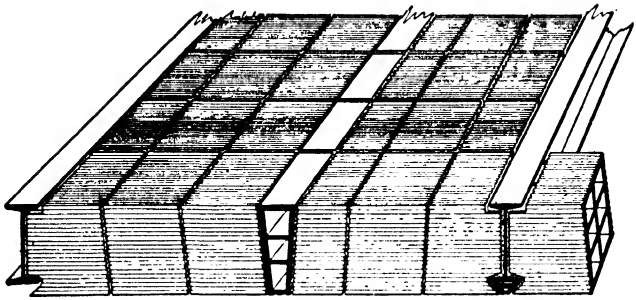
In Fig. 171 are shown various types of flat arch construction. In the flat arch the tile blocks have tapered faces and the central block, or key, wedges the others together. A thin slab of concrete in which conduits can be placed is usually poured on top of the tile. The wearing surface is then placed directly on this slab, or, if the floor is to be of wood, cinder fill and nailing strips are used.

In Table IX are shown the load capacities for various depths and spans of flat tile arch floors. The depth of arch as given in this table includes the thickness of the soffit tile underneath the bottom of the joists. If a raised floor arch, giving a paneled ceiling effect, is used, the thickness of arch can be made approximately 2 inches less.

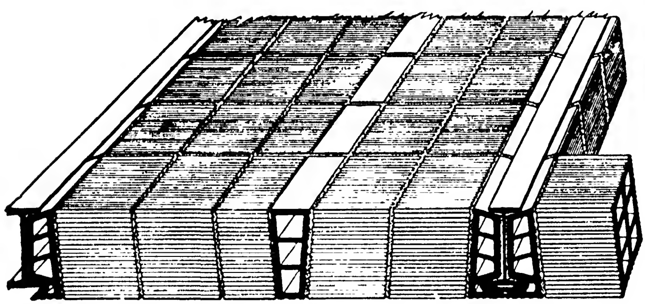
Building codes usually require that the depth of a flat arch be made $1\frac{1}{2}$ inches for each foot of span, plus the thickness of the protection below the joists.

For flat arches, $\frac{3}{4}$-inch rods are usually placed at the third points of the joists. A common rule is to place the tie rods no farther apart than 15 times the width of the joist flanges. The following formula is used for computing the thrust, and from this the size of the rod can be determined.

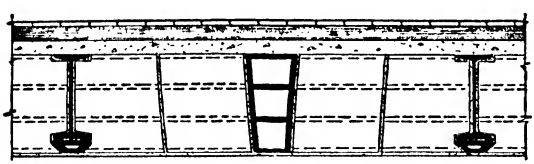
$$T=\frac{3wD^2}{2(R-2.4)}$$



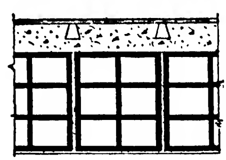
(A) Typical End Construction Arch. Extremely adaptable, as the skew can be cut to fit different elevations and sizes of beams



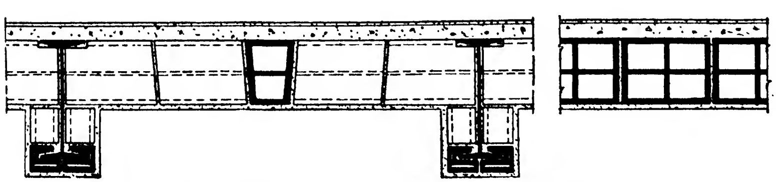
(B) Typical Combination Side and End Construction Arch The reverse direction of cells in the skews gives better protection to the beams



(C) Typical Side Elevation, End Construction Arch



(D) Typical End Elevation Showing Standard Unit



(E) Raised Floor Arch for Paneled Ceiling Effect

Fig. 171. Tile Flat Arch Floor Construction
Courtesy of National Fireproofing Corporation

TABLE IX

Safe Loads Per Sq. Ft. (Dead and Live) for Tile Flat Arch Floors

Factor of Safety of 7

Depth of arch	6 in.	7 in.	8 in.	9 in.	10 in.	12 in.	15 in.
Net sectional areas	27 sq. in.	27 sq. in.	27 sq. in.	27 sq. in.	36 sq. in.	36 sq. in.	36 sq. in.
Average wt. per sq. ft.	26 lb.	30 lb.	32 lb.	36 lb.	40 lb.	48 lb.	56 lb.
Span—ft. and in.	lb.	lb.	lb.	lb.	lb.	lb.	lb.
3–0	420	490	560	630	933	1120	1400
3–3	357	417	477	537	795	954	1193
3–6	308	360	411	462	685	823	1028
3–9	268	313	358	403	597	716	895
4–0	236	276	315	354	525	630	786
4–3	....	244	279	314	465	558	697
4–6	....	218	249	279	415	497	622
4–9	....	....	223	251	372	447	558
5–0	....	....	201	227	336	402	504
5–3	....	....	182	205	305	365	457
5–6	....	....	....	187	277	333	417
5–9	....	....	....	171	254	305	381
6–0	....	....	....	157	233	280	350
6–3	....	....	....	....	214	258	322
6–6	....	....	....	....	198	238	298
6–9	....	....	....	....	....	221	276
7–0	....	....	....	....	....	206	257
7–6	....	....	....	....	....	178	223
8–0	....	....	....	....	....	157	197
8–6	....	....	....	....	....	....	174
9–0	....	....	....	....	....	....	155
9–6	....	....	....	....	....	....	140
10–0	....	....	....	....	....	....	126

Courtesy of National Fireproofing Corporation

D = distance of arch span in ft. w = unit load on arch in pounds per sq. ft. R = effective rise of arch in inches.

Flat arch floors can be erected from centering hung from the beams. This type of floor is relatively light in weight and can be constructed quickly. The all-tile ceiling provides a good surface for plastering.

Concrete Joists and Slab Floors. Another common type of floor construction is illustrated in Fig. 172 and consists of concrete joists and slab with metal pan forms or hollow tile fillers. The tile blocks bond to the concrete and add to the strength of the slab and joists.

If it is necessary to provide for heavy loads or a shallow floor, the joists can be run in both directions. When metal pans are used, the plaster is placed on metal lath. The plaster is applied directly on the under side when tile fillers are used.

An objection raised to this floor with tile filler is that in some cases the ceiling has, after a few years, taken on a ribbed appearance where the concrete joists show through. It has been found, however, that the plastering will not stain if the floor is thoroughly dried before plaster is applied.

If a ceiling with a tile surface over the entire area is required, construction as shown in Fig. 173 can be used. At (*A*) is shown a per-

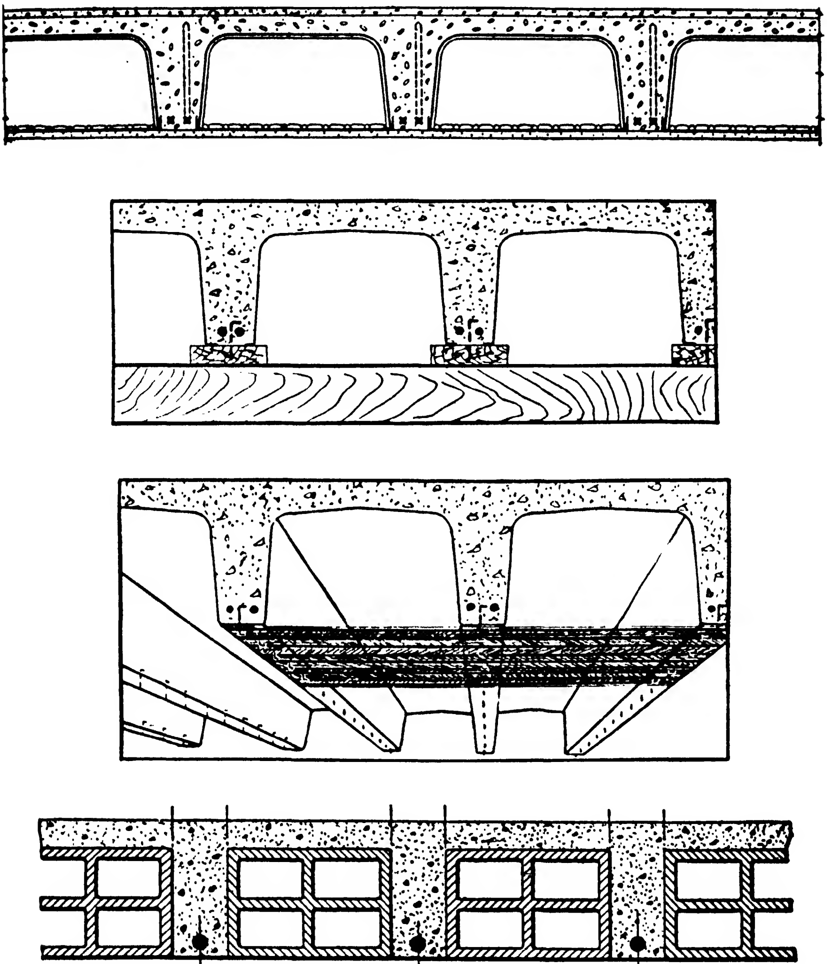
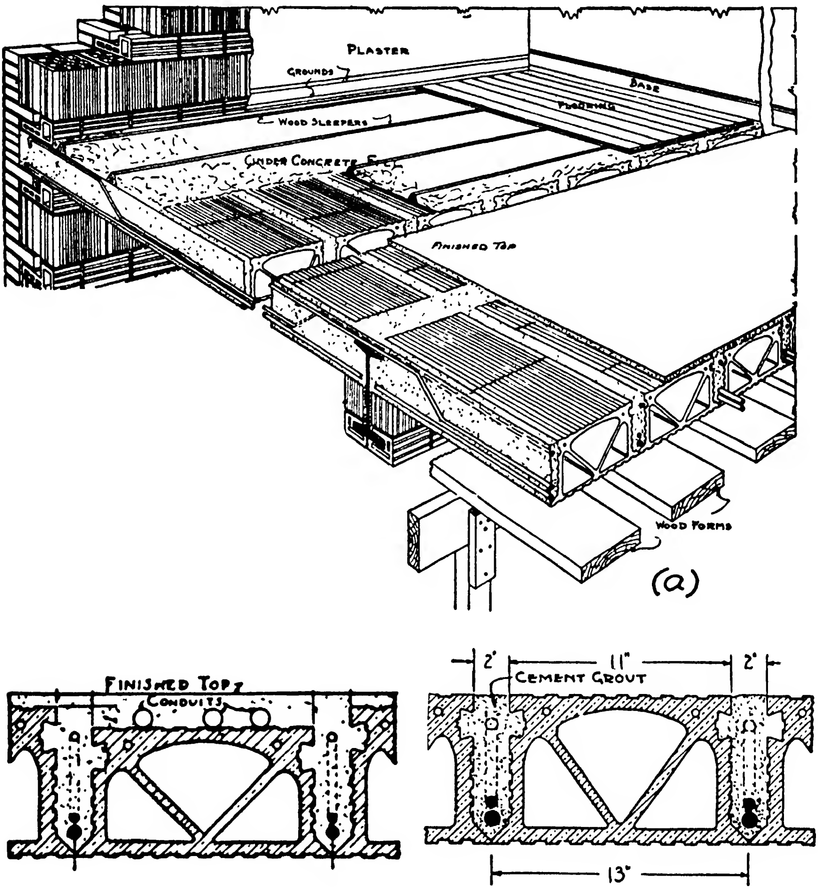


Fig. 172. Floor Constructions of Concrete Slab and Joists

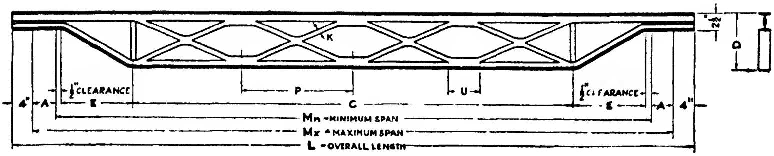
spective view of Natcoflor System. Part of the finished floor is wood, the other part cement, to show the method of laying. The wall is Natco Header Backer. When cement or terrazzo floors are used, a run for conduit may easily be made by putting in a series of tile less in depth than the rest of the floor, illustrated at (*B*). In the case of wood floors, the conduit may be run between the sleepers. Notice in the detail view at (*C*) how the tile meet at the bottom to form an all-tile ceiling.



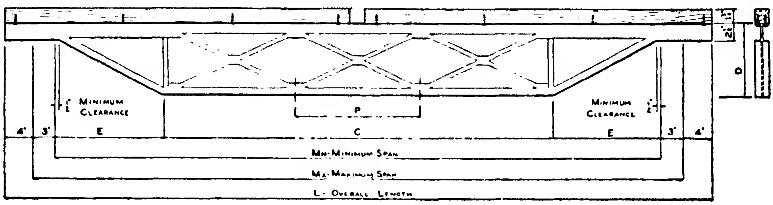
(B) (C)

Fig. 173. Tile and Concrete Joist Floor Construction
Courtesy of National Fireproofing Corporation

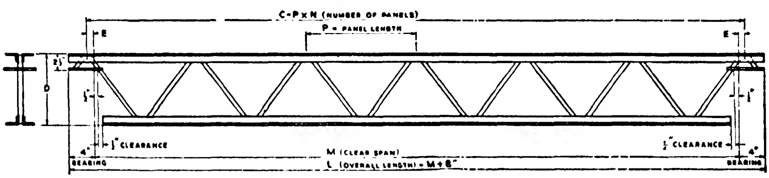
Floors with Open Web or Trussed Steel Joists. A light weight, fireproof floor can be made with the use of open web steel joists. In Fig. 174 are illustrated several types of these trussed joists. They span from girder to girder and are spaced 12 to 30 inches on centers, varying with the load to be carried. The connections to the girders are made by the use of clips or by welding. On top of the joists a



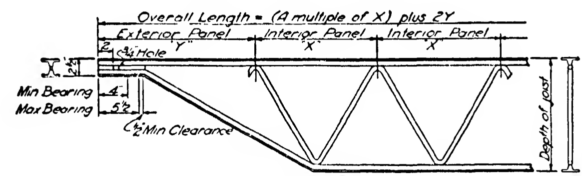
Bethlehem (Kalman) Steel Joist



The Expanded "Bates-X" Nailer Joist



Bethlehem (MacMar) Steel Joist



Truscon "O-T" Open Truss Steel Joist

Fig. 174. Open Web or Trussed Steel Joists

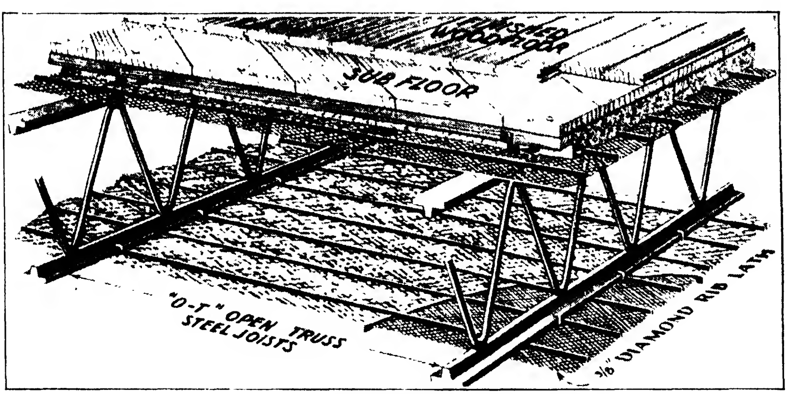


Fig. 175. Steel Joists with Finished Wood Floor

Courtesy of Truscon Steel Company

ribbed metal lath is placed, on which a 2 to 3-inch concrete slab is poured. The floor finish can be made either cement, wood, or composition.

In Fig. 175 is shown one type of wood floor on top of open truss steel joists. Bridging should be used between these joists, using one row for 15-foot spans, two rows for 20-foot spans, and three rows for 30-foot spans.

In Table X are tabulated the properties and allowed loads per linear foot for Steel Joist Institute Standard open web steel joists.

The Steel Joist Institute lists the following advantages of steel joists.

1. Steel joists are completely standardized as to lengths, depths and carrying capacities and standard load tables are readily available to the engineer. Steel joist floors are thereby more quickly, easily and accurately designed.

2. Steel joists are completely fabricated in the shop. The bearing ends are made $2\frac{1}{2}$ inches deep in all cases. The point of support is thereby raised above the center of gravity of the truss and joists will therefore remain upright and not overturn when set in place.

3. When joists rest on brick walls, the $2\frac{1}{2}$-inch depth of bearing ends fit between two successive mortar joints, causing no interference with the brick layout.

4. When the supports are at equal levels, the tops of all joists, regardless of depth of joist, lie in the same horizontal plane, thereby providing level bearing surfaces for floors and roof decks.

5. Steel joists are made in standard depths of 8, 10, 12, 14 and 16 inches and in lengths to accommodate all spans up to 32 feet. They reach the job site tagged and ready for immediate placing, needing only to be hoisted into their proper place and to be attached to supporting members. Highly skilled and expert labor is not necessary. Field costs are thereby greatly reduced.

6. Each joist is a complete, stable and independent unit. As many portions of the floors as desired may be erected simultaneously. Field work can be speeded up to high degree and the building completed in shorter time.

7. As soon as joists are erected and bridged, a working platform is available for the immediate follow-up of allied trades, allowing the work to progress more efficiently.

8. The open webs in the joists permit the ready concealment of pipes and electric conduits within the depth of floor, thereby making such installations extremely economical.

Steel Floors. Only in recent years have steel floors been used in buildings. This type of floor consists of either plane steel plates $\frac{3}{16}$ to $\frac{1}{2}$ inch in thickness, or deformed steel sheet sections. The plane plate floor is called *battledeck* construction. In Fig. 176 this type of floor is illustrated.

TABLE X. Allowed Loads for Open Web Steel Joists

STEEL JOIST INSTITUTE STANDARD

Properties and Allowable Total Loads in Pounds per Linear Foot

American Institute of Steel Construction Stresses.
Adopted by Steel Joint Institute Aug. 20, 1929. Effective Jan. 1, 1930.
In accordance with Simplified Practice Recommendation No. R 94-30 of the Bureau of Standards, U. S. Dept. of Commerce.

Steel Joist Institute Designation	SJ81	SJ82	SJ102	SJ103	SJ104	SJ123	SJ124	SJ125	SJ126	SJ145	SJ146	* SJ147	SJ166	* SJ167
Depth in Inches	8	8	10	10	10	12	12	12	12	14	14	14	16	16
Resisting Moment in Inch Pounds	29,500	52,500	63,000	82,000	100,000	92,000	115,000	142,000	175,000	156,000	205,000	246,000	232,000	281,000
Maximum End Reaction in Pounds	1600	1900	1900	1950	2200	2200	2300	2500	2700	2900	3100	3400	3200	3600
Span 4 Feet	800	...	...	...	...	...	...	...	...	...	...	...	...	...
5	640	...	...	...	...	...	...	...	...	...	...	...	...	...
6	530	...	...	...	...	...	...	...	...	...	...	...	...	...
7	402	...	...	...	...	...	...	...	...	...	...	...	...	...
8	308	...	...	...	...	...	...	...	...	...	...	...	...	...
9	243	...	...	...	...	...	...	...	...	...	...	...	...	...
10	197	350	...	...	...	...	...	...	...	...	...	...	...	...
11	162	289	...	...	...	...	...	...	...	...	...	...	...	...
12	137	243	292	...	...	...	...	...	...	...	...	...	...	...
13	116	207	248	...	...	...	...	...	...	...	...	...	...	...
14	100	178	214	...	...	...	...	...	...	...	...	...	...	...
15	87	155	187	243	...	272	...	...	...	...	...	...	...	...
16	77	137	164	213	260	240	...	...	...	...	...	...	...	...
17	...	...	145	189	230	212	...	...	...	...	...	...	...	...
18	...	...	130	169	205	189	236	...	...	...	...	...	...	...
19	...	...	116	151	184	170	212	...	...	288	...	...	...	...
20	...	...	105	137	167	153	192	237	...	260	...	...	...	...
21	...	...	...	...	...	139	174	215	...	236	...	...	...	...
22	...	...	...	...	...	127	158	196	241	215	...	...	...	...
23	...	...	...	...	...	116	145	179	221	197	258	...	...	...
24	...	...	...	...	...	106	133	164	202	180	237	...	...	...
25	...	...	...	...	...	...	...	...	...	166	218	262	247	...
26	...	...	...	...	...	...	...	...	...	154	202	243	229	...
27	...	...	...	...	...	...	...	...	...	143	187	225	212	257
28	...	...	...	...	...	...	...	...	...	133	174	209	197	239
29	...	...	...	...	...	...	...	...	...	...	...	...	184	223
30	...	...	...	...	...	...	...	...	...	...	...	...	172	208
31	...	...	...	...	...	...	...	...	...	...	...	...	161	195
32	...	...	...	...	...	...	...	...	...	...	...	...	151	183

*Note. These joists are not produced by all manufacturers.

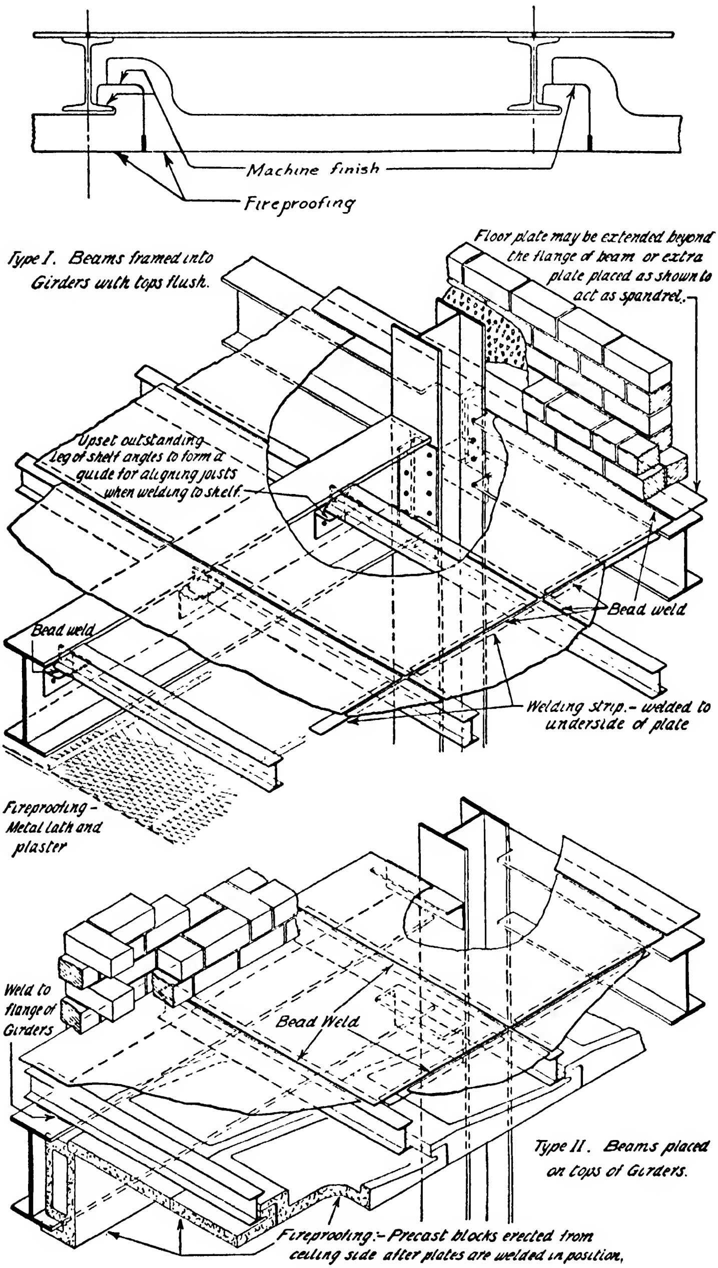


Fig. 176. Battledeck Steel Floor Construction
Courtesy of American Institute of Steel Construction

Battledeck floor construction consists of rolled beams, of whatever depth required, spaced about 24 inches apart and spanning from girder to girder of the building frame. Along the top of these joists are laid steel plates which are welded to the joists by a continuous bead directly over the web of the joists. The result is a built-up **T** section with the plate acting as compression flange and the beam as the vertical part of the **T**.

Fig. 177. Battledeck Floor in an Office Building
Courtesy of Lincoln Electric Company, Cleveland, Ohio

The A.I.S.C. has issued a bulletin explaining and illustrating this type of floor construction. Tables are also given showing the sections required for various span lengths and loads.

Fig. 177 shows a battledeck floor in an office building.

Some types of deformed sheet steel floors are illustrated in Fig. 178. These floors are prefabricated in widths of 24 inches and in any desired length. The units span from girder to girder and are connected by clips or by welding. Any desired type of finished floor surface can be used on top. These floors are quickly and easily assembled, and provide a safe floor for other trades as soon as laid in place. In Fig. 179 workmen are shown laying a prefabricated type of steel floor.

Steel floors are the lightest type of floor construction, and the cost is comparable to other commonly used types. Battledeck and

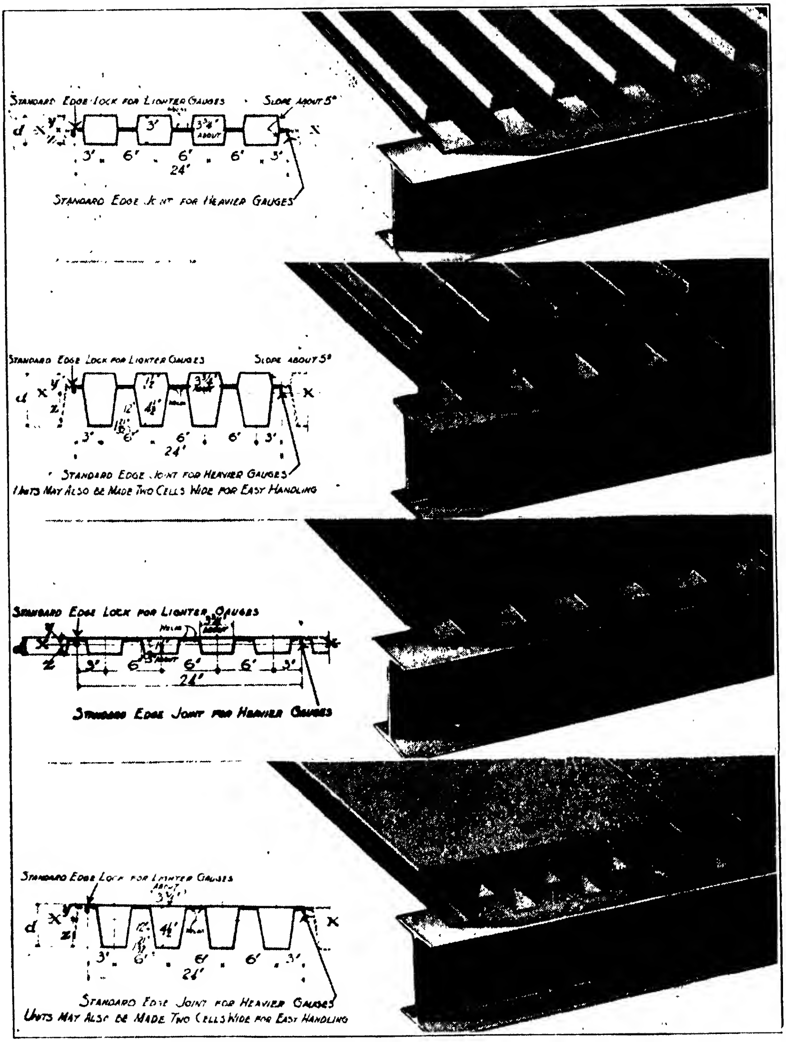


Fig. 178. Prefabricated Keystone Beam Steel Floors
Courtesy of H. H. Robertson Company

prefabricated steel floor, when welded to the girders, serves also as a stiff diaphragm to prevent distortion of a building when subjected to wind or earthquake forces.

CONCRETE ENCASED CONSTRUCTION

When concrete, or combination tile and concrete floors are used, it is desirable to fireproof the beams and columns with concrete. Fireproofing with concrete is also used to some extent with other types of floors. Tests have shown that when beams and columns are encased in concrete the strength and stiffness are greatly increased. A good many building codes allow the use of an increased steel stress for an encased beam or allow the composite beam to be computed as a reinforced concrete beam.

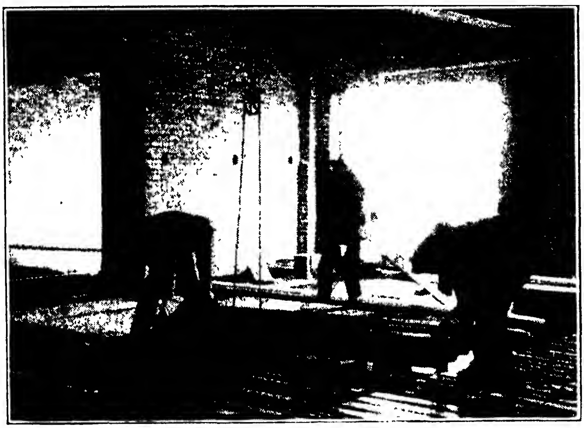


Fig. 179. Laying Prefabricated Type of Floor
Courtesy of H. H. Robertson Company

A committee of the Western Society of Engineers in cooperation with the Chicago building department reported in 1930 on a series of tests on floor construction of this type. The committee's explanation and recommendations are quoted as follows:

In floor systems composed of reinforced-concrete slabs carried by structural steel beams incased in concrete monolithic with the slab, little or no credit is given this incasement from a load-bearing standpoint. Fireproofing and weatherproofing the steel are assumed to be the only functions of such covering. On the other hand, structural steel columns incased in concrete are more commonly designed with higher unit stresses than those not incased. Many concerned with the design, construction or testing of floor beams have recognized that concrete incasement not only stiffens the beam but, by acting with the steel and concrete slab, actually carries a substantial portion of the load, or, in other words, reduces the stresses calculated in the steel. To verify this and to determine the relative effect of the concrete on the working stresses in the steel was the purpose of this floor test.

Our committee recommends unanimously:

1. That with materials of the quality used in this test, 24,000 lb. per square inch be used as a working unit stress in designing structural steel I beams up to 24 inches in depth when wrapped in steel wire fabric or mesh and incased in a rectangle of concrete with a minimum covering of 2 inches on the flanges of the I beams and when used in connection with a concrete floor slab not less than $2\frac{1}{2}$ inches thick above the top flanges of the I beams.

2. That concrete of the quality used in this test may be used in slabs 4 inches thick and reinforced with continuous steel fabric or mesh in spans of not to exceed 8 feet in accordance with the following formulas, in which W = total load, in pounds per square foot; D = depth to steel, in inches; A = area of steel, in square inches per foot of width, and L = span in feet.

$$\text{Cinder concrete} \ldots\ldots\ldots\ldots W = 26{,}000\frac{DA}{L^2}$$

$$\text{Gravel or stone concrete} \ldots W = 30{,}000\frac{DA}{L^2}$$

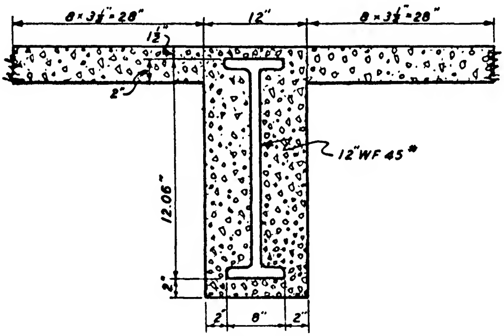


Fig 180. Composite Beam

From a review of these tests and earlier investigations, the following facts are typical: (1) The concrete and structural steel act together to form a composite beam. (2) The influence of the concrete prevails under vibration or impact. (3) Observed stresses and deflections in the steel are far below those computed on the assumption that the entire load is taken by the steel alone. (4) A considerable economy in the support of live loads may be obtained by utilizing the joint action of the concrete and steel. (5) Within practical limits, the composite beam conforms to theory and may be designed by methods analogous to those used in the case of a reinforced-concrete T beam. (6) The added strength due to the concrete encasement may be taken advantage of in the design by the simple method of adopting higher unit

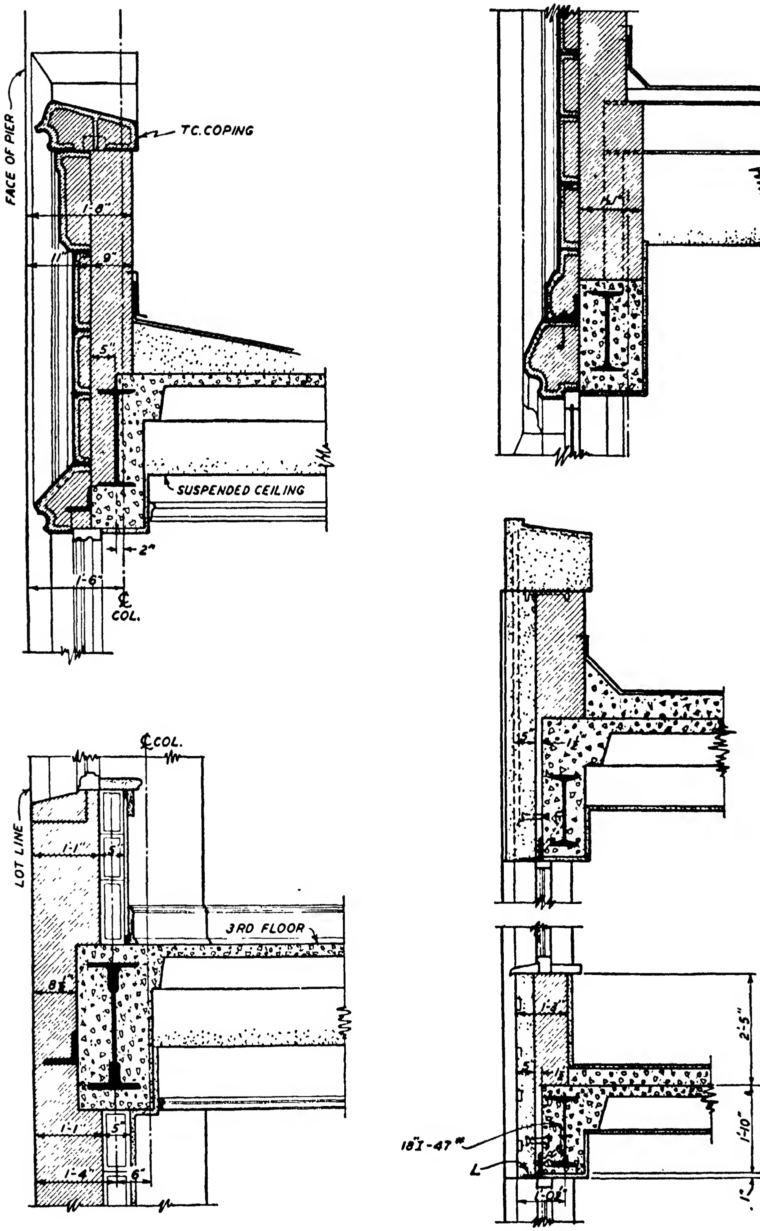


Fig. 181. Typical Spandrel Sections

working stresses in the steel and then, with these stresses, calculating the steel alone.

An encased steel beam as shown in Fig. 180 was computed as a composite beam in accordance with the requirements of the A.C.I. Code. The resisting moment of the combined beam was governed by the 20,000# per square inch tensile stress in the steel and equaled 1,310,000″#.

To secure the same resisting moment as the composite beam, the steel beam alone would have to be stressed to 22,650# per square inch.

If a beam is encased so that no flange or **T** section is formed, no advantage in strength results when figured as a composite beam, as the concrete stress in compression governs.

For encased beams and columns the Chicago Building Code allows the following:

Composite Beams: Where structural steel members are encased in concrete beams, and protected by not less than the minimum thickness of concrete specified elsewhere in this Code for the fireproofing of structural steel, the resulting composite beam may be computed as a reinforced concrete beam in accordance with the assumptions stated in Section 601 a, American Concrete Institute Code. The properties of Sections of composite beams shall be computed on the basis of the transformed area of steel.

c) *Combination Columns.* Structural steel columns of any rolled or built-up section wrapped with the equivalent of No. 8 W. & M. Standard gauge wire spaced four (4) inches on centers and encased in concrete not less than two and one-half ($2\frac{1}{2}$) inches thick over all of the metal, except rivet heads and connections, will be permitted to carry a load equal to $1 + \frac{A_c}{100A_s}$ times the permissible load for unencased steel columns.

A_c = Area of concrete
A_s = Area of steel

SPANDREL SECTIONS

In skeleton construction it is required that the walls be independently supported on the skeleton frame at each floor. This necessitates that the portion of the masonry walls outside of the spandrel girders be carried on angle supports connected to the spandrel girders.

On buildings having elaborate facades, many special details must be designed to support the masonry. In Figs. 181 and 182 some typical spandrel sections are illustrated. Some of these sections show the method used for connecting the shelf angle supports to the spandrel girders. The shelf angles extending alongside of the girder are sup-

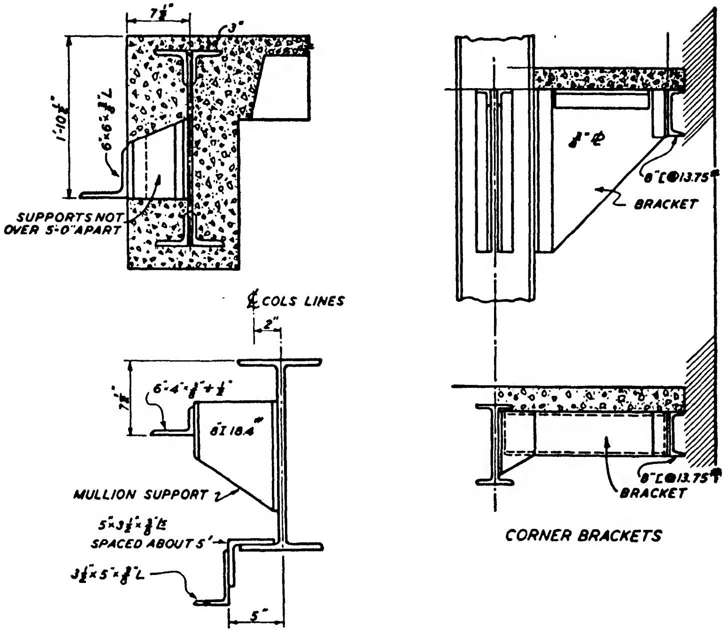


Fig. 182. Structural Details for Typical Spandrel Sections

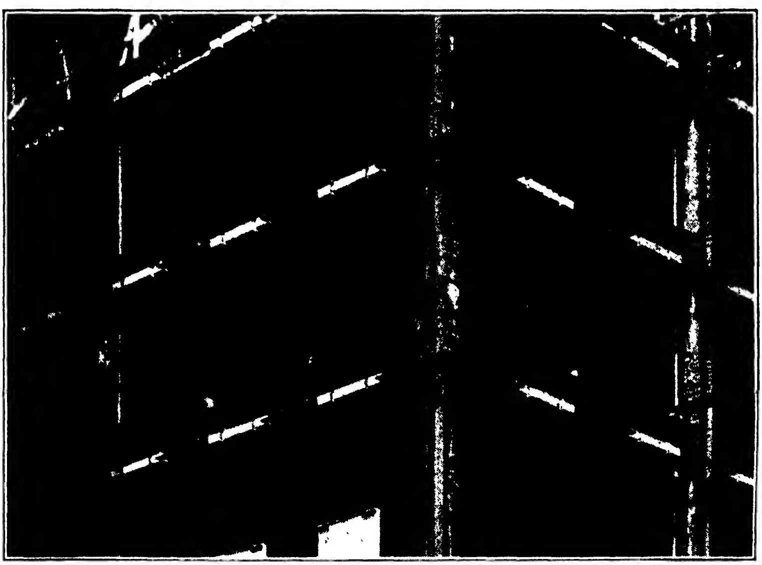


Fig. 183. Shelf Angle Supports on Spandrel Beams and Columns
Courtesy of Graham, Anderson, Probst and White, Architects

ported and connected to the girder at intervals of about five feet. These shelf angles are designed as beams between their supports, and the outstanding leg is figured as a cantilever. In Fig. 183 are shown the shelf angle supports for the masonry facing. They are placed on all spandrel beams and columns.

At all floor levels provision must also be made for supporting the masonry across the face of the column. In Fig. 184 are shown some typical shelf angle supports at columns.

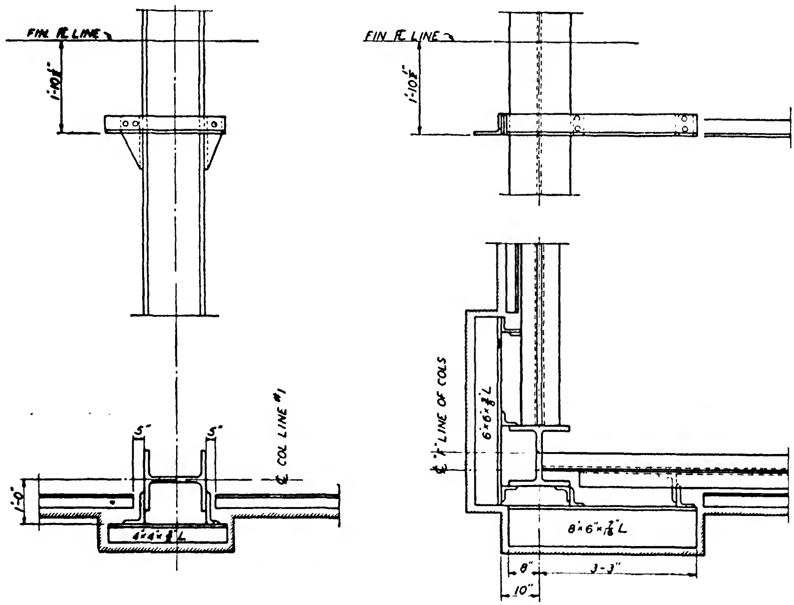


Fig. 184. Typical Shelf Angle Supports at Columns

Where terra cotta is used, anchor holes are required in the shelf angles. The necessary data in regard to the required holes can be secured from the terra cotta manufacturer.

Lintels. Usually the spandrel girders can be located so that they serve as lintels over the windows. Shelf angles or plates connected to the spandrel girders support the outside course of brick, stone, or terra cotta over these openings. When the windows are not high enough for the above type of detail, loose or detached angle lintels are used.

In Fig. 185 is shown a shop detail drawing of a spandrel girder. The wind-bracing brackets and shelf angles for supporting the architectural facing are detailed.

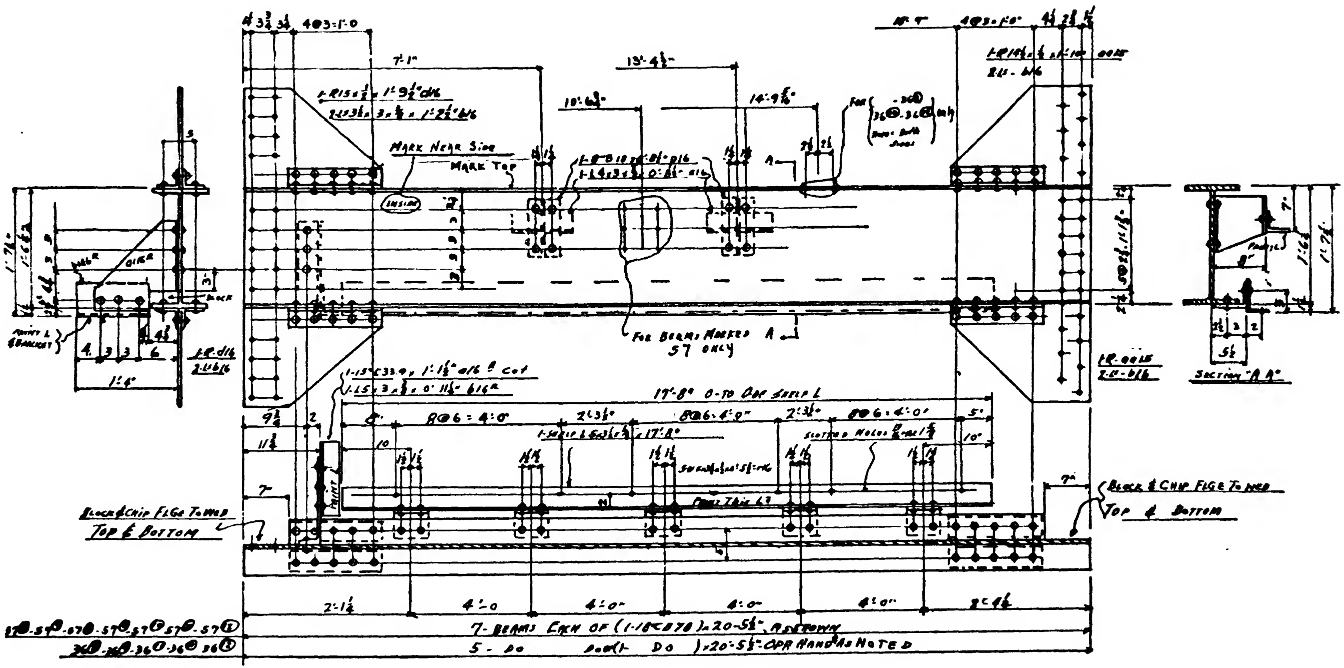


Fig. 185. Shop Detail Drawing of a Spandrel Girder

BALCONY CONSTRUCTION

The principal structural problem in balcony design is to provide framing to accommodate the required seating capacity and at the

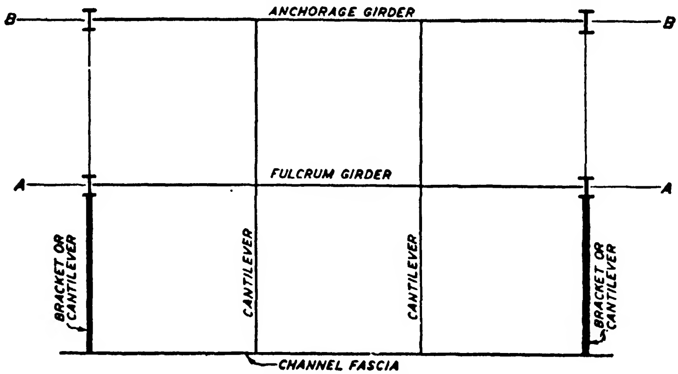


Fig 186. Framing for Balcony

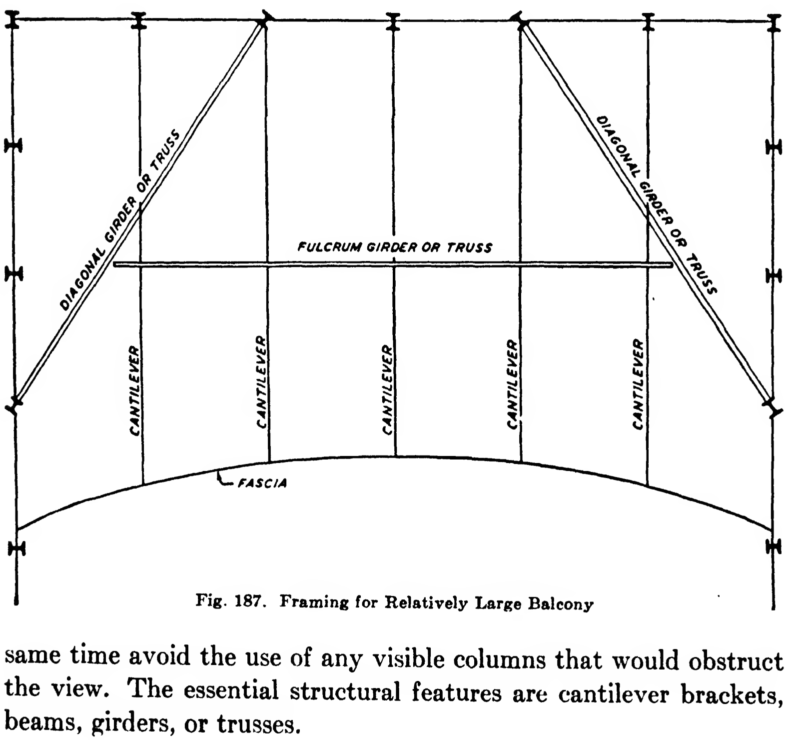


Fig. 187. Framing for Relatively Large Balcony

same time avoid the use of any visible columns that would obstruct the view. The essential structural features are cantilever brackets, beams, girders, or trusses.

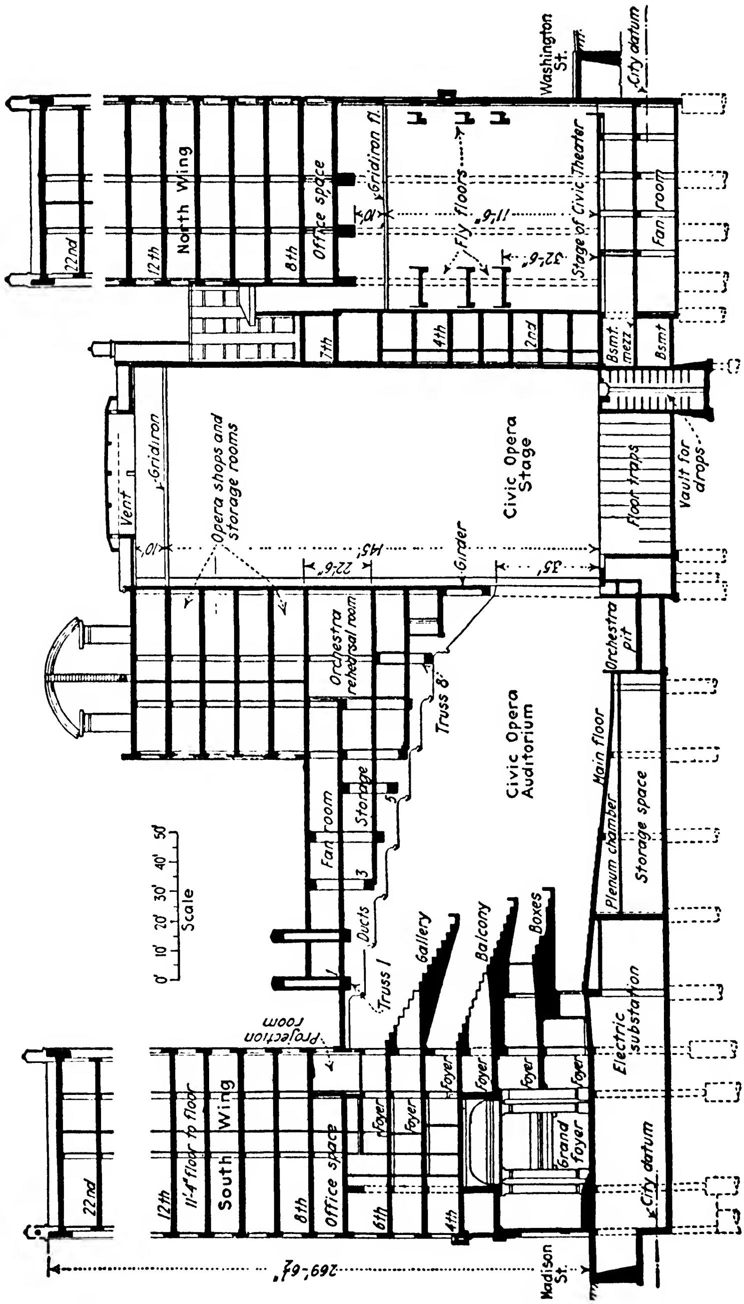


Fig. 188. Section through Chicago Civic Opera House
Courtesy of Graham, Anderson, Probst, and White, Architects

Brackets. Where it is not possible to use a cantilever beam or truss, or where the projection is not great, a bracket can be used. A bracket beam or truss is usually connected to a column which takes the bending. A cantilever bracket truss is shown in Fig. 229B. If a bracket is attached to a girder, provision should be made for the torsional stress and deflection. Usually when a bracket frames into a girder, beams or brackets can be placed on the opposite side of the

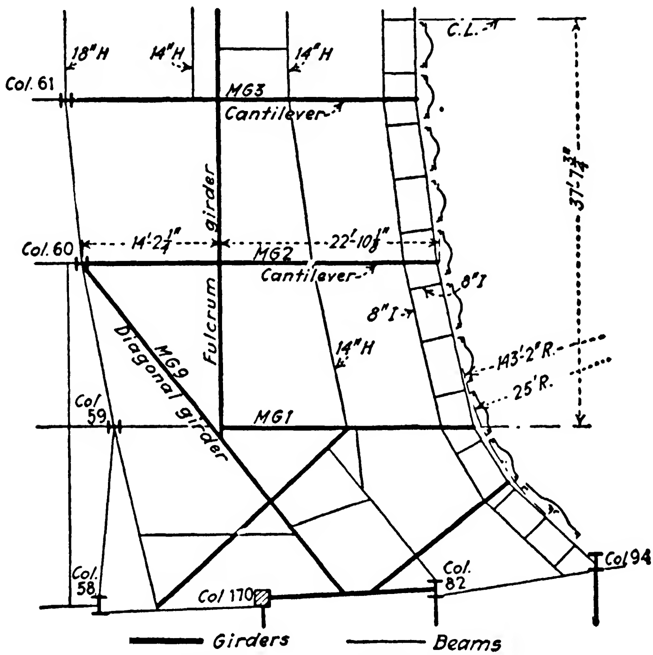


Fig. 189. Balcony Framing, Chicago Civic Opera House
Courtesy of Graham, Anderson, Probst, and White, Architects

girder and extend back to an anchorage. With this framing the result is equivalent to a cantilever beam, causing only a vertical reaction on the supporting girder.

Cantilever Beams. The principles of a cantilever beam are discussed in the chapter on Beams. The overhanging portion causes a maximum moment and shear at the fulcrum support. If the anchorage portion of the beam is long or has heavy enough loads, there may be no uplift at the anchorage.

In Fig. 186 is shown a simple framing plan for a balcony. The

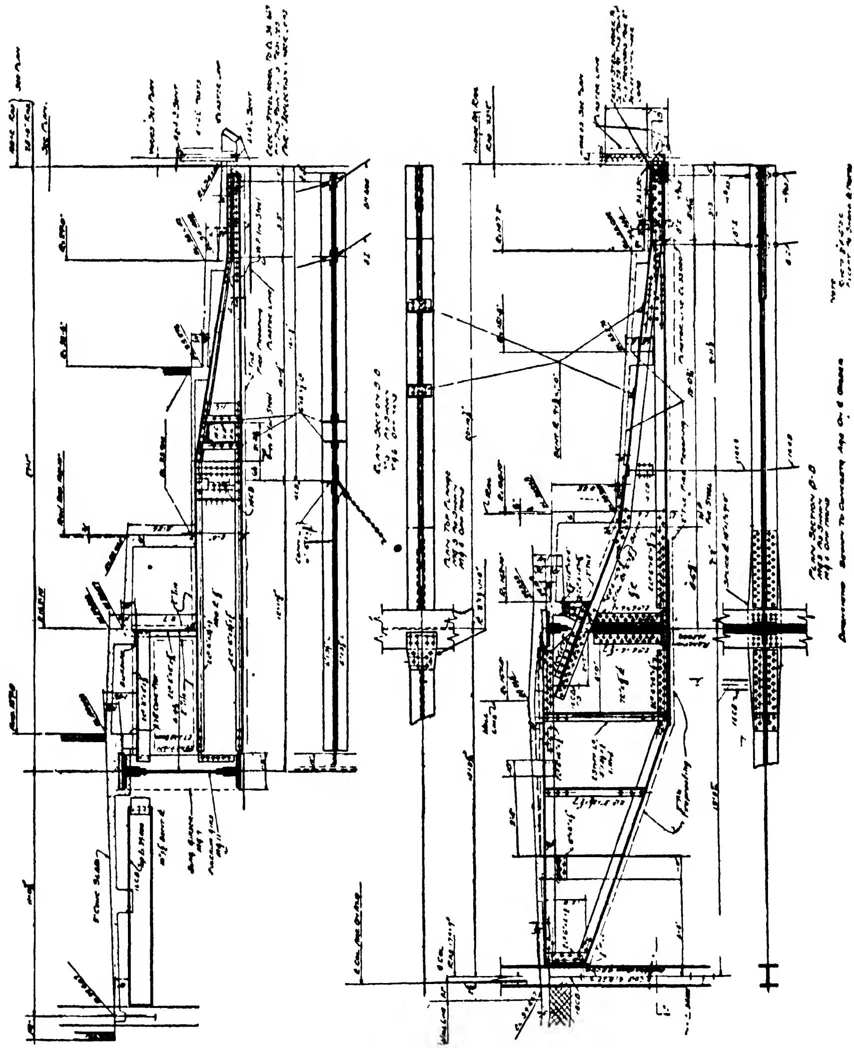


Fig. 190. Cantilever Girders in Balcony of the Chicago Civic Opera House
Courtesy of Graham, Anderson, Probst, and White, Architects

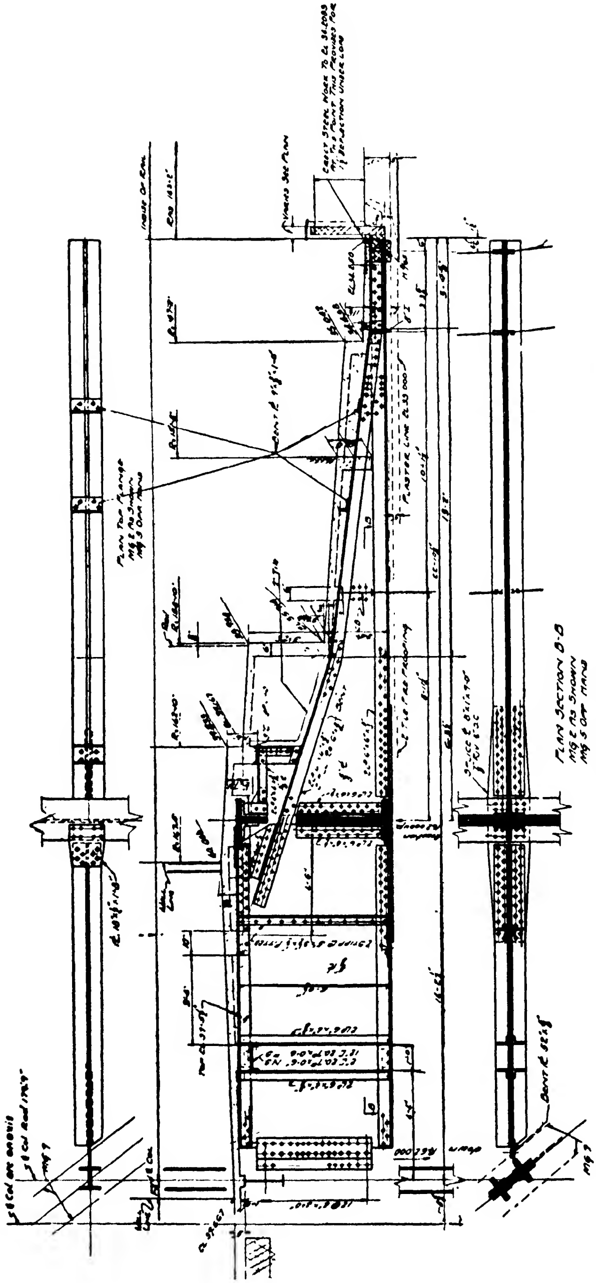


Fig. 191. Cantilever Girder in Balcony of the Chicago Civic Opera House
Courtesy of Graham, Anderson, Probst, and White, Architects

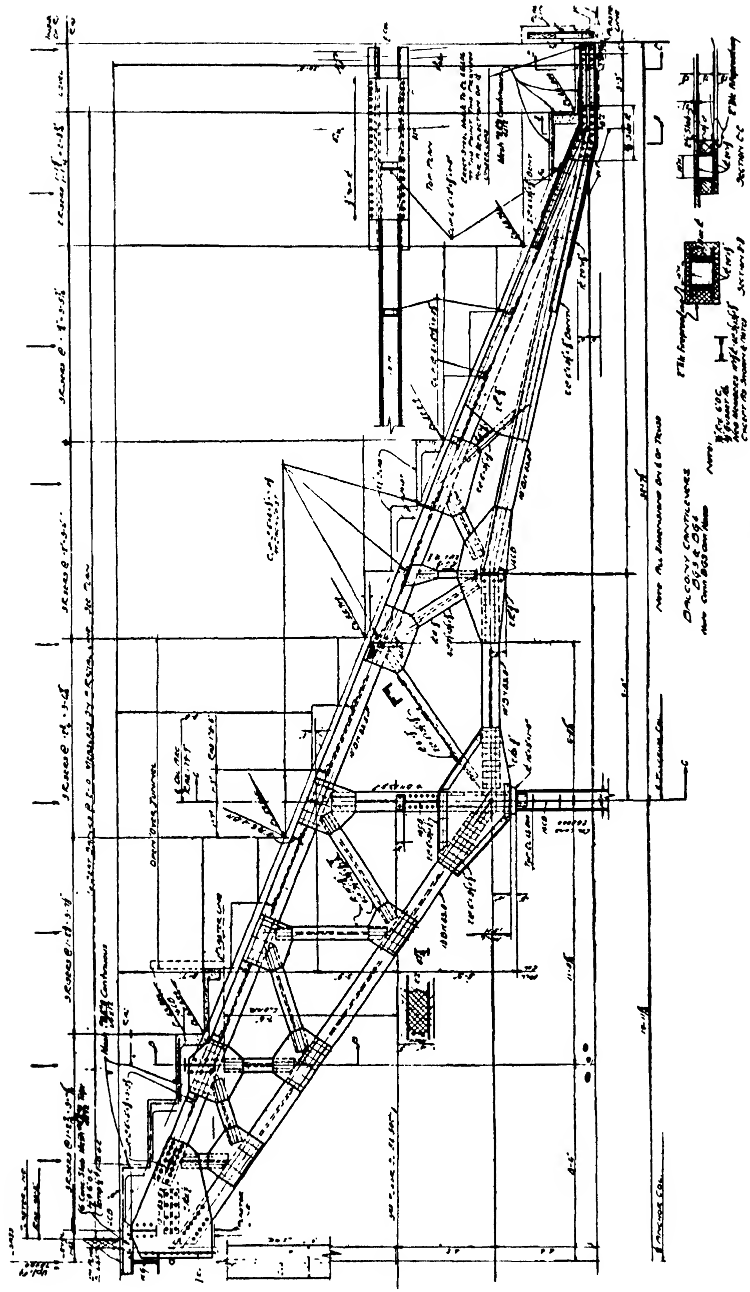
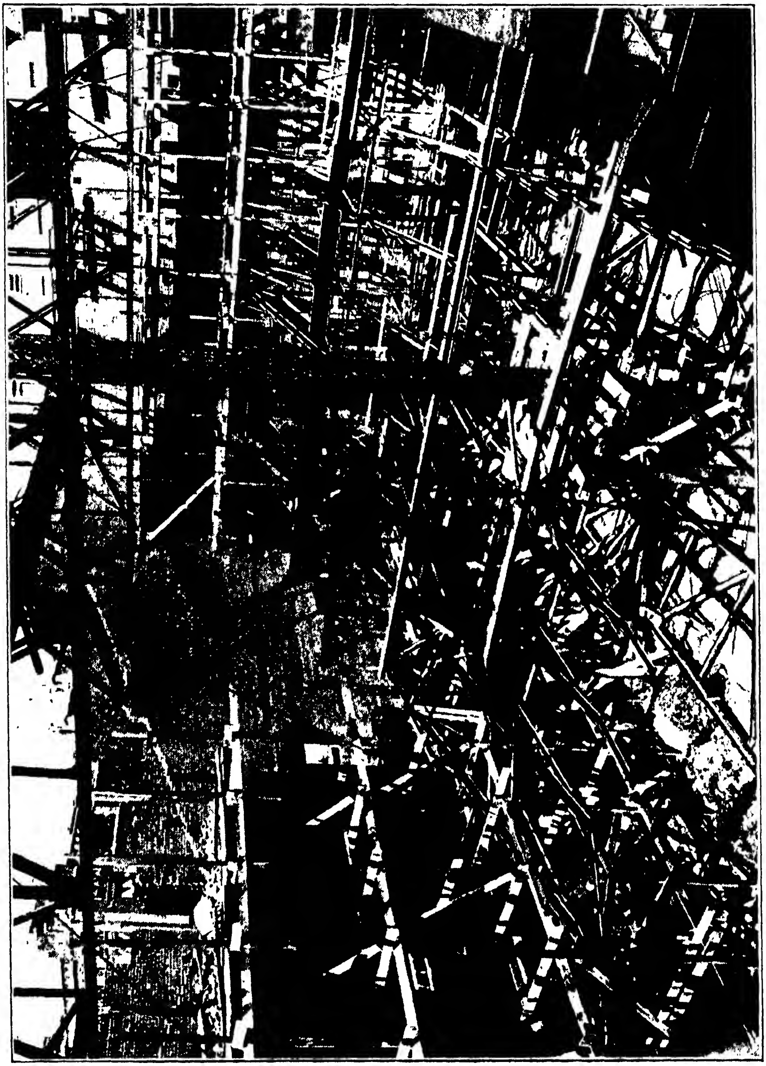


Fig. 192. Cantilever Truss in Gallery of the Chicago Civic Opera House
Courtesy of Graham, Anderson, Probst, and White, Architects



BALCONY FRAMING, THEATRE BUILDING, GREENPOINT AVE., LONG ISLAND CITY, NEW YORK
Courtesy of American Institute of Steel Construction

brackets framing into the columns can be made cantilevers if girders or trusses can be placed from Columns A back to Columns B to take the uplift.

In Fig. 187 is shown the framing for a relatively large balcony. The only columns used are in the wall. Diagonal girders or trusses are employed to reduce the length of the fulcrum girder or truss. Sufficient horizontal bracing should be provided between the lower compression chords of the cantilevers. Vertical bracing should also be used between the cantilevers where necessary.

The floor construction of balconies usually consists of concrete steps or seat banks. The vertical portions of these steps are utilized as beams or joists for carrying the load to the cantilevers.

In Fig. 188 is shown a section through the opera house and office wings portion of the Chicago Civic Opera Building. The balcony floor framing plan is shown in Fig. 189. In Figs. 190, 191, and 192 are shown some of the cantilever girders and trusses used. Attention is called to the interlocked framing of the cantilever and fulcrum girders, *MG 2, 3, 4,* and *5*. Note that the tension flange of the cantilever passes through the fulcrum girder. The compression in the bottom chord of the cantilever is taken care of by passing an $18'' \times 1''$ tie-plate under the fulcrum girder.

WIND BRACING

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.

Because of the over-turning moment caused by wind pressure, it is desirable to investigate the building's stability. Most specifications specify that over-turning moment shall in no case exceed seventy-five per cent of the moment of stability due to dead loads only.

Unit Pressures. 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. Specifications provide that for stresses produced by wind forces alone, or combined with those from live and dead loads, the units may be increased thirty-three and a third or 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 thirty-three and a third or fifty per cent of the gravity load stress.

The walls and partitions in a steel frame building are not counted on to resist any of the wind force. Actually, the walls, until they are fractured, furnish a large part of the resistance to lateral force; however, the assumption is made that the steel frame resists the full required wind load.

Wind-Stress Analysis. There are various methods of wind-stress analysis in common use; however, the method hereinafter explained is probably the one most commonly employed. The use of a so-called "exact method" does not seem warranted as it is questionable if the results are actually more correct than those secured by a simple, approximate method.

It should be realized that a building frame is one of the most indeterminate of structures. Strict theory is not applicable, as it is not possible to make correct assumptions regarding the action of a steel frame with masonry walls and floors. Some of the unknowns in this problem of wind-stress analysis are as follows: The actual wind pressure; the distribution of the wind pressure over the exposed surface of the building; the stiffening effect of the walls, partitions, and floors; the rigidity of the connections; the relative action of the girders and columns in the lower stories, where the columns are massive, compared to the action of those in the upper stories.

From the inspection of steel frame buildings that have withstood hurricanes, it has been found that the walls served as the main resistance to the lateral distortion. Later, the walls sheared or lost bond so that their resistance was destroyed, and only after such failure of the walls did the steel frame come into full action. Effective cooperation of floor slab and girders was also had in resisting the bending in the girders. It was also found that the floors distributed the horizontal forces so that there was equal deflection of all the columns at a floor. From these results it is evident that, until the walls of a building have fractured, they furnish the major part of the resistance to wind forces.

For exceptional cases of very tall buildings the deflection and vibration should be taken into consideration. The wind pressure should also be distributed among the various bents according to their rigidities. If the sway and deflection is permitted to become too large it will cause discomfort to the tenants of the top stories. The sway of a building is caused by gusts of wind or variations in wind pressure. So-called static deflection is due to an average wind pressure over a considerable period of time amounting to several minutes.

The stiffness of a tall steel frame depends upon the rigidity of its connections. To secure sufficient stiffness, knee braces and cross or **K** bracing should be used. Welded beam to column connections also provide joints that are rigid and eliminate inelastic deflections.

If it is desired to use one of the so-called "exact" methods for determining wind stresses in multistory buildings, for checking the stresses in the lower two or three stories, or for checking the results obtained by an "approximate" method, the Moment-Distribution Method by Hardy Cross* can be used.

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,

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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.

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. 193 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.

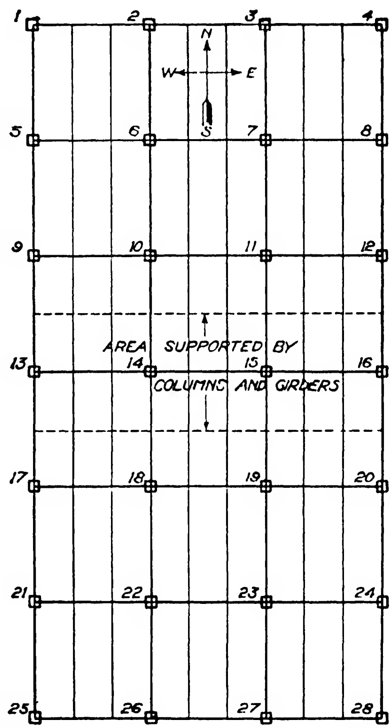


Fig. 193. Framing Plan of Building for Study of Bracing System

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. 194, having axial stresses; and (2) by rectangular framework, Fig. 195, having bending stresses.

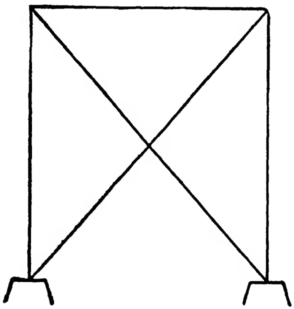


Fig. 194. Diagram of Triangular Framing

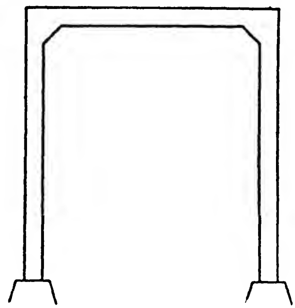


Fig. 195. Diagram of Rectangular Framing

Triangular Framework. Single Panels. Fig. 196 shows a single panel of triangular framing supporting the horizontal force W. The reactions at the foundations are R, V', and V.

$$R = W$$

$$V = V^1 = \frac{WH}{L}$$

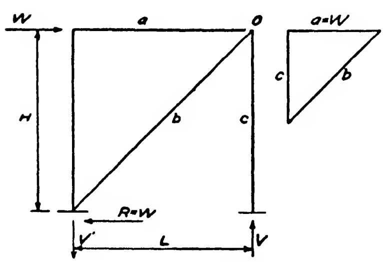


Fig. 196. Diagram of Stresses in Triangular Framing

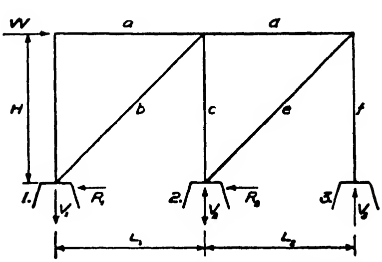


Fig. 197. Diagram of Two Horizontal Panels of Triangular Framing

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, 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. 197. 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 O.

Problem. Assume four panels similar to those shown in Fig. 197. 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.

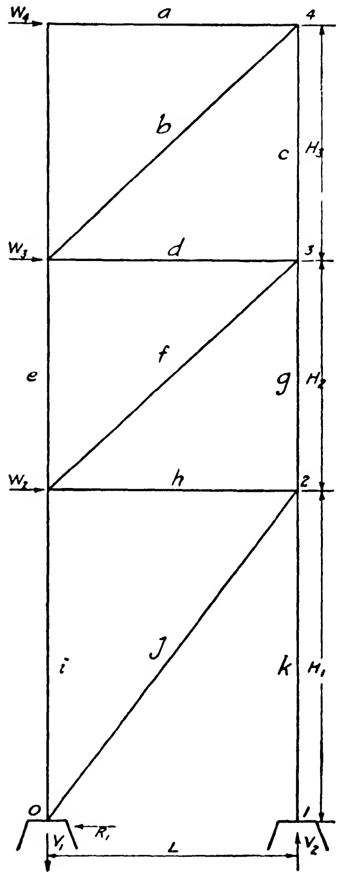


Fig. 198. Diagram of Vertical Panels of Triangular Framing

Two or More Vertical Panels. Two or more panels may be placed one above the other as in Fig. 198. 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 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.

Problem. In Fig. 198 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.

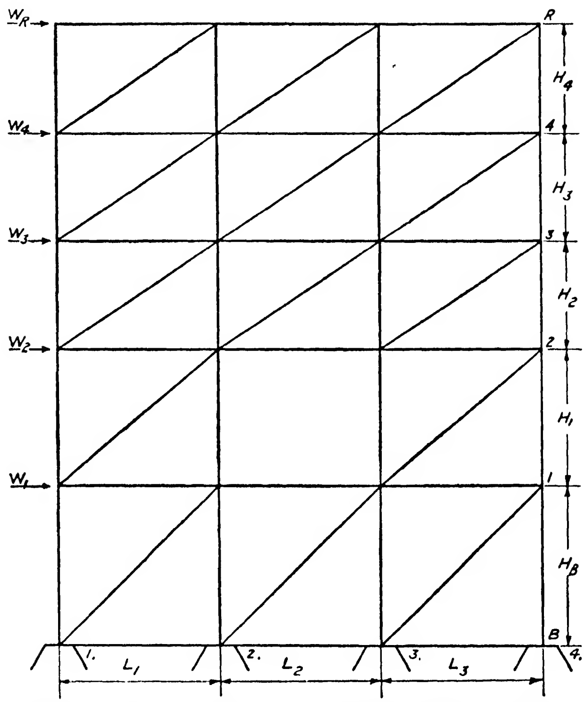


Fig. 199. Diagram of Triangular Framing Extending Over a Building

199. 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 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. 199 and compute the stresses in the diagonal members.

In Figs. 196 to 199 inclusive the diagonals are shown in one direction only. As the wind may come from either direction, both diagonals will be used in all cases. In certain panels, circumstances may

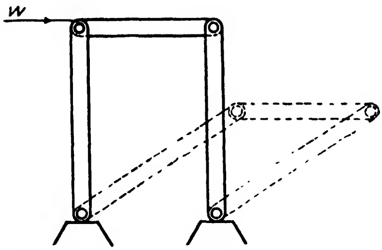


Fig. 200. Diagram of Rectangular Frame with Hinged Joints

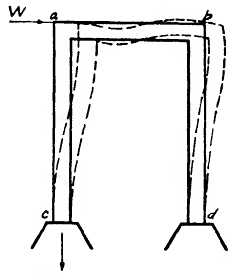


Fig. 201. Diagram of Rectangular Frame with Rigid Joints

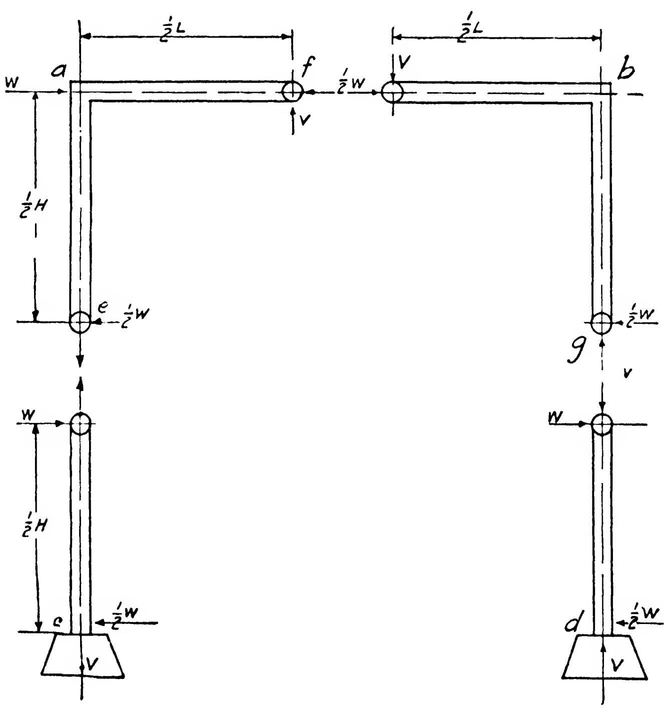


Fig. 202. Diagram of Rectangular Frame Showing Points of Contraflexure

prevent the use of any diagonal bracing, Fig. 199, in 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. 200. 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. 201. 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 through the internal forces set up in the members, 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. 202. 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

$$\tfrac{1}{2}W\times\tfrac{1}{2}H=\tfrac{1}{4}WH$$

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. 203.

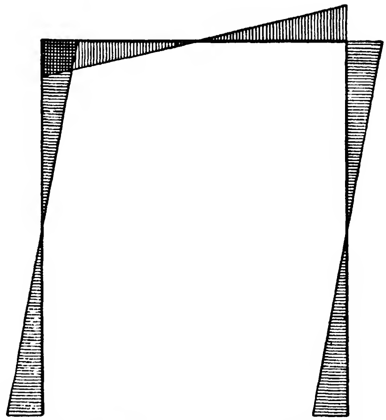


Fig 203. Moment Diagram of Single Rectangular Panel

In addition to the bending stresses in the members, there are axial stresses, as indicated by the forces and reactions illustrated:

in *a b* $\quad \frac{1}{2}W$, compression

in *b d* $\quad V=\frac{1}{2}W\frac{H}{L}$, compression

in *a c* $\quad V=\frac{1}{2}W\frac{H}{L}$, tension

Problem. Refer to Fig. 202. 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.

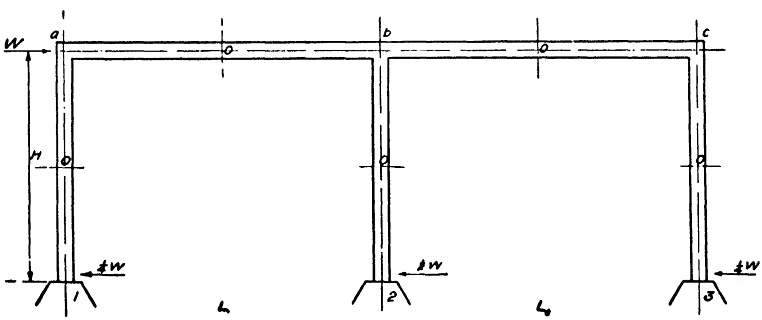
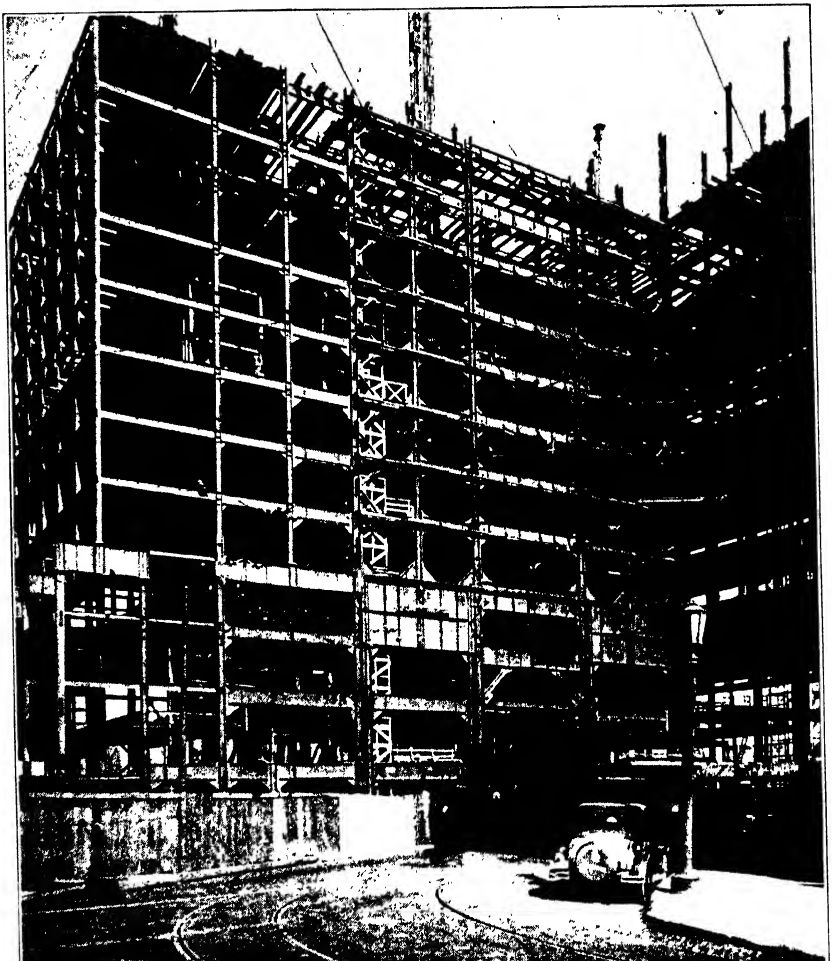


Fig. 204. Diagram of Two Rectangular Panels

Two Horizontal Panels. Next consider a framework of two panels, i.e., made of three columns and two girders, as in Fig. 204,



THE TERMINAL TOWER BUILDING, CLEVELAND, OHIO
Note K Type and Bracket Bracing, Also Omission of Some Lower Story Columns.
Courtesy of Graham, Anderson Probst and White, Architects

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.

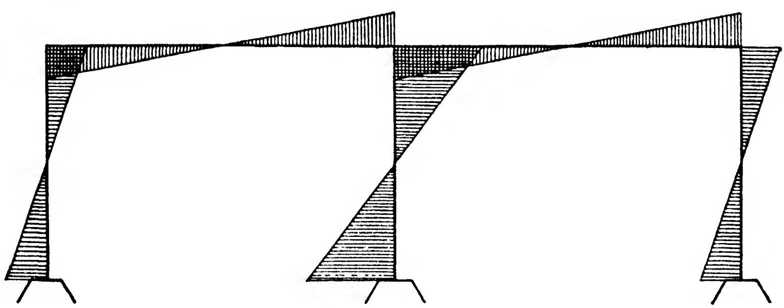


Fig. 205. Moment Diagram for Frame of Two Rectangular Panels

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. 205.

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. 206 there are five panels.

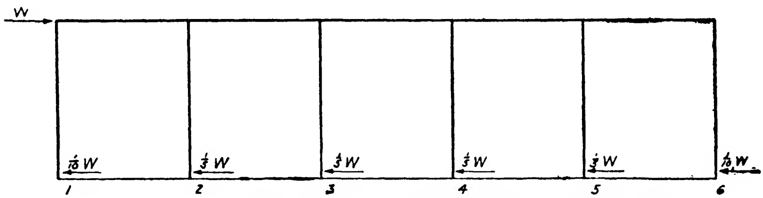


Fig. 206. Diagram Showing Division of Shear in a Frame of Five Panels

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: $\frac{1}{20}$ W H; in columns *2*, *3*, *4*, and *5*, $\frac{1}{10}$ W H; and in all girders, $\frac{1}{20}W$ H.

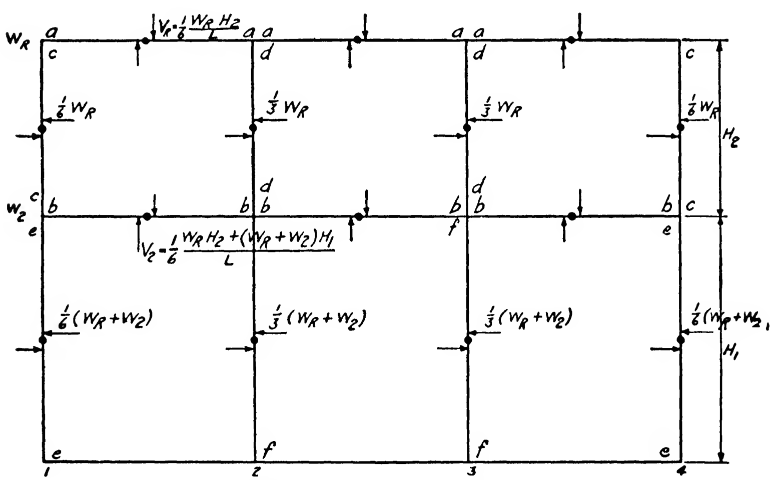


Fig. 207. Stresses in a Two-Story Rectangular Framework

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. 207. 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. 208 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'.

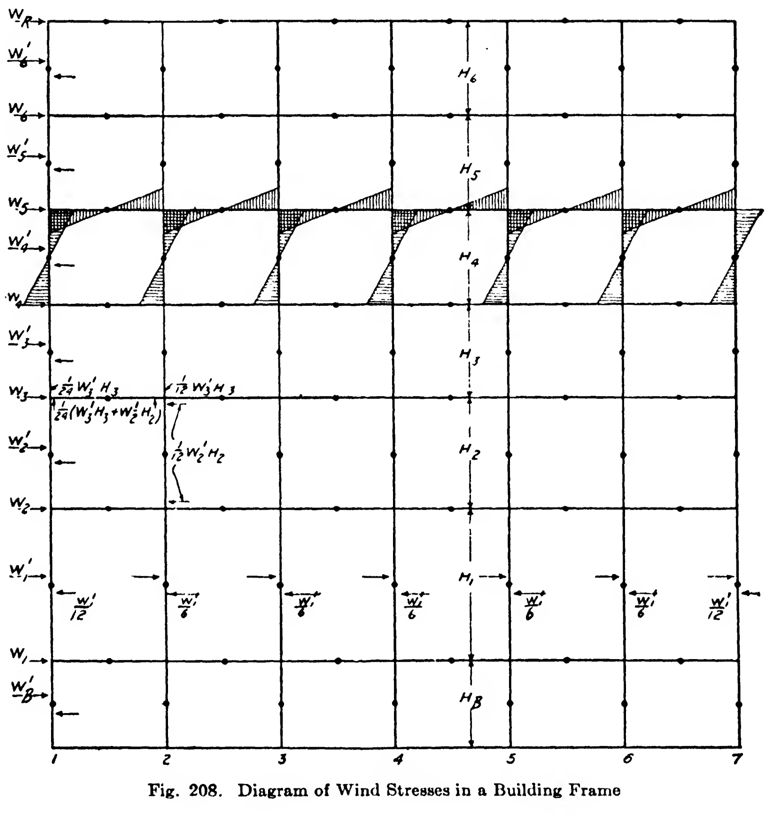


Fig. 208. Diagram of Wind Stresses in a Building Frame

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 the bending moments in the column above and below the girder. It is expressed by the formula

$$M=\frac{1}{2}\left(\frac{W_a'H_a}{2n}+\frac{W_b'H_b}{2n}\right)=\frac{1}{4n}\,(W_a'H_a+W_b'H_b)$$

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. 208. 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

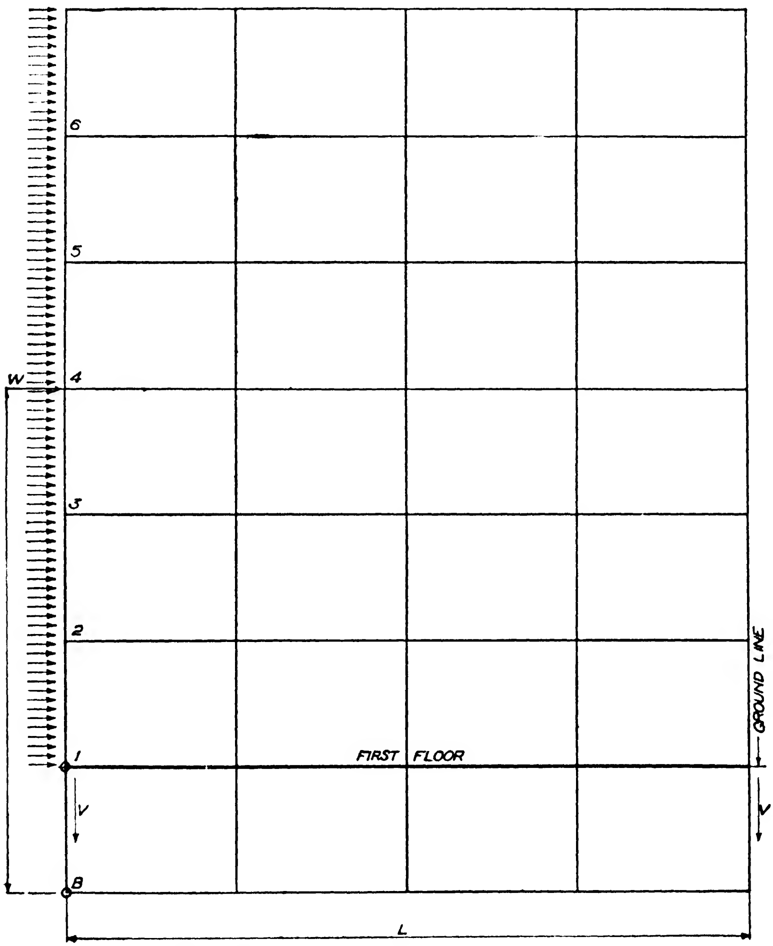


Fig. 209. Diagram of Overturning Stresses in a Building Frame

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. 209 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. 209 and compute the axial stress in the second-story sections of the end columns.

2. In Fig. 208 assume the following values:

$$
\begin{aligned}
&H_B &&= 10'\text{-}0'' \\
&H_1 &&= 16'\text{-}6'' \\
&H_2,\ H_3 - \text{as } H_6 &&= 12'\text{-}6'' \\
&W_1 &&= 8{,}000\# \\
&W_2 &&= 14{,}500\# \\
&W_3,\ W_4,\ W_5,\ W_6 &&= 12{,}500\# \\
&W_R &&= 10{,}000\#
\end{aligned}
$$

(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 CONNECTIONS

Heretofore, in designing end connections they 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 columns is chiefly to resist the bending moment. These connections require careful designing to insure effective results.

The shallow type of wind-bracing connections shown in Fig. 210 usually do not encroach on the architectural clearances of a building. These connections are satisfactory where the end moments in the girders are not too large. The strength of each one of these connections is governed by the strength of the rivets in tension, the rivets through the column, and the outstanding legs of the angles or clipped I or WF beams.

For the type of connection shown in Fig. 210 at (a) it is customary to assume that the shear is carried by the web connection and the moment by the top and bottom seat angles. To illustrate, the resisting moment will be computed for this connection. Assume the depth of beam as 12 inches, $d = 16''$. The top and bottom angles have four rivets each into the column flange. Rivets, $\frac{7}{8}''$. A.I.S.C. Specifications allow 15,000# per square inch on rivets in tension and can be increased $33\frac{1}{3}\%$ when wind is included.

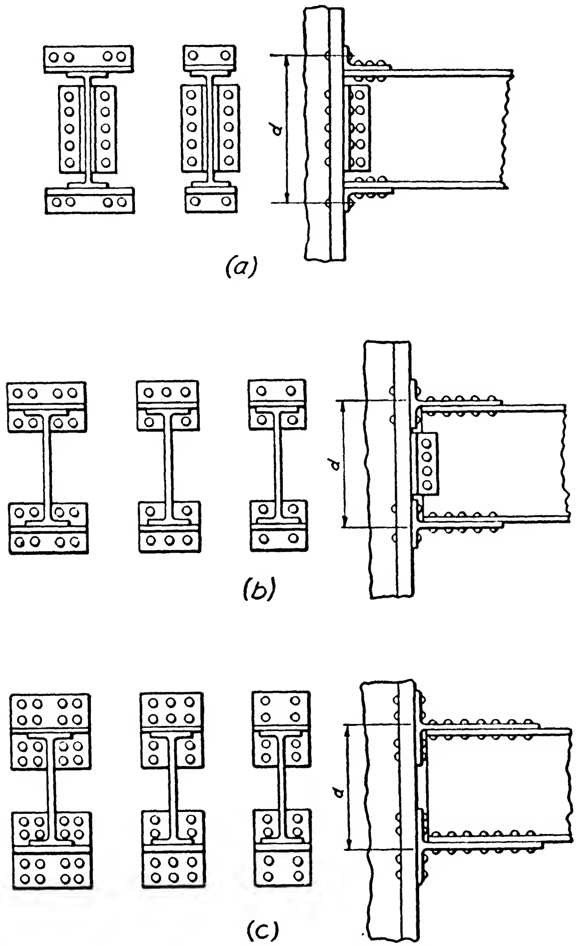


Fig. 210. Shallow Wind-bracing Connections

Resisting moment equals

$$M_R = 4 \times 16 \times 12{,}027 = 770{,}000''\#$$

The number of rivets required for connecting the top and bottom angles to the beam flanges equals

$$\frac{770{,}000}{12 \times 12{,}027} = 5.3$$

Therefore 6 rivets in each angle should be used.

The required thickness of the top and bottom angles are computed as follows. Refer to Fig. 211.

$$M = \frac{R(g-t)}{2}$$

$$t = \sqrt{\frac{3R(g-t)}{pf_s}}$$

In computing the required thickness of these angles, some designers reduce the distance $g-t$ by the size of the angle fillet. This refinement is not justified, because the results are not exact, as the theory of flexure does not hold for a beam of these proportions and because of the uncertainty in the location of the point of contraflexure. However, the results are conservative.

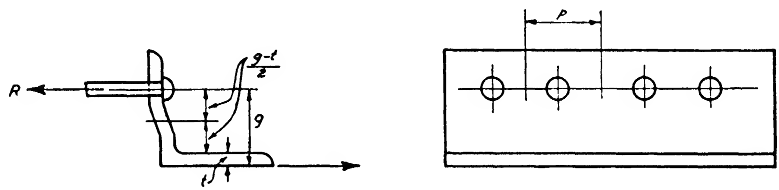


Fig. 211. Bending of Angle Legs

Using the A.I.S.C. Specification stresses, the following angle thicknesses and gages are satisfactory.

Rivet size, inches	$\frac{3}{4}$	$\frac{7}{8}$	1
Gage	$1\frac{3}{4}$	2	2
Thickness of angle	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$

The connections shown in Fig. 210 at (*b*) and (*c*) are computed in a manner similar to that used for connections in (*a*). Note that the distance *d* is not between the extreme rivet lines.

When this type of connection does not permit the use of web angles to take the vertical shear, the number of rivets into the column is large, and the shear per rivet is small enough to be neglected. Also, there is always compression between either the top or bottom connection and the column flange, therefore there is no stress in these rivets due to moment, and they can be counted on to help carry the vertical shear.

The required thickness of flange for the clipped I beam or structural **T** sections used in these connections can be computed in the same manner used for the angles in the previous connection.

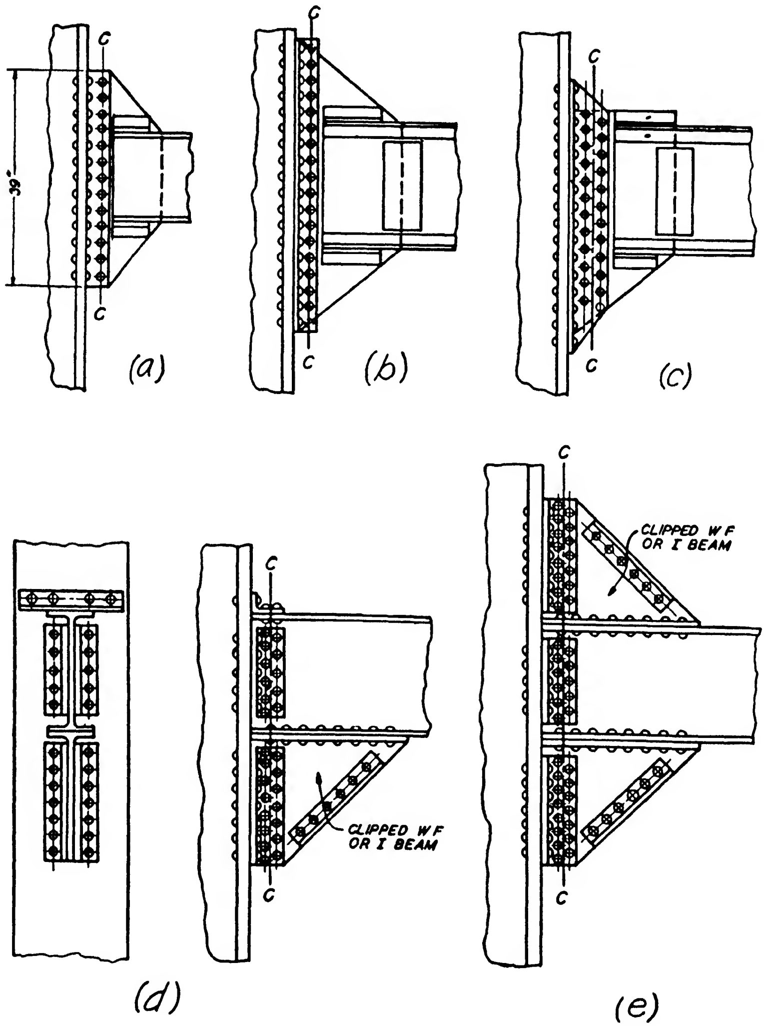


Fig. 212. Bracket Types Wind-bracing Connections

The rivets connecting into the beam flanges should be computed the same as for the connections in Fig. 210 at (*a*).

The bracket type of connections shown in Fig. 212 are more rigid than those in Fig. 210 and should be used wherever possible. As these connections encroach on the architectural clearances, they are used in outside walls or where partitions occur. Where encroachment in the ceiling is not objectionable, the brackets in Fig. 212 at (*c*) and (*d*) can be used.

If there is one row of rivets along the lines *c–c*, they govern the design. These rivets are in shear and bearing. When two rows of rivets are used along lines *c–c*, then the rivets into the column govern. These rivets are stressed in tension.

The point of rotation for the rivets along lines *c–c* is at the center of gravity of the rivet group. For the rivets in tension (rivets into the column) the point of rotation moves toward the bottom of the connection which is in compression, due to the fact that the tensile deformation is greater than the compressive deformation. A satisfactory and conservative assumption is to take the point of rotation up from the bottom of the bracket two tenths of the depth.

To illustrate, the design of the rivets in the bracket connection shown in Fig. 212 at (*a*) will be computed*. First, the rivets in tension (rivets into column) will be considered. Point of rotation will be located up from the bottom of the bracket

$$.2 \times 39'' = 7.8 \text{ inches}$$

Say 7.5″, which is at the third rivet up from the bottom.

The variation in stress is shown in Fig. 213 at (*a*). The top rivet is stressed to the allowed, which is 6630# plus $33\frac{1}{3}\%$ increase due to wind loads. The values for each rivet can be measured from a diagram drawn to scale as shown in (*a*) or they can be computed as follows:

$$\begin{aligned}
8840 \times \tfrac{30}{30} &= 8{,}840 \\
8840 \times \tfrac{27}{30} &= 7{,}950 \\
8840 \times \tfrac{24}{30} &= 7{,}060 \\
8840 \times \tfrac{21}{30} &= 6{,}180 \\
8840 \times \tfrac{18}{30} &= 5{,}300 \\
8840 \times \tfrac{15}{30} &= 4{,}420 \\
8840 \times \tfrac{12}{30} &= 3{,}530 \\
8840 \times \tfrac{9}{30} &= 2{,}650 \\
8840 \times \tfrac{6}{30} &= 1{,}765 \\
8840 \times \tfrac{3}{30} &= \underline{\quad 885} \\
&\ 48{,}580\#
\end{aligned}$$

*All computations are by slide rule.

This resultant force is multiplied by two, as there are two rows of rivets. This total force of 97,160# acts at a distance up from the point of rotation of 21 inches, or two thirds of the distance to the extreme top rivet. The force P_c is equal to P_t (97,160#) and acts 5 inches, or $\frac{2}{3}$ of $7\frac{1}{2}$ inches, below the point of rotation.

The resisting moment equals

$$\begin{aligned} &(97{,}160\times21)+(97{,}160\times5)\\ &\text{or } 97{,}160\times26=2{,}520{,}000''\# \\ &\qquad\qquad\qquad =210{,}000'\# \end{aligned}$$

The vertical shear on these rivets is usually neglected as it is small, due to the large number of rivets; also, the rivets in the compression portion are not stressed in tension, therefore they are available to help carry the vertical load.

The rivets into the web plate will now be investigated. These rivets are in shear and bearing. The point of rotation is at the center of gravity. See Fig. 213 at (*b*). Assume that the web is $\frac{1}{2}$ inch thick, then double shear will govern. Increasing the rivet shear value by $33\frac{1}{3}\%$, the result is 17,680#.

The resisting moment equals

$$\begin{aligned} 17{,}680\times\tfrac{18}{18}\times18&=318{,}000\\ 17{,}680\times\tfrac{15}{18}\times15&=221{,}000\\ 17{,}680\times\tfrac{12}{18}\times12&=141{,}300\\ 17{,}680\times\tfrac{9}{18}\times\ 9&=\ 79{,}500\\ 17{,}680\times\tfrac{6}{18}\times\ 6&=\ 35{,}360\\ 17{,}680\times\tfrac{3}{18}\times\ 3&=\ \underline{\ \ 8{,}840}\\ &\ \ \ 804{,}000''\# \end{aligned}$$

$$2\times\frac{804{,}000}{12}=134{,}000'\#$$

If the vertical load is large, the stress in the extreme rivet due to moment should be combined with that from vertical load. Usually this stress due to vertical load is so small it can be neglected. As the resisting moment of the rivets along line *c–c* is 134,000′# and the resisting moment of rivets in the column is 210,000′#, it is economical and desirable to place a double row of rivets along lines *c–c*.

In Fig. 214 the girder web connects directly to the flange of the column. The method of designing the connection is the same as that explained for a double row of rivets along line *c–c* of Fig. 212 at (*b*),

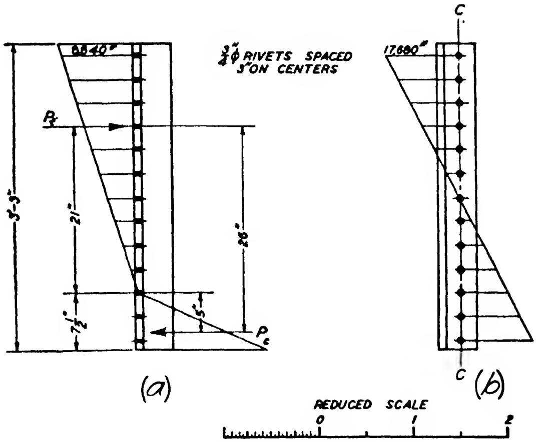


Fig. 213. Resistance of Rivet Groups

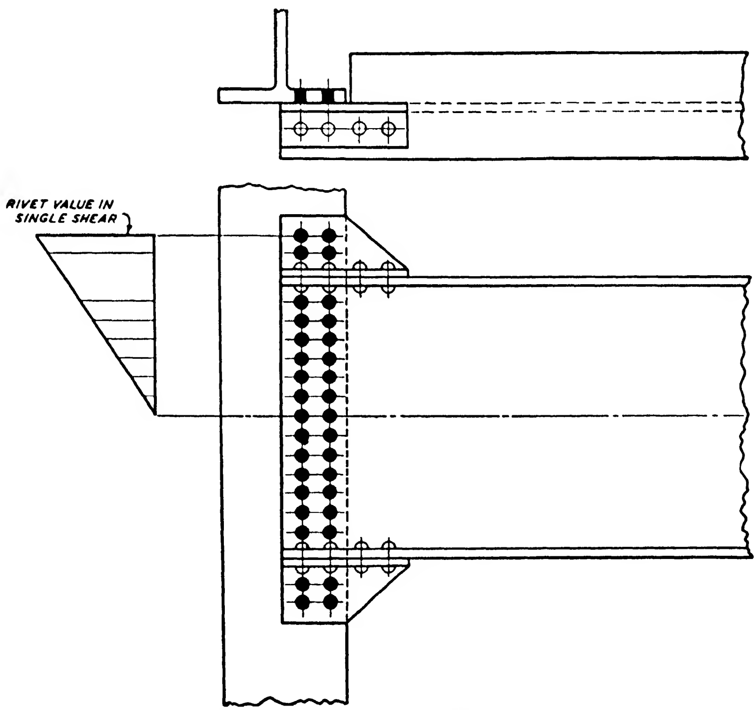


Fig. 214. Girder Connection to Face of Column

except that the rivets are in single shear. As the rivets in this connection are not evenly spaced, the value of each rivet can be measured from the stress diagram shown in the figure.

Fig. 215 shows the **K** and cross-frame types of bracing. Wherever conditions permit, bracing of these types should be used as they are the most direct, efficient, and economical types.

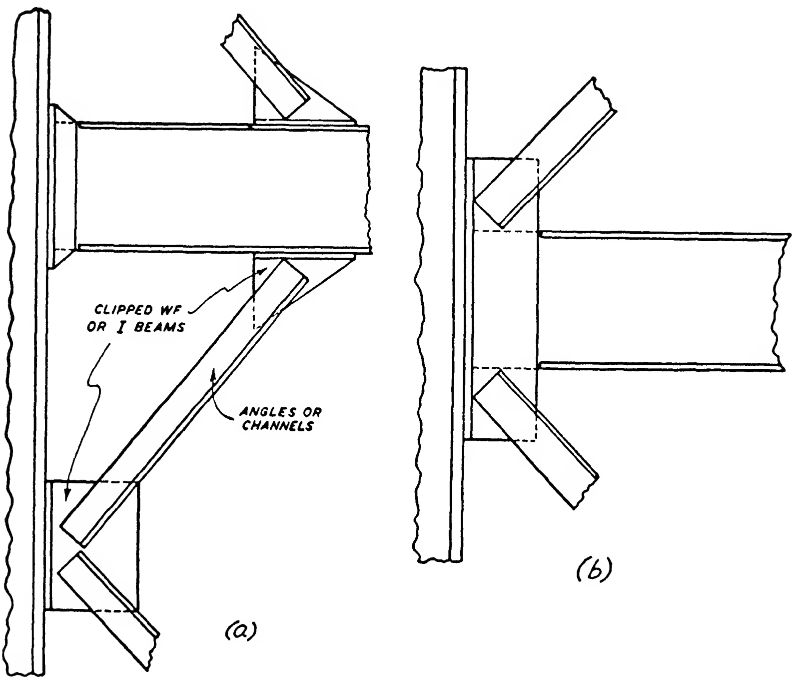


Fig 215. Cross-Frame and **K** Types Wind-bracing. (*a*) **K** Type Bracing; (*b*) Cross-Frame Bracing

Bracing for Earthquake Forces. Bracing similar to types used for wind forces are employed to resist seismic forces. However, as explained in the section under "Loads," the forces due to wind and earthquakes are not the same and do not act in the same manner.

Among several theories of design for earthquake resistance, two have been given particular credence, these being flexible and rigid construction. In the flexible design, the columns in the lower stories are made flexible but adequately strong, and are securely connected at top and bottom. The theory is that these columns will sway and bend with the ground movements and thus relieve the upper stories of dangerous stresses. One of the disadvantages of this type of construction is that in earthquake territories there are numerous small tremblers, and with flexible lower story columns the walls tend to crack with each small seismic disturbance.

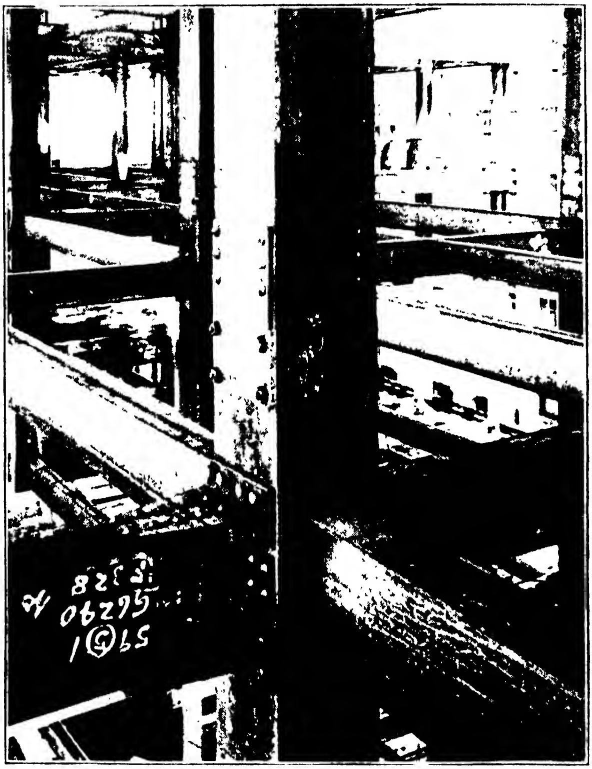


Fig. 216. Wind-bracing on an Interior Column
Courtesy of The American Bridge Company

The rigid type of construction is now widely advocated. This type requires that the building be so braced, stiffened, and connected together vertically and horizontally, that it will move bodily with the ground without overstressing any members or connections, and also without appreciable deformation.

In bracing a building for earthquake forces, diagonal cross-frame bracing should be used wherever possible. Where openings will not

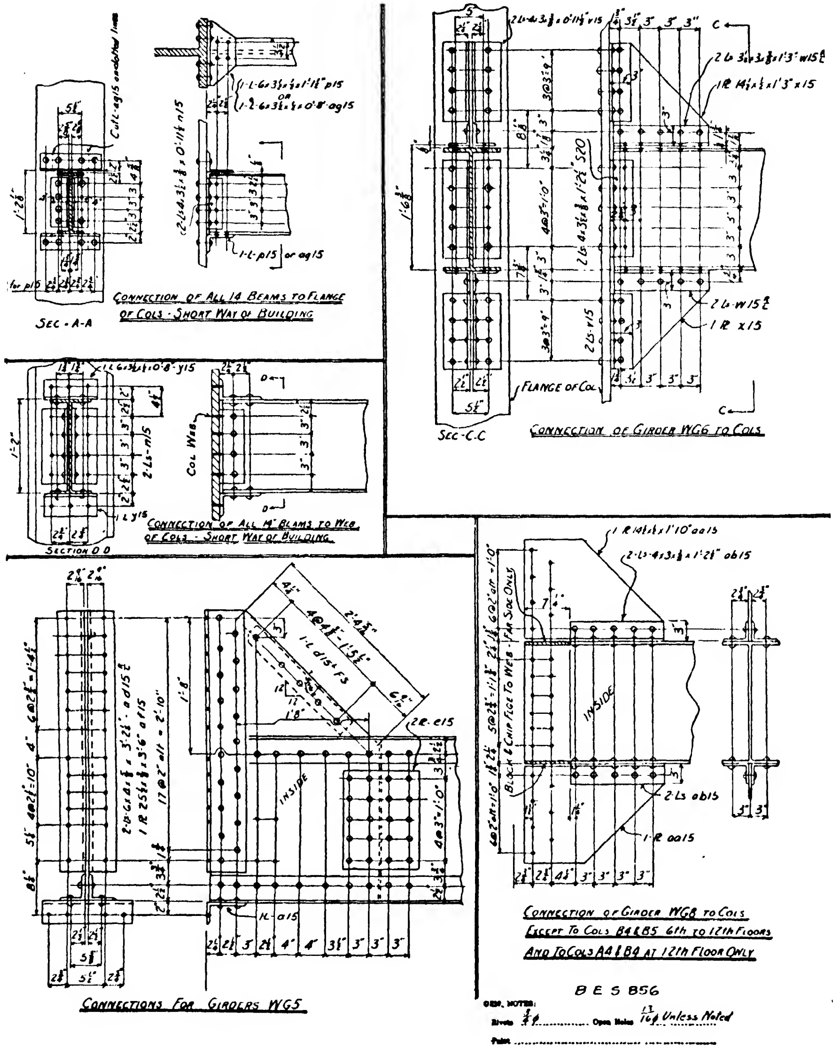


Fig. 217. Shop Detail Drawing of Wind-bracing Connections, A. T. & S. F. Railway Office Building, Amarillo, Texas

permit this type of bracing, knee brackets or **K** bracing should be used. Bracing should also be placed in the interior of the building, and this bracing should be located as symmetrically as possible to avoid torsional effects.

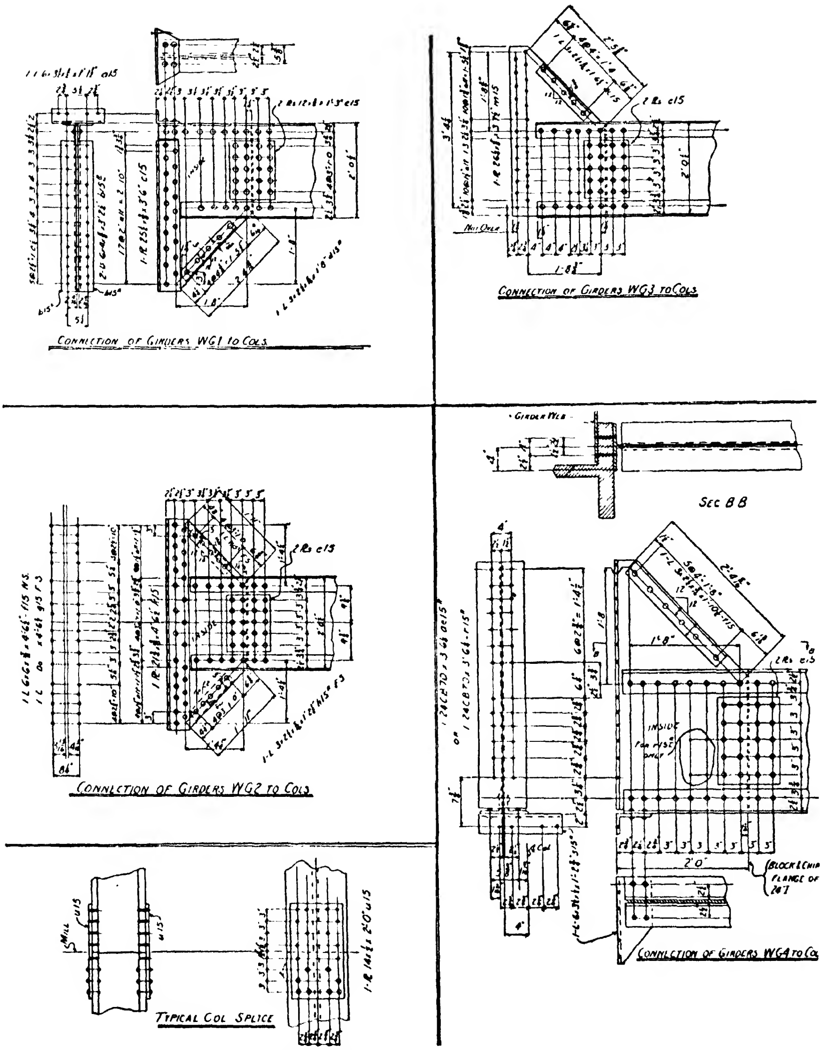


Fig. 217. Shop Detail Drawing of Wind-bracing Connections, A. T. & S. F. Railway Office Building, Amarillo, Texas

In Fig. 216 is shown a wind-bracing connection on an interior column. This bracing is made up of clipped I beams. It will be noted that the field connections are a combination of bolts and welds.

Fig. 217 shows the shop detail drawing covering the wind-bracing connections for A. T. & S. F. Ry. Office Building, Amarillo, Texas.

Problems. 1. Compute the resisting moment of the rivets for the bracket similar to the one in Fig. 212 at (*b*). The total depth of the bracket is 4′-6″. There are 18 rivets $\frac{3}{4}''$ in diameter, spaced 3 inches on centers.

2. From the above problem compute the resistance of the rivets along the line *c-c*, assuming two rows spaced $2\frac{1}{2}$ inches apart.

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.

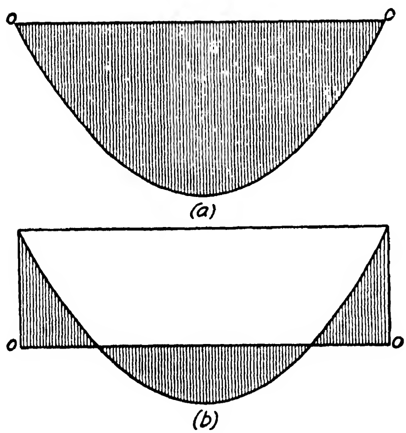


Fig. 218. Moment Diagrams (*a*) of Simple Beam; (*b*) of Restrained Beam

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. 218 at (*a*). If the beam is restrained by rigid connections at the ends, it tends to deflect in the form of a compound curve and the moment diagram, Fig. 218 at (*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.

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 $-\frac{1}{12}\,WL$, and at the middle is $+\frac{1}{24}WL$.

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 unless end connections are strong enough to develop the strength of the beam. When there is a lack of rigidity at the ends, the negative moment decreases and the positive moment increases. 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. 219, 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 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. 219, shows a very large resultant negative bending moment at the left end of the diagram, and a small resultant positive bending moment at the right end. If the respective end connections be designed to resist

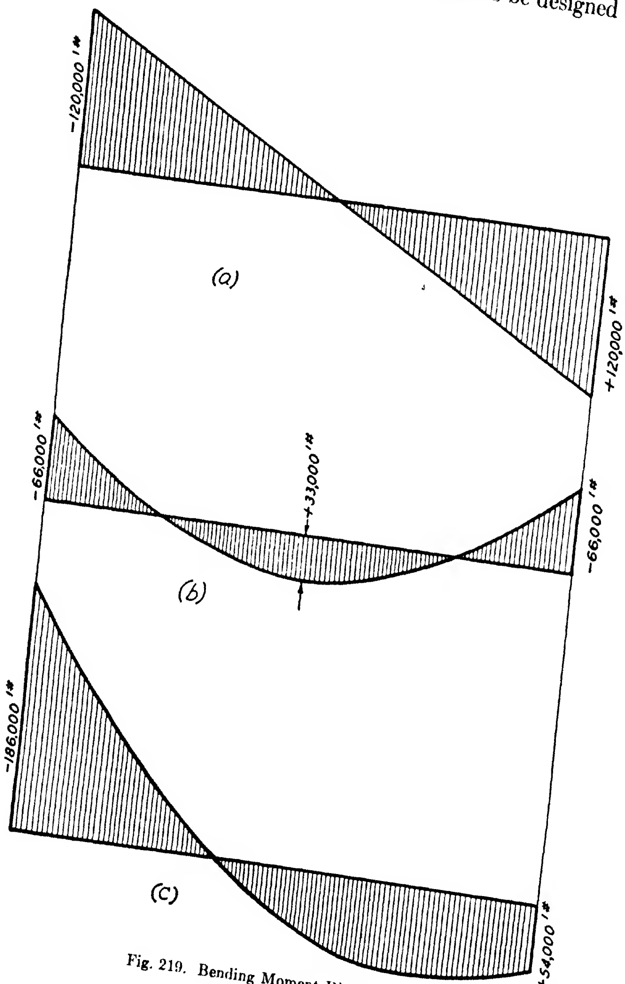


Fig. 219. Bending Moment Diagrams

these moments, i.e., the left end with a very heavy connection and the right end with a light connection, 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.

The moment diagrams in Fig. 219 are drawn for a span length extending between column centers. If the connection is made to the column flange, the moment for which the connecting rivets must be designed is less than the maximum because the end of the girder is the distance of half the depth of the column away from the column center. For a girder framing into the web, the maximum moment must be provided for, as the end lacks only a fraction of an inch of reaching the column center. The actual amount of moment to use can be computed or scaled from the moment diagram.

Problem. In Fig. 219 assume values given for diagrams *a* and *b*. Construct diagram *c* accurately to scale.

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.

The combination of the direct and the bending stress is illustrated in Fig. 220. 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 one-third $a\,b$, 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-third 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. 214. With these conditions only one half of the column section should be used in computing the resulting extreme fiber stress.

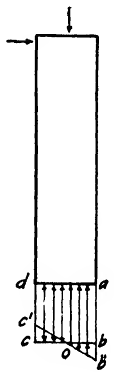


Fig. 220. Diagram of Combined and Direct Stress

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. 188. The method there given for computing the concentric equivalent load also applies, as well as the formula

$$W'_w = W'\frac{ec}{r^2} \text{ or } W'_w = \frac{Mc}{r^2}$$

As applied to wind load (refer to Fig. 221) 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. 208); 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.

When designing according to the A.I.S.C. Specifications, the foregoing method is used to select a trial section. The column is then checked to see that the combined stress formula $\left(\frac{f_a}{F_a}+\frac{f_b}{F_b}\right)$ equals unity.

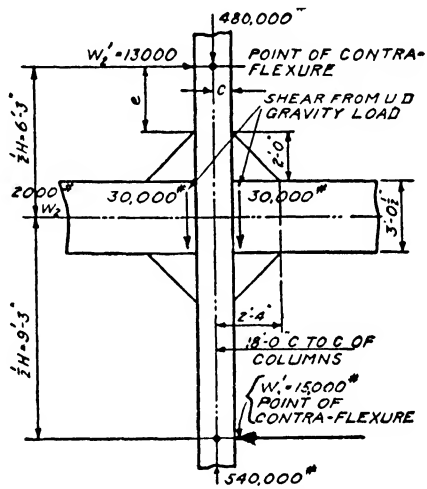


Fig. 221. Details of a Problem in Wind-bracing

In building design the columns are designed for gravity loads, and later the picked column section is checked to see if it is satisfactory when the wind stress is added. When the gravity and wind stresses are combined, the allowed stress is increased $33\frac{1}{3}\%$.

The bending moments in the columns are usually tabulated when making the wind stress analysis. These moments are at the intersection of the center lines of the columns and girders. Where deep brackets are used, the moments are taken at the critical section which is at the base of the brackets. Referring to Fig. 221 the moment in the upper story equals

$$13{,}000\#\times 6.25' = 81{,}250'\#$$

The moment at the base of the bracket is

$$\frac{2.75}{6.25}\times 81{,}250'\# = 35{,}750'\#$$

The critical section of the column is taken at the base or top of the bracket, as the bracket has the effect of enlarging the column section.

To illustrate, the top portion or story of the column shown in Fig. 221 will be investigated to see if it is satisfactory with wind stresses added. Assume that the column section selected to carry the 480,000# concentric load is a 14″ WF 103#. The allowed unit stresses are increased $33\frac{1}{3}\%$.

Area of section = 30.26□″

Section modulus = 163.6 (about strong axis)

Least radius of gyration = 3.72

$$F_a \times 1.333 = \left(17{,}000 - 0.485 \times \frac{l^2}{r^2}\right) \times 1.333$$

$$= 17{,}000 - 0.485 \times \frac{(150)^2}{(3.72)^2} \times 1.333$$

$$= 21{,}600\# \text{ per sq. in.}$$

$$F_b = 20{,}000 \times 1.333 = 26{,}660\# \text{ per sq. in.}$$

$$f_a = \frac{480{,}000}{30.26} = 15{,}900\# \text{ per sq. in.}$$

$$f_b = \frac{35{,}750 \times 12}{163.6} = 2{,}620\# \text{ per sq. in.}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{15{,}900}{21{,}600} + \frac{2{,}620}{26{,}660} = 0.736 + 0.098 = 0.838$$

As this result is less than unity, the column is satisfactory for the combined axial and bending stresses.

Problems. 1. Design the lower story column shown in Fig. 221 for axial load.

2. Check the column section secured in (1) for the additional wind load.

ILLUSTRATIVE DESIGN OF A SCHOOL BUILDING

Having studied the analysis and design of individual steel members, attention will now be given to the problems which arise in designing 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.

References are not 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. This is an apprentice school building erected by the A. T. & S. F. Ry. System at Topeka, Kansas. This building was selected for the purpose of illustrating the practical problems of design because it has an auditorium and balcony in addition to the usual types of framing.

Fireproof construction is used throughout. The framework consists of structural steel columns and girders. The floor construction is reinforced concrete slab and joists made with removable sheet metal forms. The partitions are of hollow tile plastered on both sides, and, as they are fixed in position, no additional load per square foot of floor is added to provide for movable partitions.

The basement walls are of reinforced concrete. The walls above grade are brick. The concrete incasement of the columns and the spandrel beams at the fourth floor and at the ends of the building are exposed and flush with the outside surface of the brick wall.

Figs. 222 to 236 give the complete structural plans and part of the architectural floor plans and elevations of this school building.

Arrangement of Beams. Some attention has already been given to the arrangement of the beams 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 structural plans of the first, second, or third floor.

It is necessary, of course, to have spandrel beams around the entire perimeter of the building to support the exterior walls. The next thing to be determined is whether the interior beams shall be parallel or perpendicular to the length of the building. The former arrangement is used, as it works out most advantageously due to the corridor in the center of the building and the layout of the rooms. It is to be noted that the beams, parallel with the length of the building, and their covering, project several inches below the ceiling line, hence it is desirable to place them so that they interfere as little as possible with the arrangement of the rooms. In the plan adopted, about half of the principal lines of beams are along the sides of the corridors and are therefore partially or wholly concealed by the partition wall.

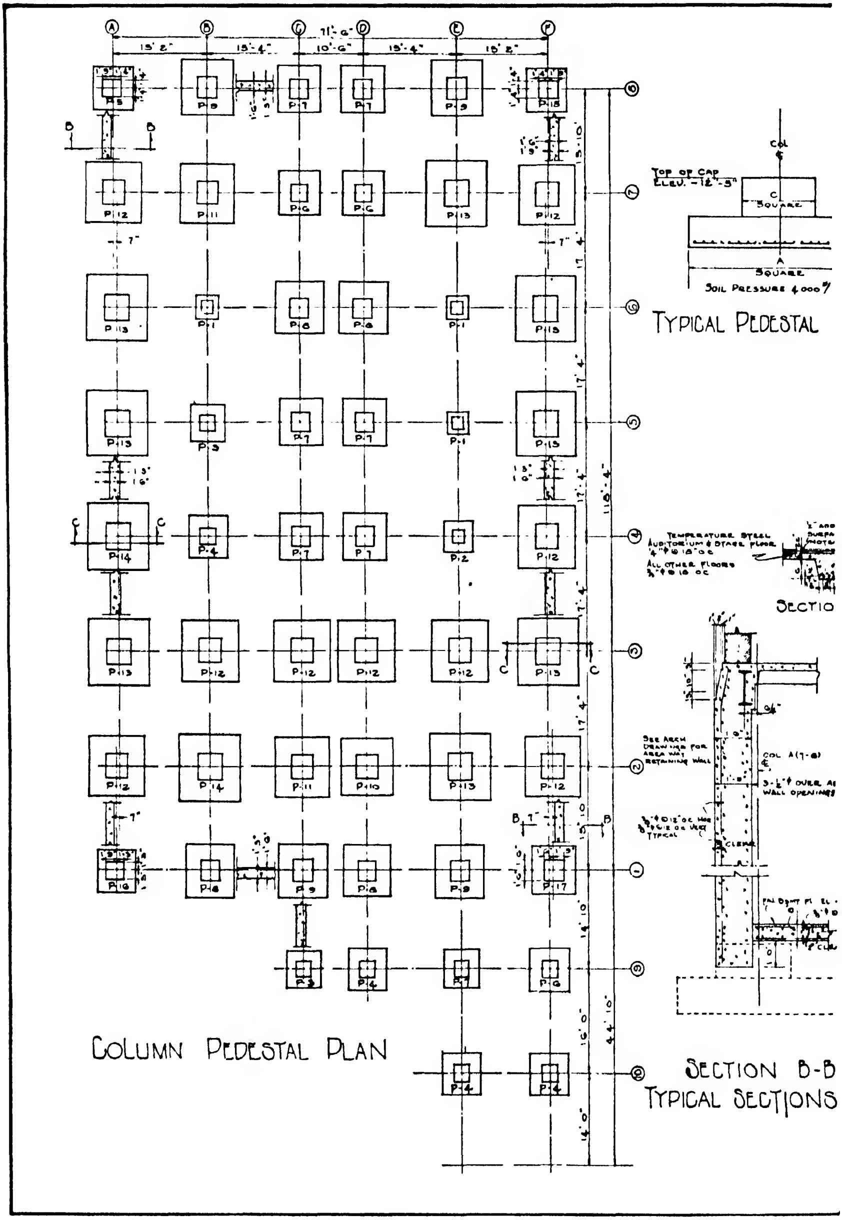


Fig. 222A

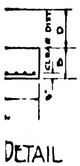
Courtesy of A. T. & S. F. Railway System

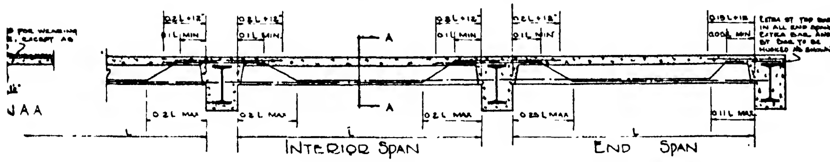
CONCRETE PEDESTAL SCHEDULE					
PEDESTAL NUMBER	DIMENSIONS A	B	C	D	STEEL EACH WAY
P-1	3'-2"	0'-9"	2'-0"	1'-0"	10-½"φ
P-2	4'-10"	0'-11"	2'-0"	1'-0"	16-½"φ
P-3	5'-8"	1'-0"	2'-6"	1'-0"	20-½"φ
P-4	6'-4	1'-4'	2'-6"	1'-0"	20-½"φ
P-5	6'-8"	1'-4	See Plan	4'-1"	21-½"φ
P-6	6'-8"	1'-4	2'-4	1'-3"	21-½"φ
P-7	7'-3"	1'-5'	3'-0"	1'-3"	21-½"φ
P-8	7'-10	1'-7"	3'-0"	1'-3"	21-½"φ
P-9	8'-2"	1'-9"	3'-8"	1'-3"	21-½"φ
P-10	8'-8"	1'-11'	4'-0"	1'-3"	21-½"φ
P-11	9'-0"	1'-11"	4'-0"	1'-3"	22-½"φ
P-12	9'-5"	2'-1"	4'-0"	1'-6"	23-½"φ
P-13	9'-10"	2'-2"	4'-0"	1'-6"	24-½"φ
P-14	10'-2"	2'-3"	4'-0"	1'-6'	27-½"φ
P-15	6'-8"	1-4"	See Plan	1'-3"	21-½"φ
P-16	6'-4'	1'-4"	See Plan	1'-0'	20-½"φ
P-17	7'-3'	1'-3"	See Plan	1'-3"	21-½"φ

JOIST SCHEDULE					
JOIST MARK	SIZE	WIDTH	C. C	STEEL See Detail	REMARKS
J-1	6+2½	5"	25	2-⅝"φ	Single Tapered End Forms
J-2	6+2½	5"	25	2-½"□	" "
J-3	6+2½	5"	25	2-¾"φ	"
J-4	6+2½	5"	25	2-⅝"φ	" "
J-5	6+2½	5"	25	2-½"φ	" "
J-6	6+2½	5"	25	2-⅞"φ	" "

NOTE:

Steel pan construction to be made with removable corrugated metal forms of not less than 18 gauge thickness.
If preferred, Meyer Steel Forms, or similar, of not less than 18 gauge may be used.
Double joists and headers around all openings.
Where pipes cut joists, headers shall be used.
Wire conduits placed in slab are not to exceed ¾" in dia and shall not cross.
In the panels where shower rooms or toilets occur lower the joist in these panels 2"





TYPICAL BENDING DETAIL OF JOIST STEEL

NOTE·

Contractor will furnish Bridge Engineer System with working drawings of reinforcement and concrete floor construction for approval.
Temperature steel in floor shall extend continuously thru middle of joist flange, transversely to joist, with joints staggered and shall extend over the top of flange of spandrel beams.
All reinforcing steel shall be held up by substantial bar chairs.
Contractor to furnish all spacer bars necessary.
Steel fabricator to furnish Bridge Engineer System with two sets of blue prints of all structural details for approval.

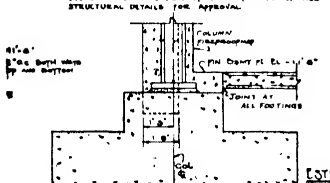
GENERAL NOTES FOR STRUCTURAL STEEL

Steel. Material & Fabrication A.R.E.A. Specifications
Rivets ¾"φ unless noted
Reaming. None

Shop Paint: One coat of boiled linseed oil. Steel work to be entirely embedded in concrete shall not be painted. Where part of the beam or column is exposed the whole shall be painted, with two coats of red lead. No fitting up paint.
Design: A.R.E.A. Specifications for Buildings.
Live Loads: Roof 25 #/□', Storage rooms 150 #/□', Balcony, Auditorium floor, Stage, Kitchen, Library, File rooms and all corridors and stairs 100 #/□', All others 50 #/□'.

GENERAL NOTES FOR CONCRETE

Concrete shall be mixed and placed in accordance with the A T & S F Ry System "Employes Concrete Manual," C.E.S. 50/53-22.
Ultimate compressive strength of concrete to be 3000 #/□" in 28 days, except for pedestals where compressive strength shall be 2000 #/□" in 28 days.
Reinforcing bars to be deformed bars, open hearth steel, intermediate grade, A.S.T.M. Specification A15-14.



SECTION C-C
THROUGH BASEMENT WALL

ESTIMATE OF QUANTITIES

Footings – Concrete 270 cu yds
Reinforcing Steel 6 Tons

Structural Steel 280 Tons

Fig. 222B
Courtesy of A. T. & S. F. Railway System

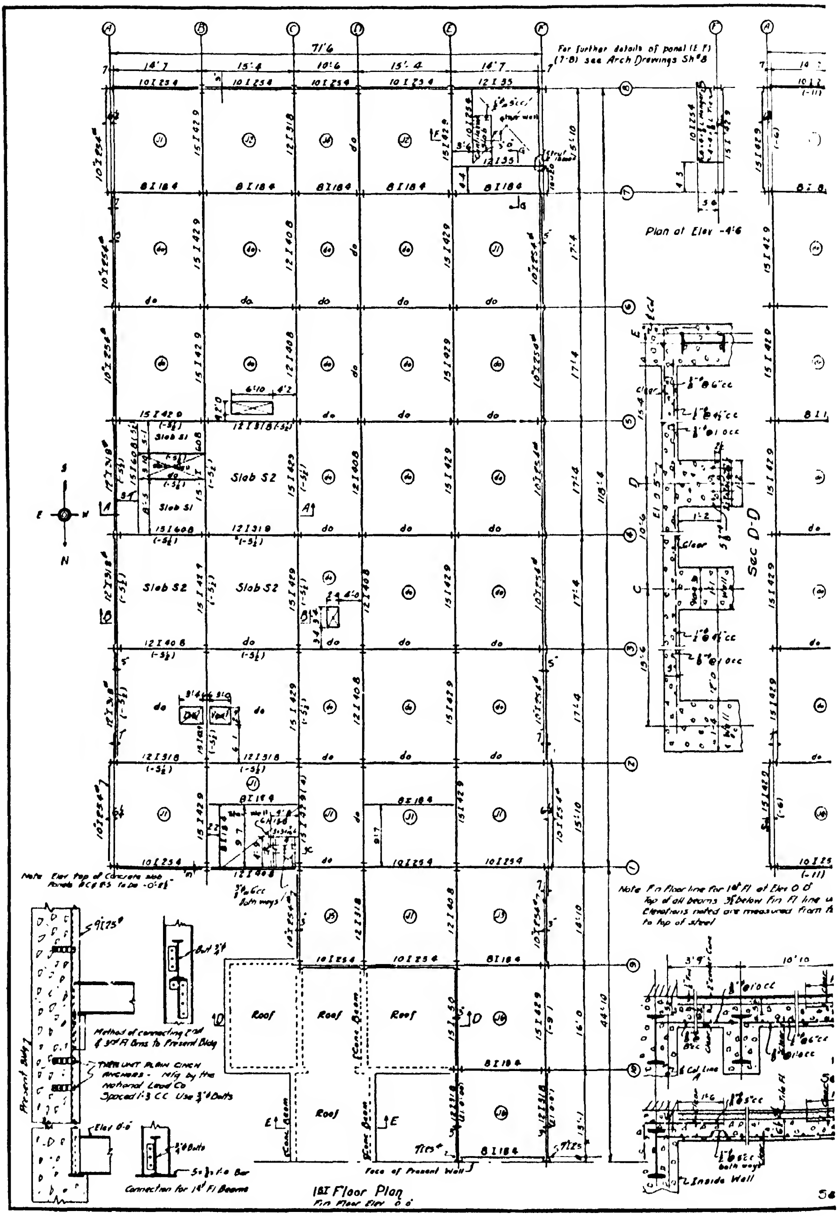


Fig. 223A
Courtesy of A. T. & S. F. Railway System

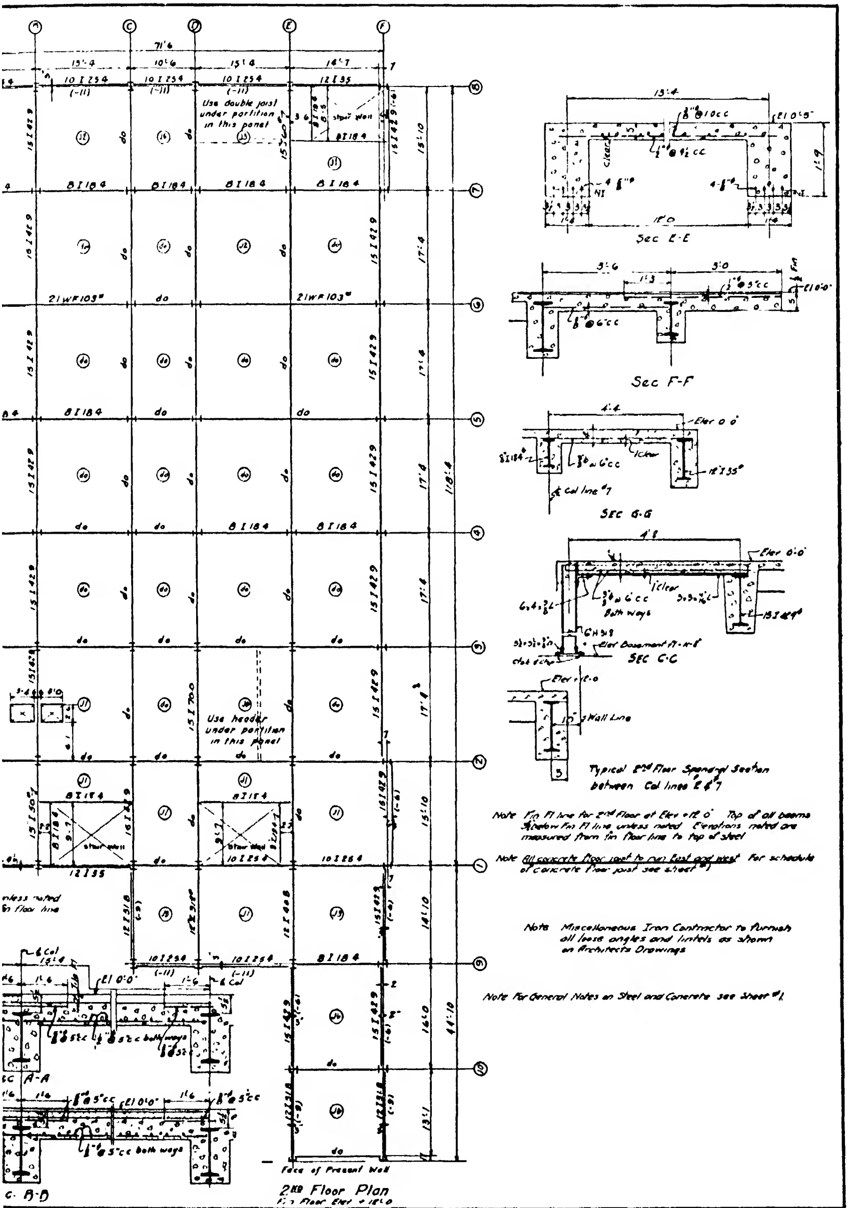


Fig. 223B
Courtesy of A. T. & S. F. Railway System

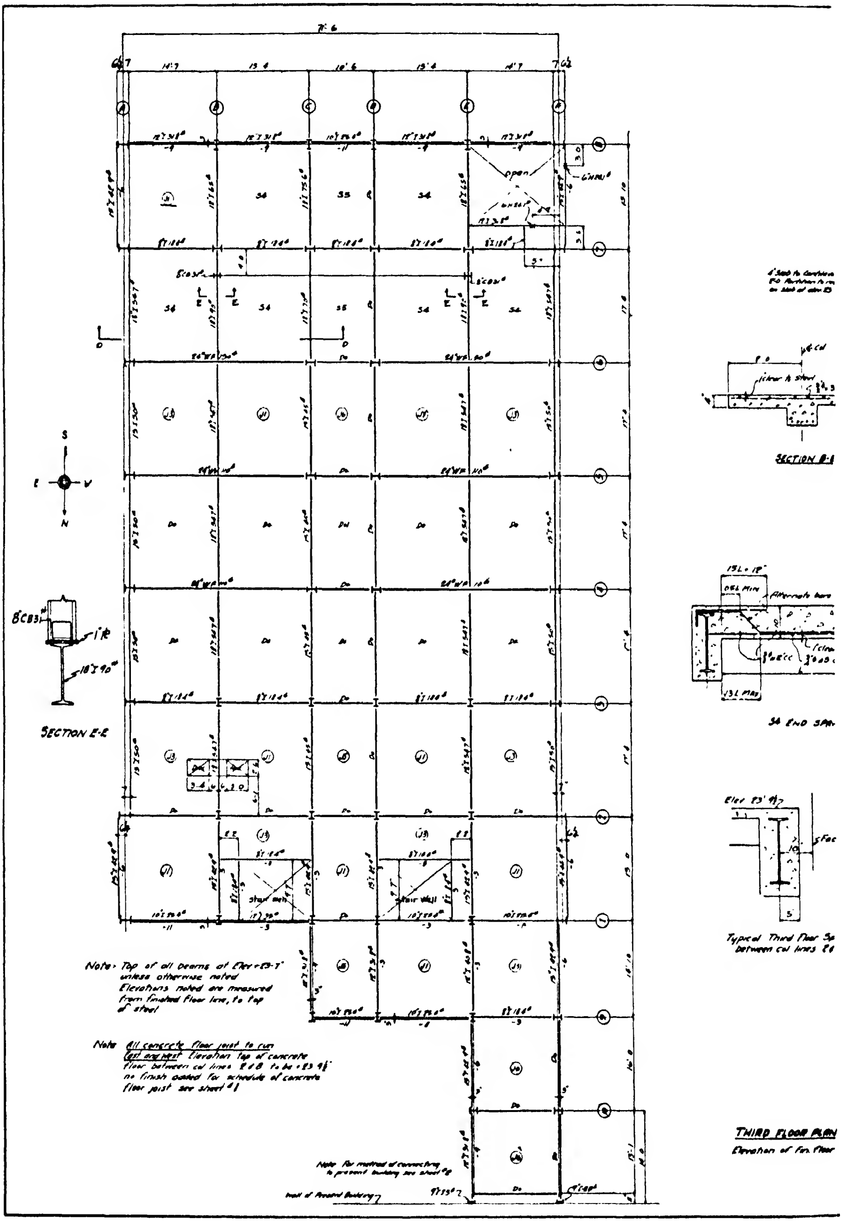


Fig. 224A
Courtesy of A. T. & S. F. Railway System

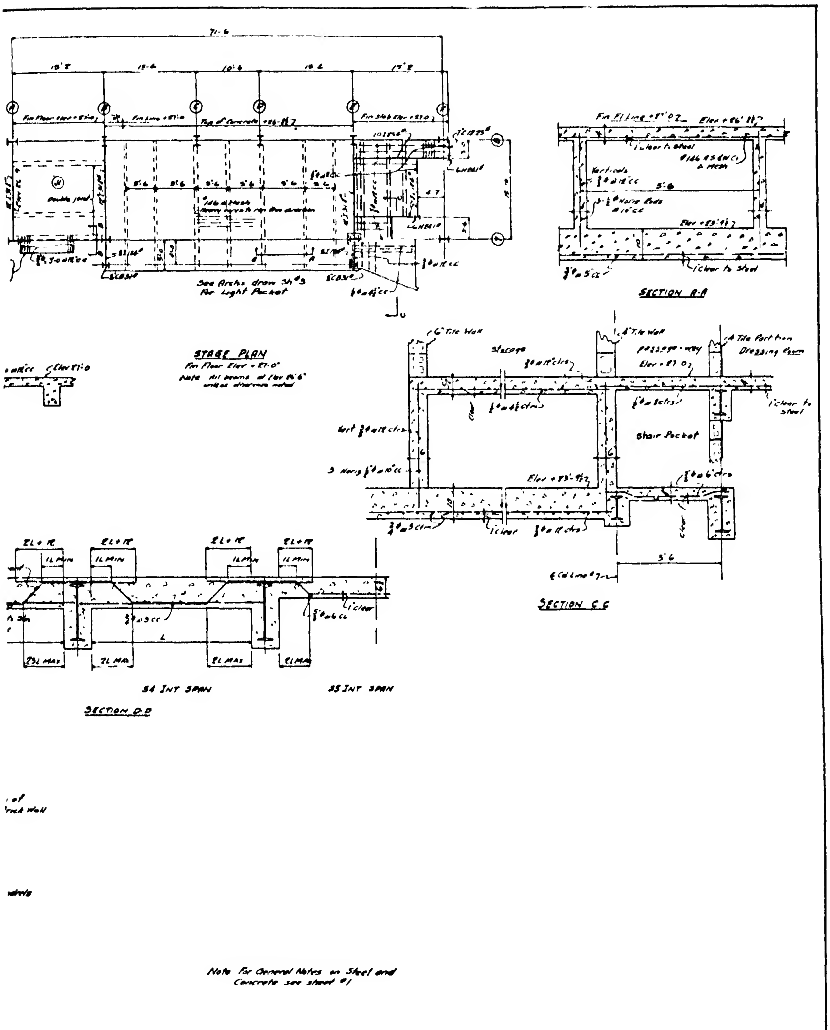


Fig. 224B
Courtesy of A. T. & S. F. Railway System

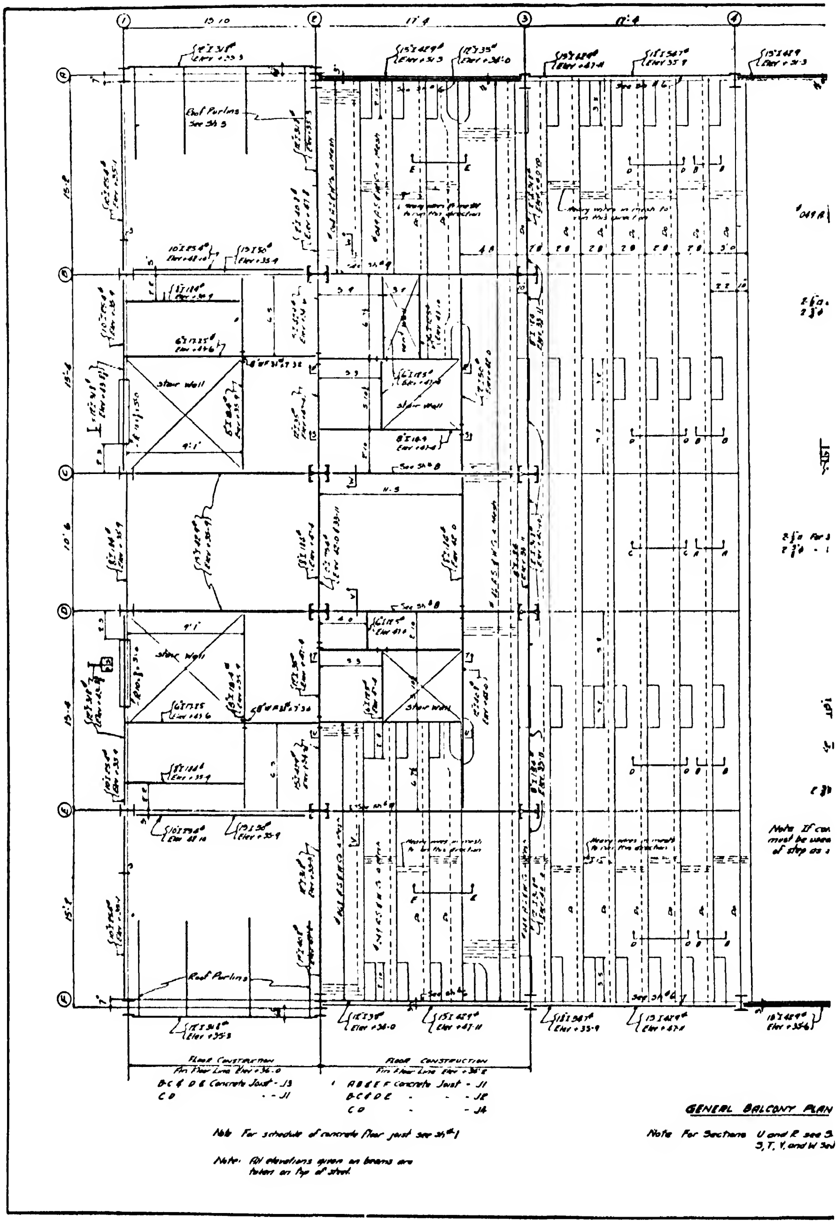


Fig. 225A
Courtesy of A. T. & S. F. Railway System

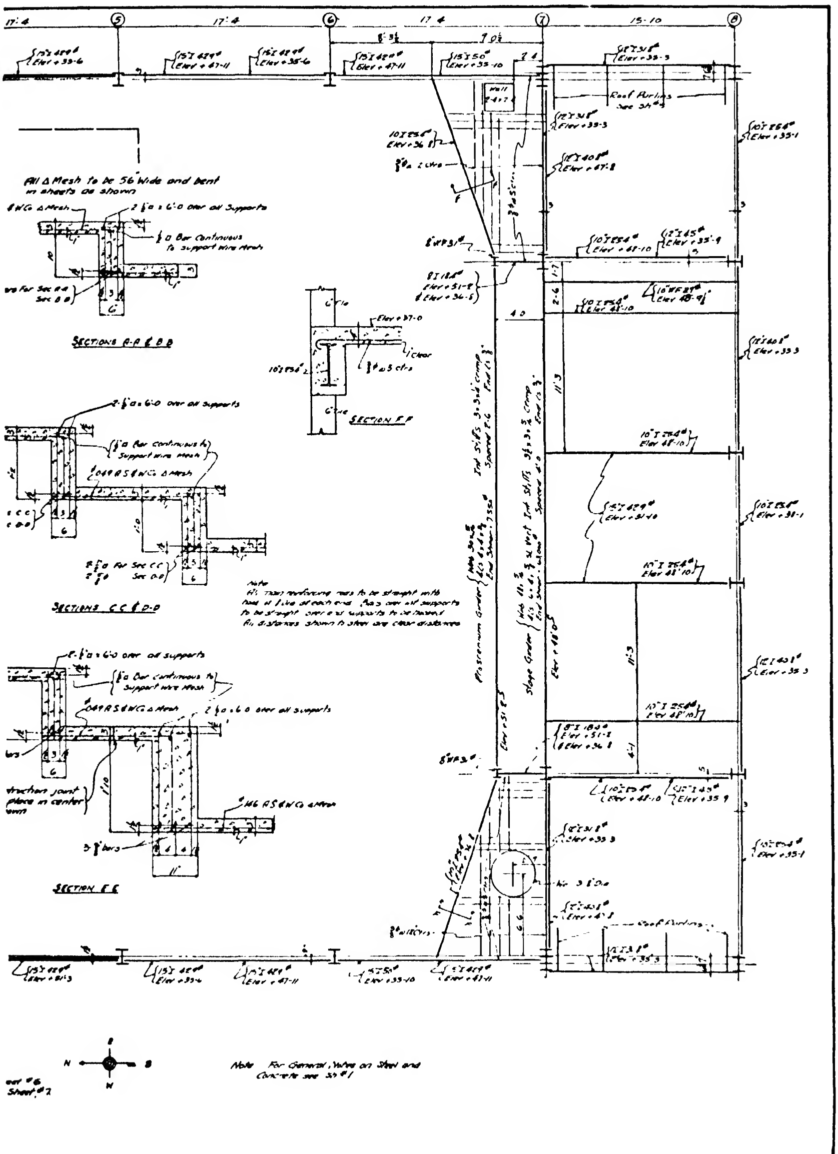


Fig. 225B
Courtesy of A. T. & S. F. Railway System

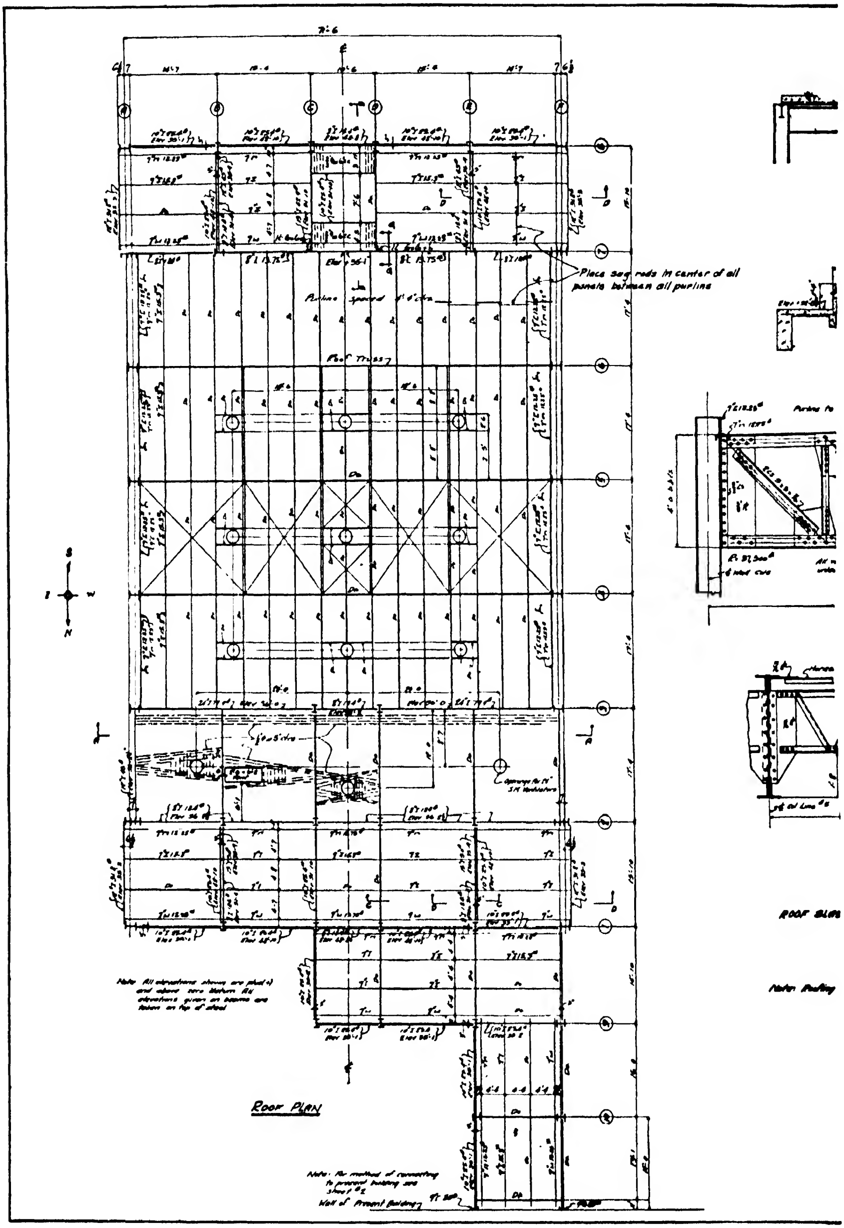


Fig. 226A
Courtesy of A. T. & S. F. Railway System

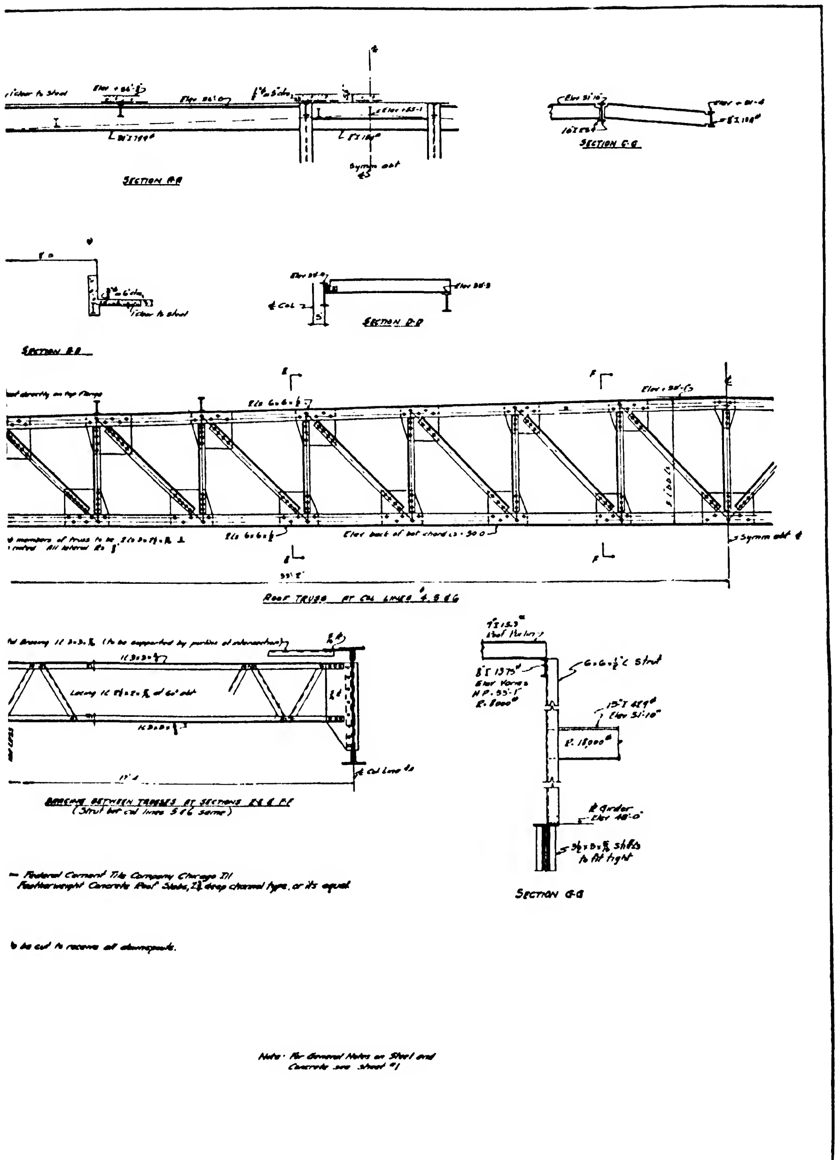


Fig. 226B
Courtesy of A. T. & S. F. Railway System

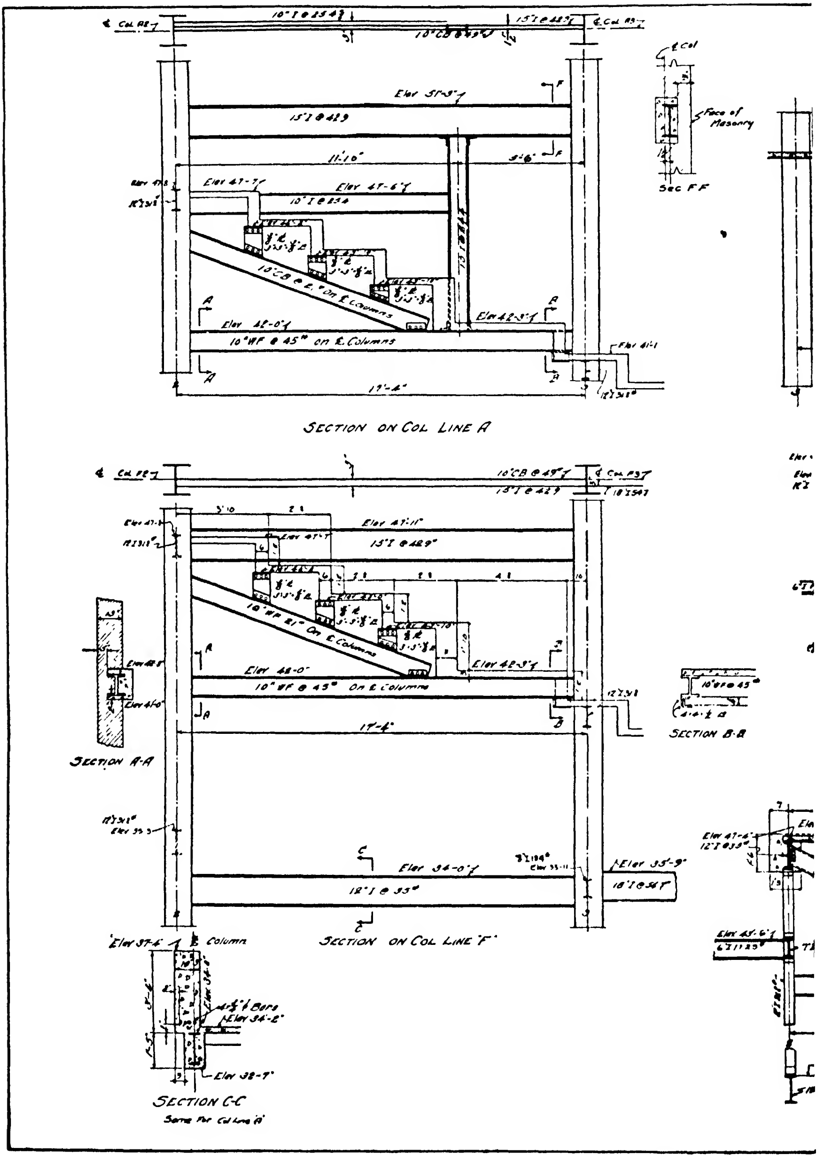


Fig. 227A
Courtesy of A. T. & S. F. Railway System

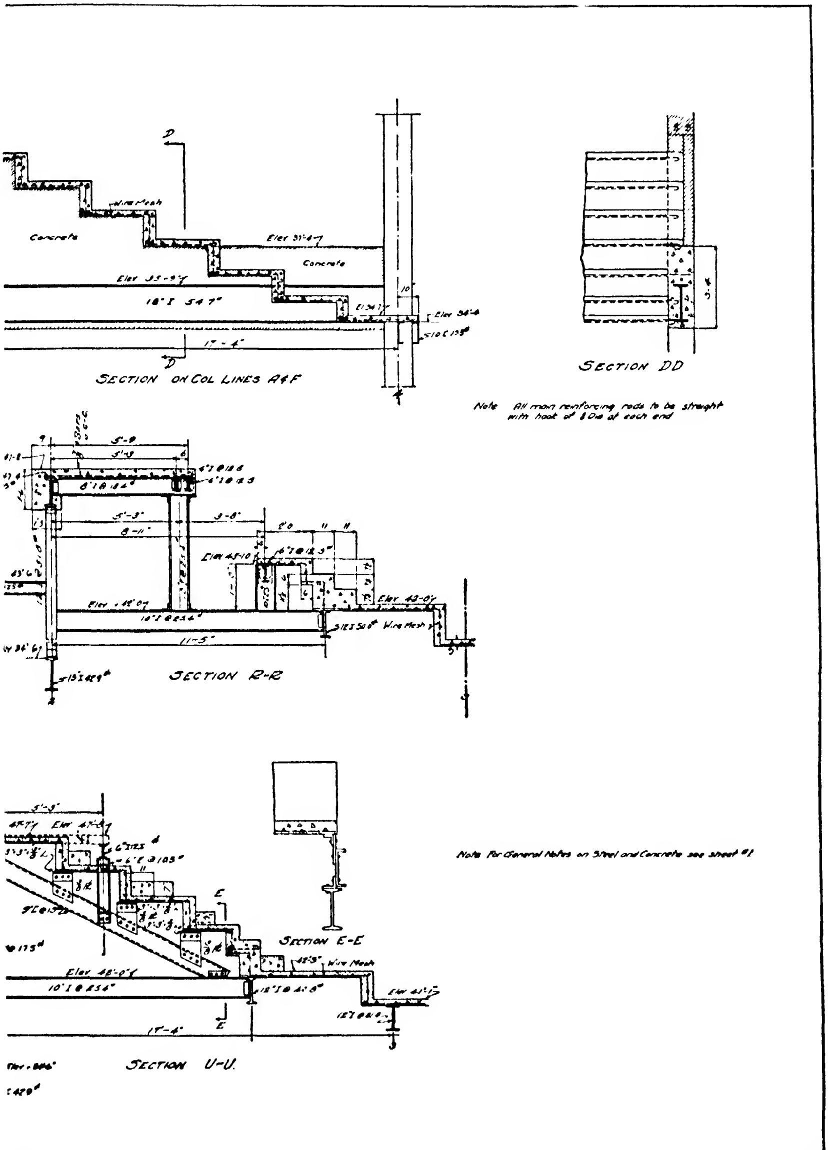


Fig. 227B
Courtesy of A. T. & S. F. Railway System

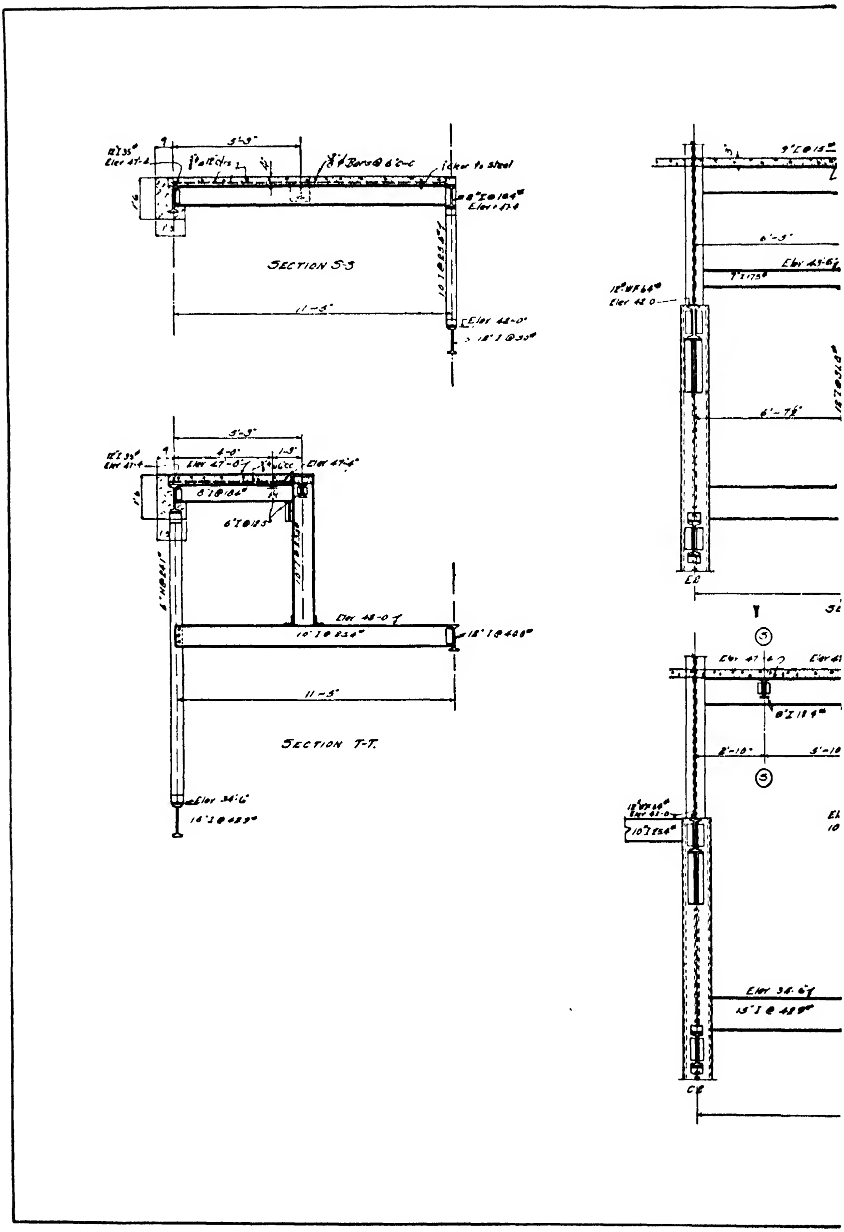


Fig. 228A

Courtesy of A. T. & S. F. Railway System

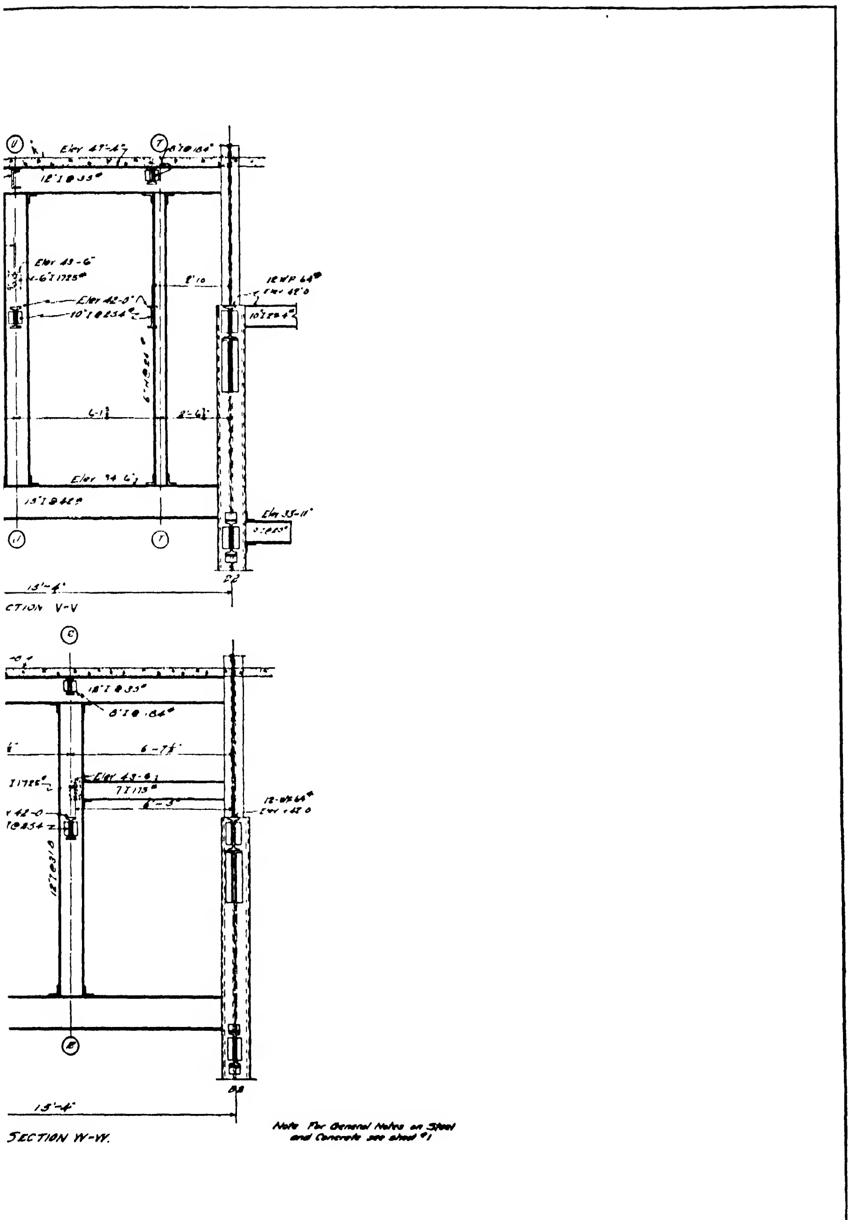


Fig. 228B
Courtesy of A. T. & S. F. Railway System

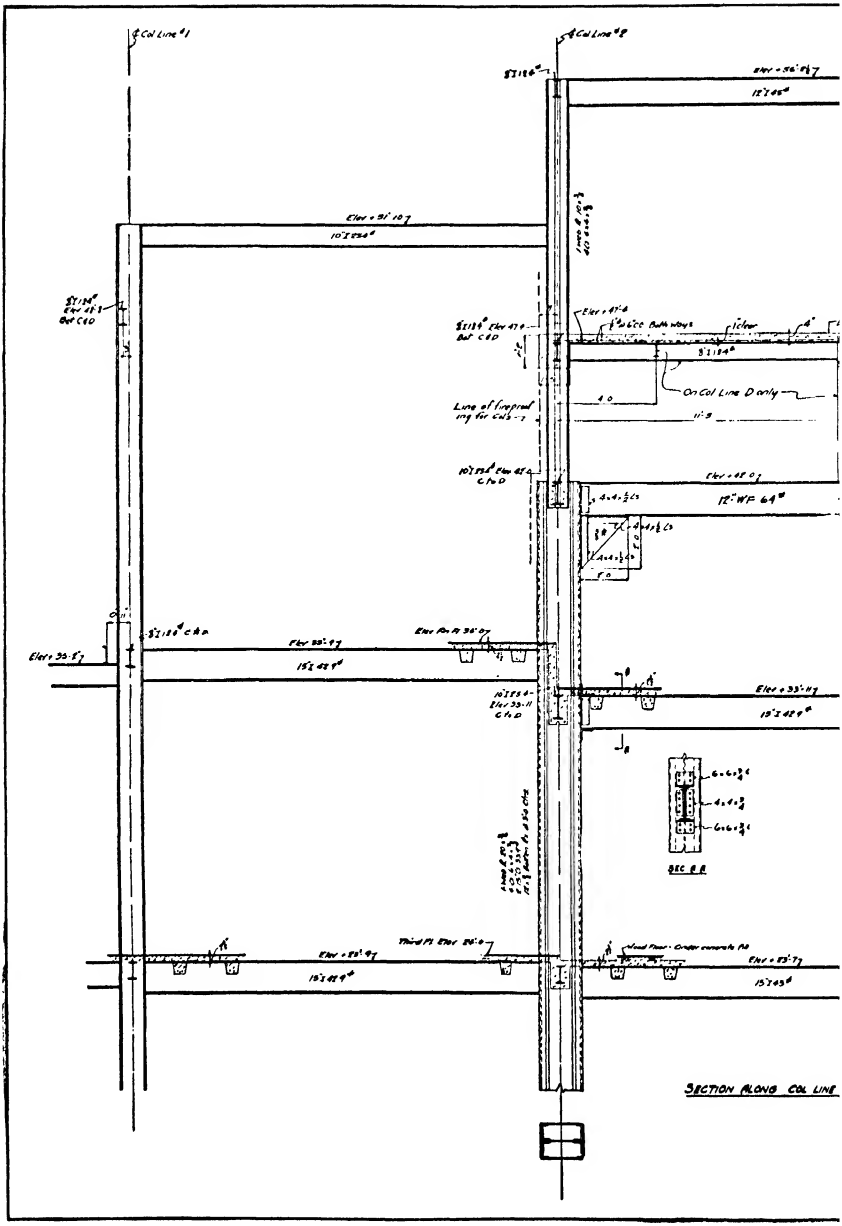


Fig. 229A

Courtesy of A. T. & S. F. Railway System

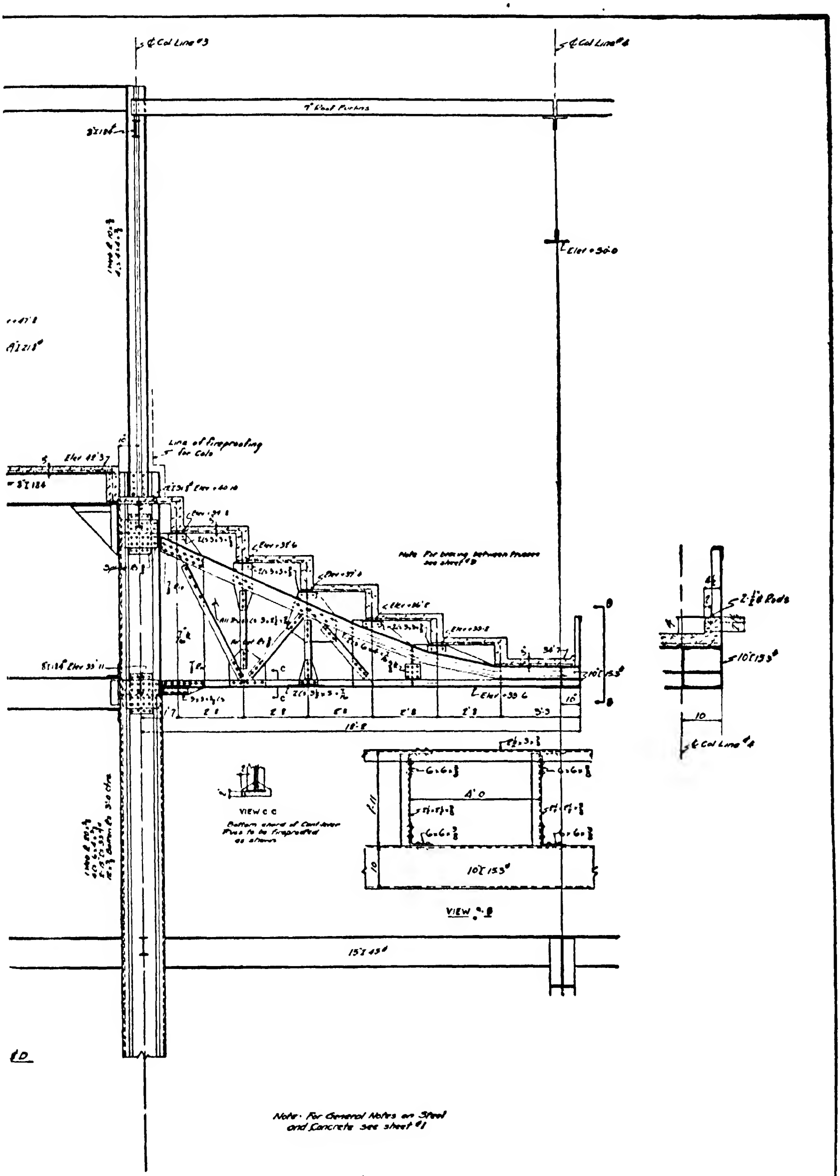


Fig. 229B
Courtesy of A. T. & S. F. Railway System

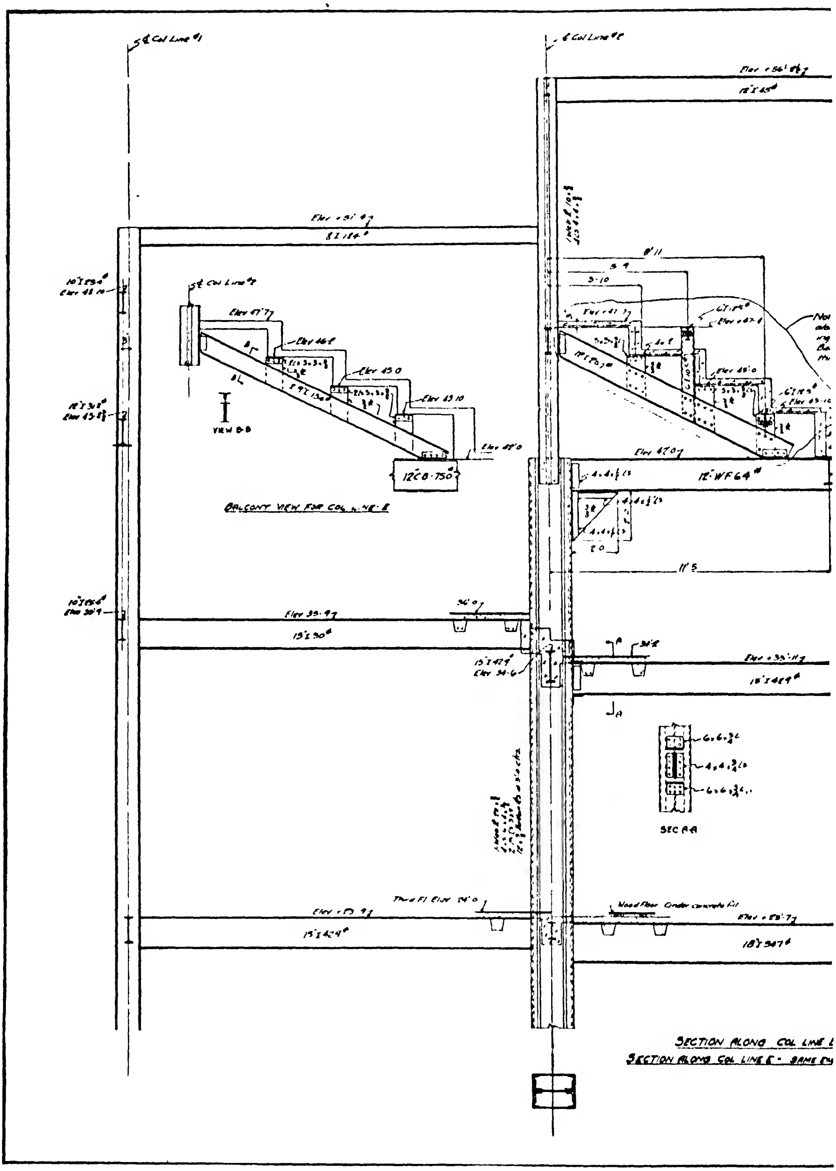


Fig. 230A
Courtesy of A. T. & S. F. Railway System

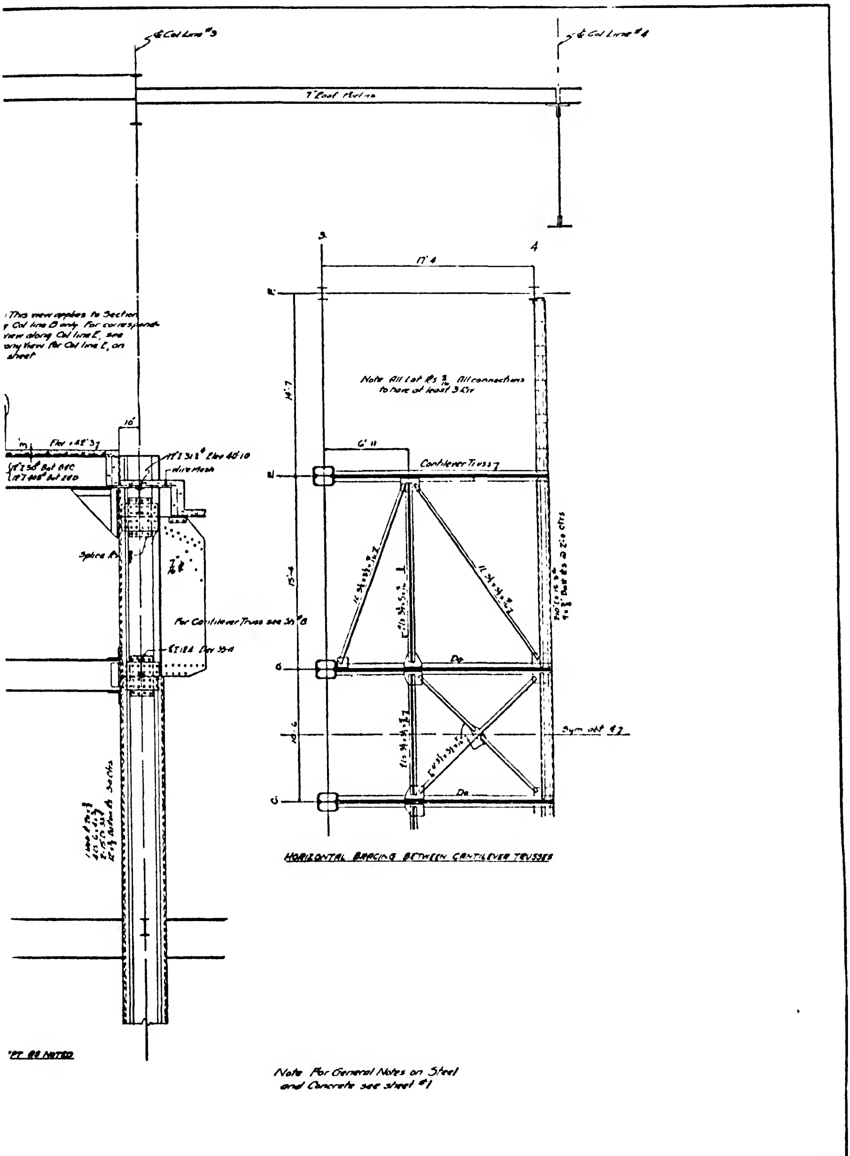


Fig. 230B
Courtesy of A. T. & S. F. Railway System

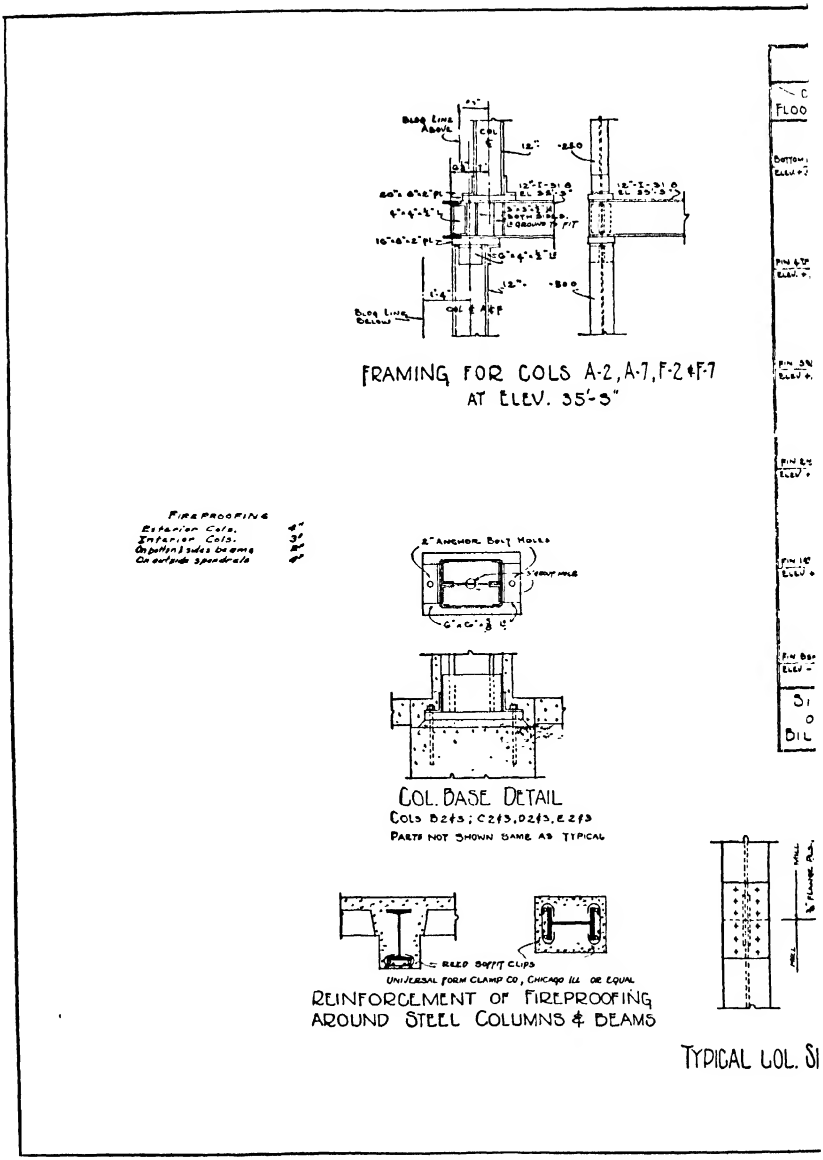


Fig. 231A
Courtesy of A. T. & S. F. Railway System

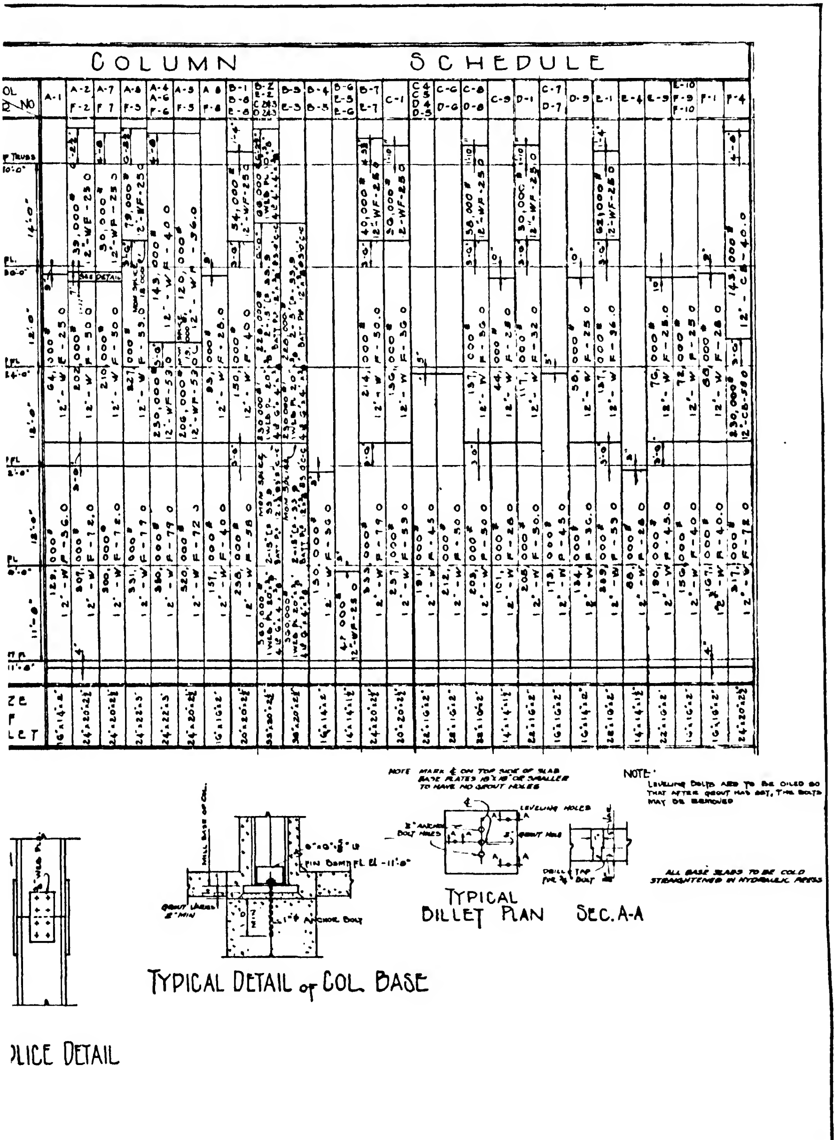


Fig. 231B

Courtesy of A. T. & S. F. Railway System

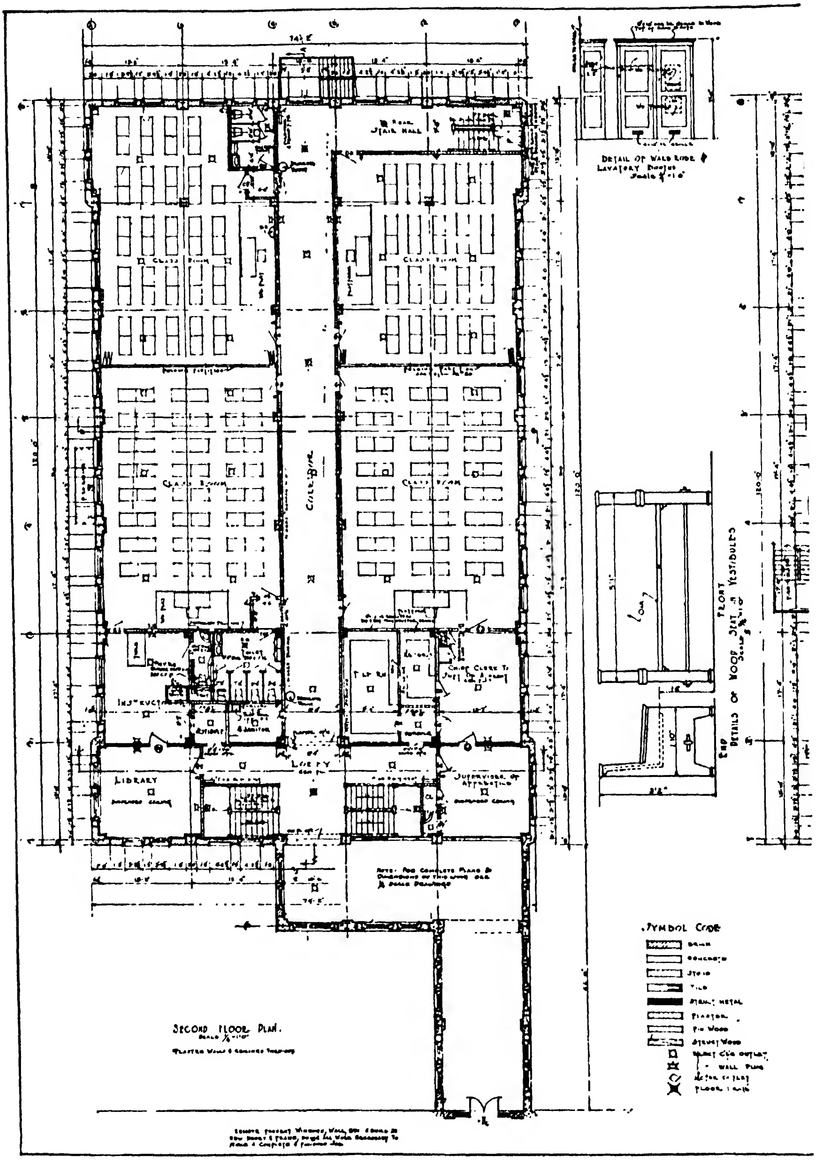


Fig. 232A

Courtesy of A. T. & S. F. Railway System

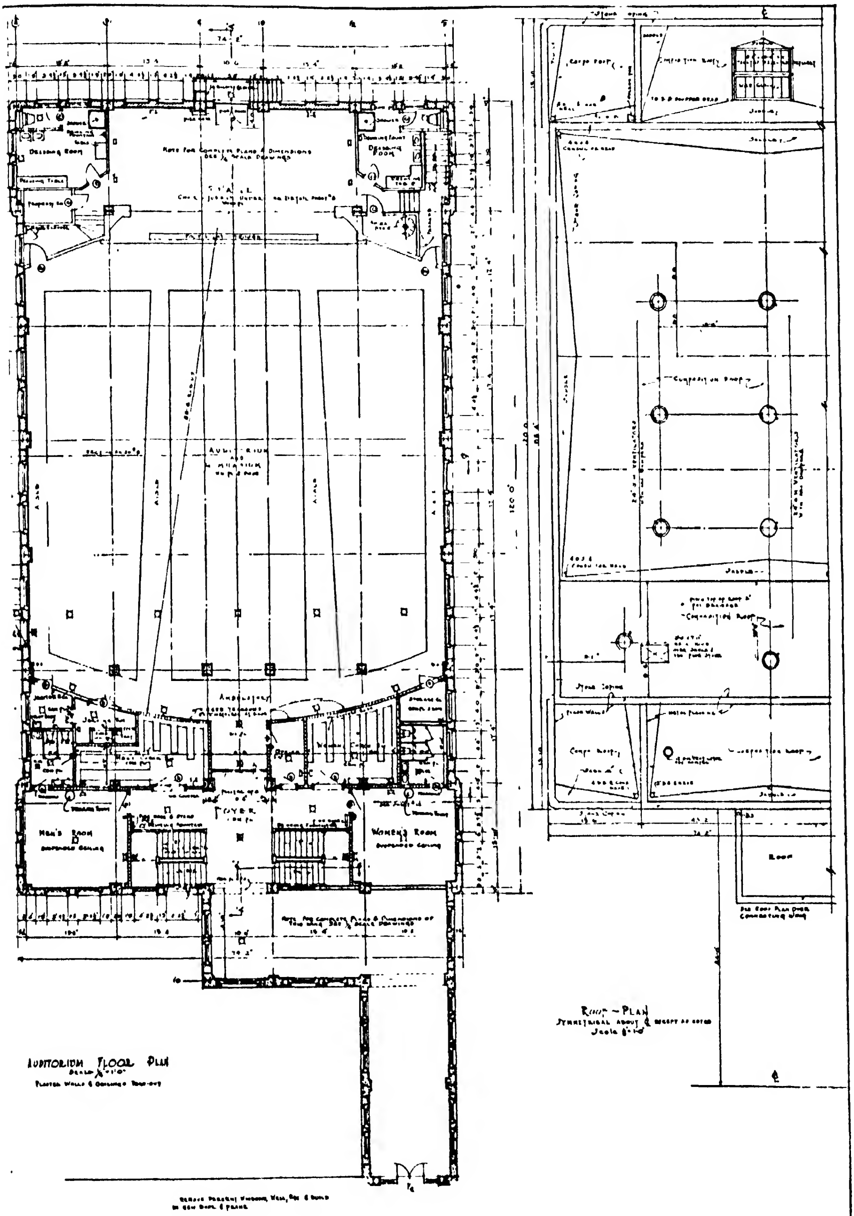


Fig. 232B
Courtesy of A. T. & S. F. Railway System

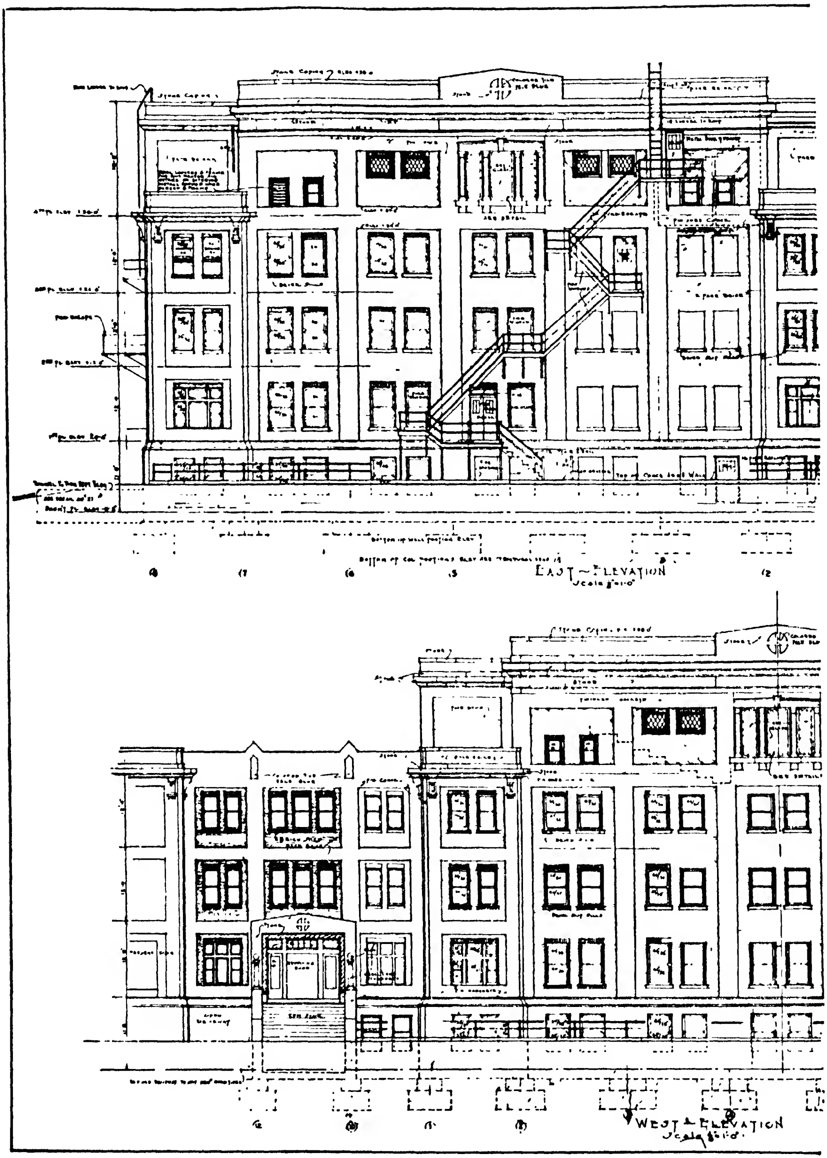


Fig. 233A
Courtesy of A. T. & S. F. Railway System

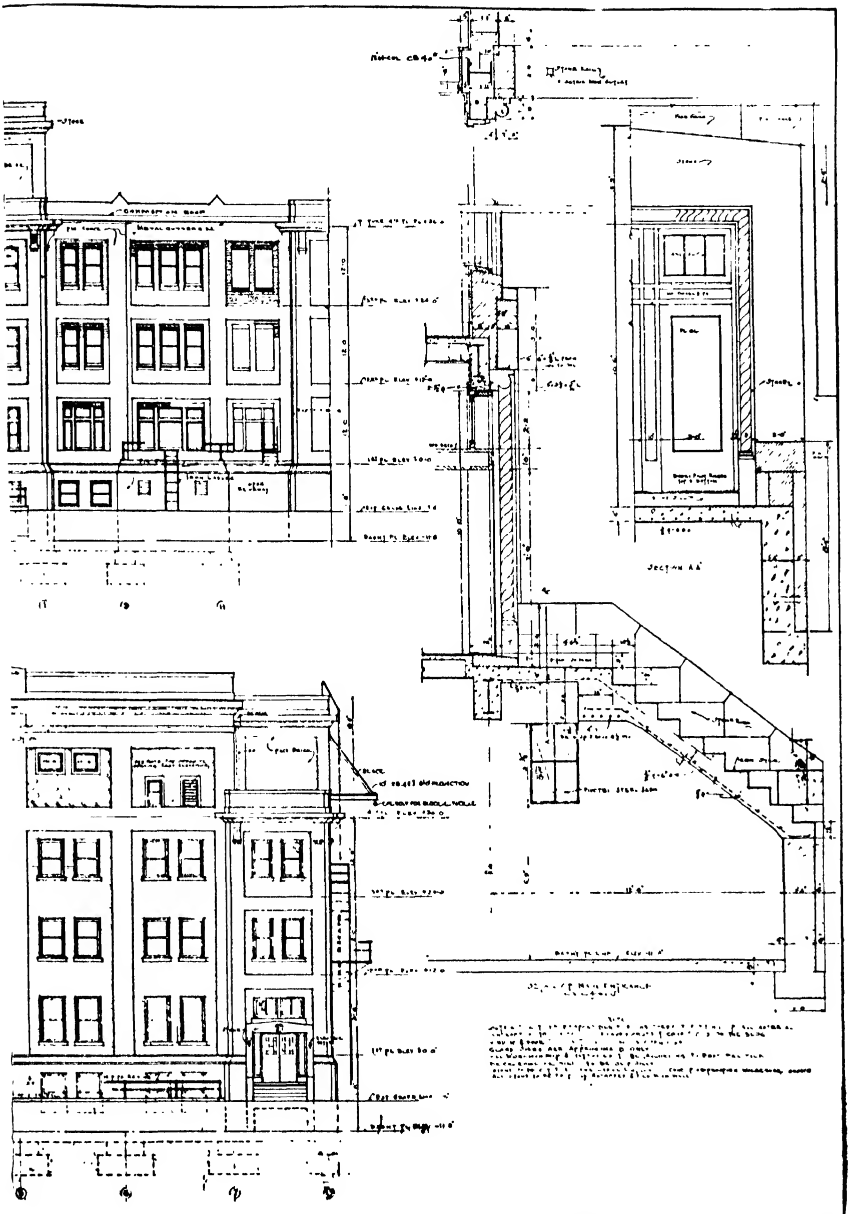


Fig. 233B
Courtesy of A. T. & S. F. Railway System

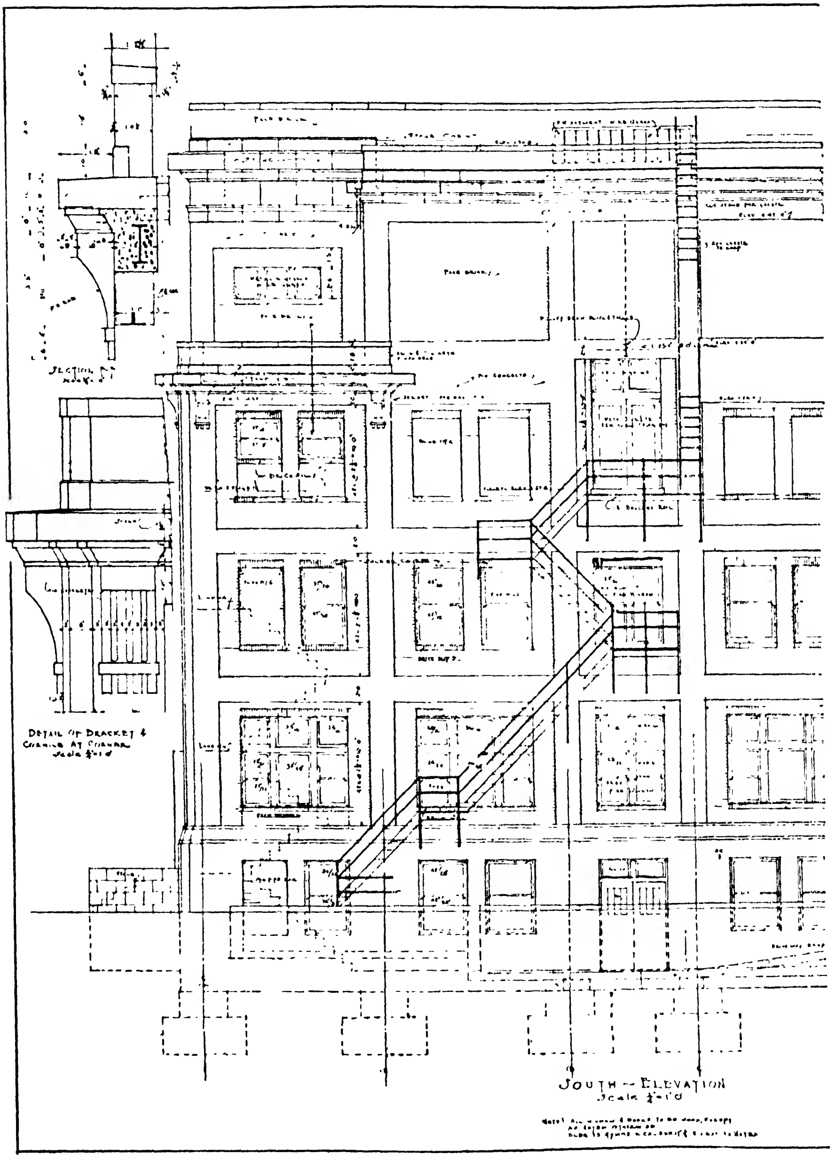


Fig. 234A
Courtesy of A. T. & S. F. Railway System

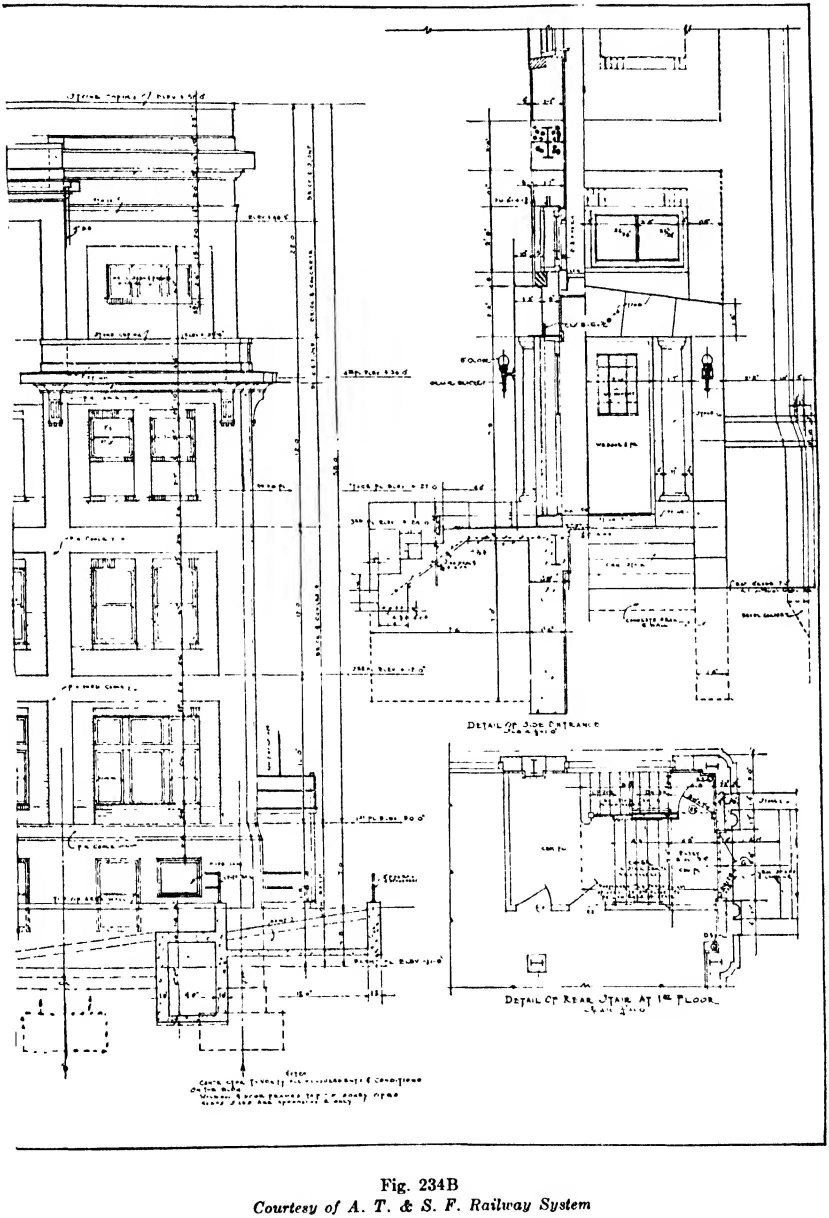


Fig. 234B
Courtesy of A. T. & S. F. Railway System

Fig. 235A

Courtesy of A. T. & S. F. Railway System

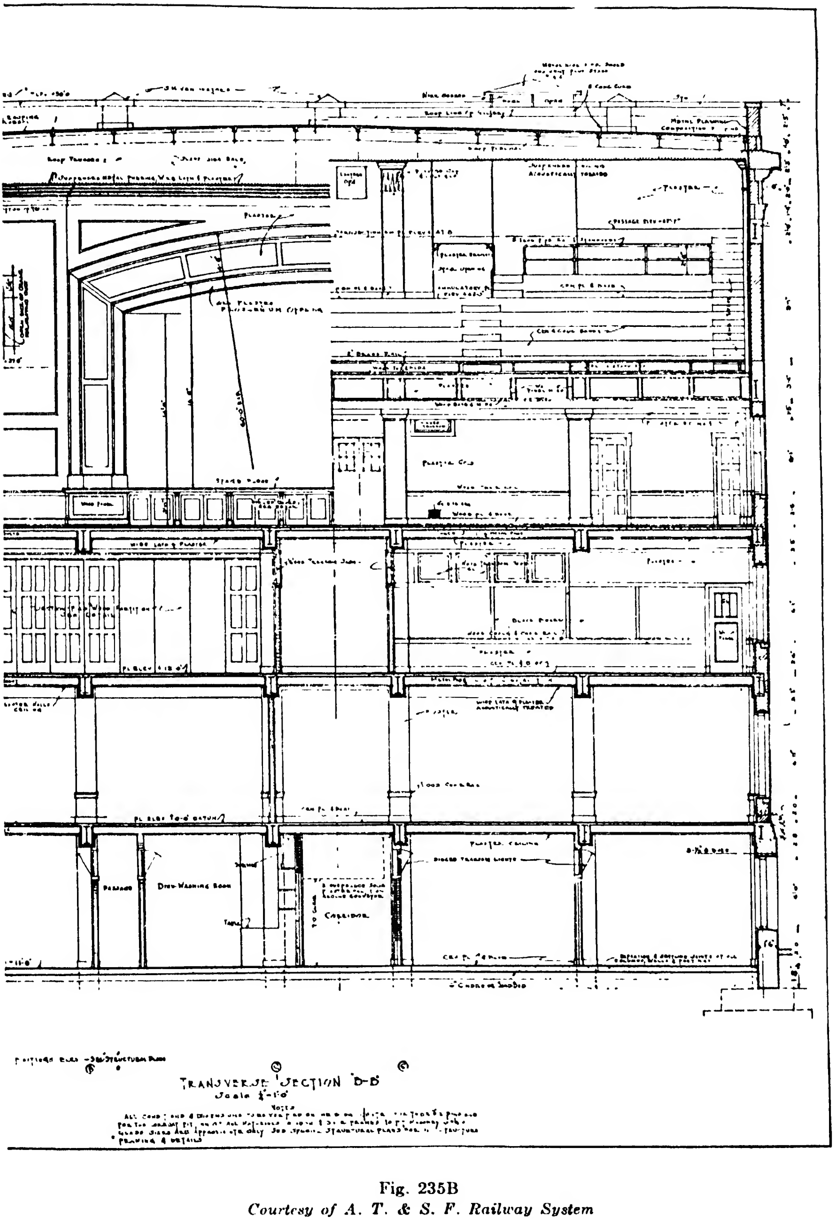


Fig. 235B
Courtesy of A. T. & S. F. Railway System

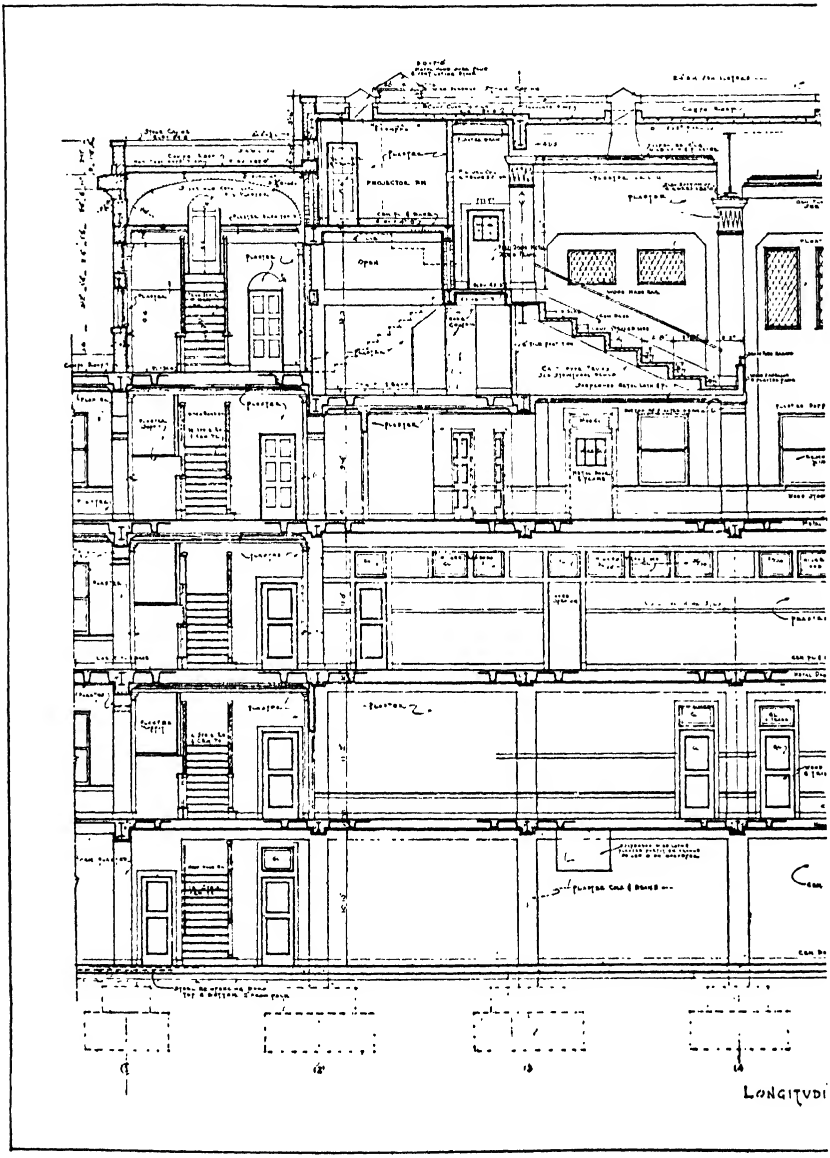


Fig. 236A
Courtesy of A. T. & S. F. Railway System

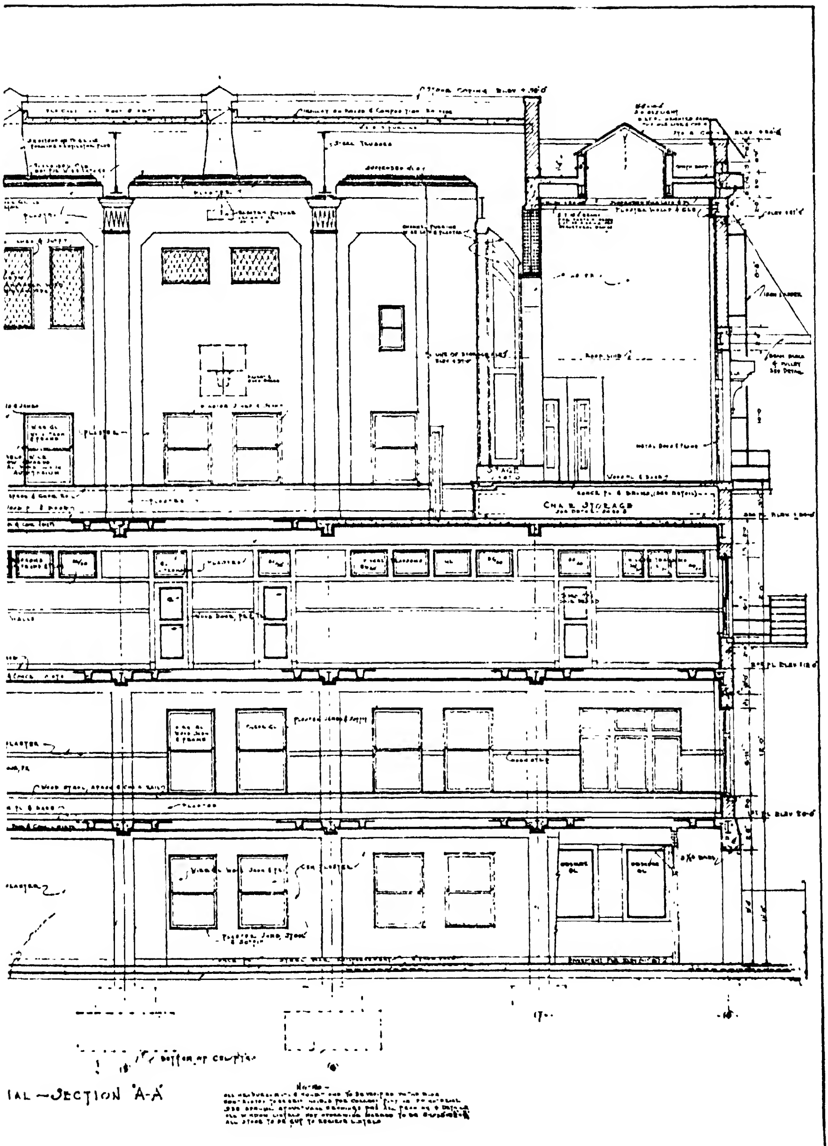


Fig. 236B
Courtesy of A. T. & S. F. Railway System

The arrangement used gives a large number of duplications.

Special framing is required around openings for stair wells, dumb waiters, vent and flue stack, etc. Special framing is also required for the balcony and stage.

Each building has its special conditions affecting the placing of the beams and columns. Flat ceilings, permitting no projecting beams, may govern the arrangement. The use of reinforced concrete floors with rods in two directions requires beams on all four sides of the panels.

Arrangement of Joists. Having established beam lines, the joists, if used, are spaced as uniformly as practicable. A joist or strut 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 floor used in this building, no joists are required.

For struts between the columns, transverse to the line of the main beams, 8-inch I beams are used. These struts brace the columns transversely, keep the columns plumb during erection, and carry the floor load and the load of any partition directly above.

Beam Elevations. The elevations of the beams are given in reference to the elevations of the finished 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 the flooring, whether of wood, marble, finished concrete, 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 this school building the tops of the beams are down $3\frac{1}{2}$ inches from the finished floor where concrete floor surface is used, and 5 inches where there is wood floor.

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 will be 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 concrete, combination tile and concrete, and steel 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 so much by the elevation of the floors. In some cases these beams serve as the lintels over the windows and their elevations are fixed accordingly.

For flat roofs the beams may be set on a slope and a suspended level ceiling provided, or the beams may be set level and the roof built up to give a slope.

Arrangement of Columns. It is desirable that the columns be arranged in rows across the building in both directions, but this may be prevented by the layout of the rooms in the building. The column spacing is also affected by the design of the exterior and the architectural requirements.

The distances of the exterior columns from the building lines are governed either by architectural requirements or the thickness of the fireproofing.

Specifications. The American Railway Engineering Association Specifications were used for the design, fabrication, and erection of this building. The structural steel is in accordance with the specifications of the American Society for Testing Materials. The concrete is according to the A. T. & S. F. Ry. Specifications which are very similar to the A.R.E.A. Masonry Specifications.

The unit stresses allowed by the 1927 A.R.E.A. Structural Steel Specification are as follows.

Structural Steel	Pounds per Square Inch
Axial tension, net section	18,000
Axial compression, gross section	$18{,}000-60l/r$
but not to exceed	15,000
l = the length of the member in inches	
r = the least radius of gyration of the member in inches.	
Tension in extreme fibers of rolled shapes, built sections and girders, net section	18,000
Tension in extreme fibers of pins	27,000
Shear in plate girder webs, gross section	12,000
Shear in power-driven rivets and pins and turned bolts in reamed holes	13,500

Structural Steel	Pounds per Square Inch
Shear in hand-driven rivets and unfinished bolts	10,000
Bearing of outstanding legs of stiffener angles and similar steel parts in contact	27,000
Bearing on power-driven rivets, pins, turned bolts in reamed holes	
(single shear)	24,000
(double shear)	30,000
Bearing on hand-driven rivets, countersunk rivets and unfinished bolts,	
(single shear)	16,000
(double shear)	20,000
Bearing on expansion rollers, per lineal inch	$600d$

d = the diameter of the rollers in inches.

For cast steel in shoes and bearings, the above mentioned stresses shall apply.

Other parts of this Specification will be noted and discussed where they apply in the design computations. The Building Committee of the A.R.E.A. is now revising the structural steel design specifications and it appears that when completed it will be in line with the A.I.S.C. Specifications.

Selection of Floor Type. As discussed in the section on floors, the selection of the type of floor construction is affected by a number of items. The concrete slab and joist type with removable metal pans is used for this building on account of its economy, all conditions being considered.

Fireproofing. The general subject of fireproofing is discussed in Chapter XII. 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 beams and columns, and can be placed easily in connection with the concrete used in the floor construction.

The fireproofing affects the steel design through the weight of material to be supported, and through the locations of the steel members in relation to the openings, as allowance must be made for the thickness of fireproofing. The thicknesses used are the same as required by the Chicago Building Ordinance and are as follows:

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″

In general the spandrel girders are placed as near the outer face of the wall as the fireproofing requirements will permit. In order to provide support farther out, shelf angles or plates are used, projecting no nearer to the outside face than 2 inches.

Around openings in the interior, 2 inches of covering is required for fireproofing, and usually 1 inch is needed for plaster, stair fascia, or other finish. To these must be added the half width of the beam to get the distance from finished edge of opening to center of beam. The actual amount required varies for different sizes of beams.

Loads on the Building Illustrated. In the school building the following live loads were used.

Roof	25# per sq. ft.
Storage rooms	150# per sq. ft.
Balcony, auditorium, and stage	100# per sq. ft.
Kitchen, library, and file rooms	100# per sq. ft.
Stairs and corridors	100# per sq. ft.
All other floors	50# per sq. ft.

DESIGN OF STEEL MEMBERS

Design of Beams. After the spacing of the columns, arrangement of girders or beams, and type of construction have been decided, the next step is the design of the beams and columns.

Due to the fireproofing of the beams with concrete, it was more economical to use I beams than WF beams, as the flanges are not so wide. WF beams are used only in special cases. For this building increases in steel stresses were not allowed due to the encasement; therefore, the regular stresses allowed by the A.R.E.A. Specifications (1927) are used. All computations are made with a slide rule.

First Floor Girders and Beams. *Girders (2-3) (3-4) (4-5) (5-6) (6-7) Line E.* The floor in this portion of the building has a $\frac{1}{2}$-inch wearing surface on top of the slab and joist construction. The dead load per square foot is as follows:

Wearing surface	6#
Floor construction	46#
Plastered ceiling	10#
Total	62#

The live load per square foot, as applied to the girders, equals 85% of 50 lb. or 42.5 lb. The total dead plus live load carried by the girders then equals 104.5# per square foot. For ease in calculation, 104# will be used. These beams support half of each adjacent floor panel, which is the following width.

$$\tfrac{1}{2}\times(15'\text{-}4''+14'\text{-}7'')=14'\text{-}11\tfrac{1}{2}'', \text{ say 15 feet}$$

The span length of these girders is 17′-4″.

The load per linear foot on these girders will then be:

Assumed weight of steel girder	50#
Fireproofing	300#
Floor load, 104#×15′	=1560#
Total	1910#

Bending moment equals

$$M=\tfrac{1}{8}\,w\,l^2$$
$$=\tfrac{1}{8}\times1910\times(17.34)^2\times12$$
$$=861{,}000''\#$$

Section modulus of the required girder equals

$$S=\frac{M}{f_s}$$

$$=\frac{861{,}000}{18{,}000}=47.9$$

Use a 15″ I 42.9#. The section modulus of this beam is 58.9. A 12″ I beam of sufficient strength would weigh 50# per foot.

Girder (1-9) Line E. This girder supports a portion of the corridor on which the live load is 100# per square foot. The span length is about 14 feet, as connections are made to the column flanges. The dead load per square foot is 62#. The live load applied to the girder is 85% of 100# or 85# per square foot. Total dead and live load then equals 147# per square foot.

The load per linear foot of girder then equals

Assumed dead load of steel girder	40#
Fireproofing	225#
Floor load, 147#×15′	=2205#
Total	2470#

$$M=\tfrac{1}{8}\times2470\times(14)^2\times12$$
$$=726{,}000''\#$$

$$S=\frac{M}{f_s}=\frac{726{,}000}{18{,}000}=40.4$$

Use a 12″ I beam 40.8# for which $S=44.8$.

Girders (2-3) (3-4) (4-5) (5-6) (6-7) Line D. The floor adjacent to these beams carries a live load of 50# per square foot. The dead load of the floor construction equals 62# per square foot. The floor width that these beams support is

$$\tfrac{1}{2}\times(10'\text{-}6''+15'\text{-}4'')=12'\text{-}11''$$

say 13 feet. The span of these girders is 17′-4″.

The load per linear foot of girder equals

Assumed dead load of steel girder	50#
Floor load, 104#×13′	=1350#
Fireproofing	225#
Total	1625#

$$M=\tfrac{1}{8}\times1625\times(17.34)^2\times12$$
$$=734{,}000''\#$$
$$S=\frac{M}{f_s}=\frac{734{,}000}{18{,}000}=40.7$$

Use a 12″ I 40.8#.

Problem. Design girder (7-8) Line *D*. The floor construction and live load are the same as for girder (6-7) Line *D*.

Girders (2-3) (3-4) Line B. These girders support a portion of the floor in the kitchen. Tile flooring is used which weighs, including the necessary fill, 40# per square foot. To better carry the partitions and framing necessary, a flat concrete slab $6\frac{1}{2}$ inches thick is used. This slab is reinforced in two directions so that the load is carried on four girders. The location of the partitions is not certain; therefore, a load of 25# per square foot is added.

The load per square foot as applied to the girders is as follows:

Dead load of concrete slab	81#
Tile floor and fill	40#
Partitions	25#
Live load, 85% of 100#	=85#
Total	231#

This load will be distributed to the girders according to the span of the slab in both directions. Size of panel is 15′-4″ by 17′-4″.

Load carried in short direction equals

$$W_1=\left(\frac{l}{l_1}-\frac{1}{2}\right)W$$

$$W_1 = \left(\frac{17.34}{15.34} - \frac{1}{2}\right)231 = .63 \times 231 = 146\#$$

The load per linear foot of girder will then be:

Assumed weight of steel girder	50#
Fireproofing	300#
Floor load, 146#×15′	=2190#
Total	2540#

The spans of these two girders are slightly different, due to one column being turned opposite to the other two. A span length of 16′-6″ will be used.

$$M = \frac{1}{8} \times 2540 \times (16.5)^2 \times 12$$
$$= 1{,}040{,}000''\#$$
$$S = \frac{M}{f_s} = \frac{1{,}040{,}000}{18{,}000} = 57.8$$

Use 15″ I 42.9#.

Girders (*B-C*) *Lines 3 and 4*. As the slab carrying the kitchen floor spans in two directions, these beams carry the following load per square foot.

$$231 - 146 = 85\# \text{ per square foot.}$$

The load per linear foot of girder equals:

Assumed weight of steel beam	35#
Fireproofing	225#
Floor load, 85#×17.34′	=1475#
Total	1735#

The span lengths of these two beams are slightly different; therefore the longest, 15′-4″, will be used.

$$M = \frac{1}{8} \times 1735 \times (15.34)^2 \times 12$$
$$= 612{,}000''\#$$
$$S = \frac{M}{f_s} = \frac{612{,}000}{18{,}000} = 34$$

Use 12″ I 31.8#.

Strut or Joist (*D-E*) *Line 3*. This is a typical strut. The purpose of these members is to carry the load directly above and also to brace the building transversely. Struts also permit the building to be kept plumb during erection.

Four-inch tile partitions are directly over some of these struts, therefore they should be strong enough to carry this load. Four-inch tile plastered on both sides weighs 25# per square foot. These members carry the load from a strip of floor about 2′-6″ wide.

The load per linear foot on a strut would be about as follows for the maximum condition:

Weight of steel strut	18#
Live load, $2\frac{1}{2}' \times 85\#$	=212#
Partition, $11' \times 25\#$	=275#
Fireproofing	175#
Total	680#

$$M = \tfrac{1}{8} \times 680 \times (15.34)^2 \times 12$$
$$= 240{,}000''\#$$

$$S = \frac{240{,}000}{18{,}000} = 13.3$$

It is felt that these struts should be a minimum of 8 inches, therefore the lightest section is used, which is 8″ **I** 18.4#. This section has a section modulus of 14.2 and will be strong enough for the maximum condition.

Beam (*E-F*) *Line 1.* This beam carries the strip of floor directly above it, the width of which is about 2′-6″. There is an 8-inch tile partition, plastered on both sides, located directly above this beam. The floor load on one side of this beam is 100# per square foot and on the other side, 50#. The span of the beam is 14′-8″ feet.

The load per linear foot of beam equals:

Weight of steel beam	25#
Fireproofing and adjacent slab	250#
Live load, $2.5'\left(\frac{85\# + 42.5\#}{2}\right)$	=159#
Partition, $11' \times 38\#$	=418#
Total	852#

$$M = \tfrac{1}{8} \times 852 \times (14.67)^2 \times 12$$
$$= 275{,}000''\#$$

$$S = \frac{275{,}000}{18{,}000} = 15.3$$

Use 10″ **I** 25.4#.

An 8″ **I** 23# could have been used, and would have been selected if more head room under this beam had been necessary; however, this was not the case. It was felt that with the slight difference in weight the 10-inch beam was preferable.

First-Floor Spandrel Girders. In designing these girders, refer to the architectural and structural floor plans and to the architectural elevations.

Spandrel Girders (2-3) (3-4) (4-5) (5-6) Line F. The concrete encasement is made 2′-8″ deep so that these beams serve as lintels over the basement windows. Each beam supports one-half panel of floor, also the outside wall. The amount of wall load is calculated as follows:

Gross wall area, 10′-0″×14′-4″	=143 sq. ft.
Less window area, $2\times4'\text{-}4\frac{1}{2}''\times6'\text{-}7''$	= 57 sq. ft.
Net wall area	86 sq. ft.

The weight per square foot of the material composing the wall is:

4 inches pressed brick weighing 140# per cu. ft.	47#
9 inches common brick weighing 120# per cu. ft.	90#
Total weight per sq. ft. of wall	137#

The weight of the wall on the spandrel girders is

$$137\times86=11{,}800\#$$

Load per linear foot of girder is

$$11{,}800\div14.34=822\#$$

The weight of the windows is small and is usually not taken into account. More exact computations would take into account the position of the windows but this would not change the results a great deal and usually is not done.

The total load per linear foot of girder will then be

Assumed weight of steel beam	50#
Concrete lintel and fireproofing	600#
Wall load	822#
Floor load, 104#×7.5′	= 780#
Total	2252#

The depth and size of the concrete encasement is such that by placing reinforcing bars in the bottom it can be utilized as a reinforced concrete girder. A nominal size steel beam will then be used. A 10″ I 25.4# is selected.

Spandrel Girder (9-10) Line F. This girder is under the main entrance and supports half a panel of corridor and half of the outside landing slab. The revolving door and wall framing are assumed to weigh 500 pounds per linear foot of girder.

The total load per foot is:

Weight of steel girder	50#
Fireproofing	300#
Sill of door	75#
Dead load of outside landing	150#
Live load from landing, 2′×85#	170#
Floor load, 7.3′×147#	1070#
Wall load	500#
Total	2315#

The span of this girder is 16′-0″.

$$M=\tfrac{1}{8}\times 2315\times(16)^2\times 12$$
$$=890{,}000''\#$$
$$S=\frac{890{,}000}{18{,}000}=49.5$$

Use 15″ I 42.9#.

In determining the sizes of beams required, the bending moment and section modulus has been computed. This procedure is not necessary when the beams carry uniform loads as the total load on the beam can be figured and then the required beam picked from a steel handbook.

Second-Floor Girders and Beams. *Girders (3-4) (4-5) (5-6) (6-7) Lines C and D.* These girders carry one-half panel of classroom floor and one-half panel of the corridor floor. There is a 4-inch tile partition, plastered on both sides, directly above these girders.

The load per linear foot on these girders is:

Weight of steel girder		50#
Fireproofing		300#
Floor load, 104#×7.67′	=	798#
Floor load, 147#×5.25′	=	772#
Partition, 25#×11′	=	275#
Total		2195#

Total uniform load equals

$$2195\#\times 17.34'=38{,}000\#$$

Use 15″ I 42.9#.

Girders (A-C) (D-F) Lines 5 and 6. The length of these girders is 29 feet, spanning two panels. The columns *B6*, *E6*, and *E5* have been omitted above the first floor. These girders carry the reactions, at the center, of both the main floor girders on each side. These floor girders, on lines *B* and *E*, carry the same load as the like girders on the first floor which have already been designed.

The reaction from the girders on lines *B* and *E* is, per girder,

$$1910\# \times \frac{17.34}{2} = 16{,}600\#$$

Then the concentrated load at the center of girders (*A-C*) and (*D-F*) is

$$2 \times 16{,}600\# = 33{,}200\#$$

The uniform load on the girders is

Weight of steel girder	100#
Fireproofing	550#
Total	650#

Bending moment at the center line of the girders equals

$$\tfrac{1}{2} \times 33{,}200 \times 14.5 = 241{,}000'\#$$
$$\tfrac{1}{8} \times 650 \times (29)^2 = 68{,}400'\#$$
$$309{,}400'\#$$

Required section modulus equals

$$S = \frac{309{,}400 \times 12}{18{,}000} = 206$$

Use a 21″ WF 103#.

It is desirable to check the deflection of a long girder like this one.

The deflection formulas are:

for uniformly distributed loads

$$d = \frac{5}{384}\,\frac{Wl^3}{EI}$$

for load concentrated at center

$$d = \frac{1}{48}\,\frac{Wl^3}{EI}$$

$$d = \frac{5 \times 18{,}870 \times 348 \times 348 \times 348}{384 \times 29{,}000{,}000 \times 2268} = 0.157''$$

$$d = \frac{1 \times 33{,}200 \times 348 \times 348 \times 348}{48 \times 29{,}000{,}000 \times 2268} = 0.444''$$

Total deflection at center 0.601″

Deflections are usually limited to $\frac{1}{360}$ of the span, as this amount will not cause cracking of the plaster.

$$\tfrac{1}{360}\times 29\times 12=0.97''$$

The deflection of these girders is satisfactory.

Stair Well Beams. There are three stair wells on the second floor that require special framing. The beams required for this framing are usually never very large. The live load on the stairs is 100 pounds per square foot. The dead load is taken as 50 pounds per square foot. The framing around stair wells should be designed so that the load from the stairs can be carried either by the beams at the sides or those at the ends. In some stair designs the load is carried by the stringers to the beams at the ends of the well, and in other designs hangers or struts are used to transmit the loads to the side beams. Unless the structural engineer designs the stairs or knows what make will be used, he does not know how the loads will be transmitted to the beams.

Second-Floor Spandrel Girders. *Spandrel Girders (2-3) (3-4) (4-5) (5-6) (6-7) Lines A and F.* These girders support the wall and one-half panel of floor.

Gross wall area, 10′-0″×14′-4″	=143 sq. ft.
Less windows, 2×4′-4½″×6′-1″	= 53 sq. ft.
Net wall area	90 sq. ft.

Weight per square foot of wall equals 137#.

The total load of the wall on the spandrel girder is

$$90\times 137=12{,}330\#$$

Total load per linear foot of spandrel girder exclusive of wall load will then be:

Weight of steel girder	50#
Fireproofing	300#
Floor load, 104×7.3	= 760#
Total	1110#

The total uniform load on the girders is

$$17.34'\times 1110\#+12{,}330\#=31{,}580\#$$

Use 15″ **I** 42.9#.

Problems. 1. Compute the deflection of the spandrel girder (*2-3*) *Line F.*

2. Design the spandrel beams (*B-C*) (*D-E*) *Line 8.*

3. Compute the deflection of the beam in Problem 2. Is this deflection less than $\frac{1}{360}$ of the span?

Third-Floor Girders and Beams. A large portion of this floor is occupied by the auditorium, on which the live load is 100 pounds per square foot. The floor construction consists of concrete joists and slab with maple floor covering.

The load per square foot is

Maple floor	5#
Nailing strips and $1\frac{1}{2}''$ cinder fill	12#
Floor construction	46#
Plastered ceiling	10#
Live load to girders, 85%×100	= 85#
Total	158#

Girders (2-3) (3-4) (4-5) (5-6) Lines B and E. The load per linear foot of girder equals

Weight of steel girder	60#
Fireproofing	300#
Floor load, 158#×15′	=2370#
Total	2730#

$$M=\tfrac{1}{8}\times 2730\times(17.34)^2\times 12$$
$$=123{,}200''\#$$

$$S=\frac{123{,}200}{18{,}000}=68.5$$

Use an 18″ I 54.7#.

Girders (2-3) (3-4) (4-5) (5-6) (6-7) Lines C and D. The total load per linear foot of girder is

Weight of steel girder	55#
Fireproofing	300#
Floor load, 158#×13′	2055#
Total	2410#

$$M=\tfrac{1}{8}\times 2410\times(17.34)^2\times 12$$
$$=109{,}000''\#$$

$$S=\frac{109{,}000}{18{,}000}=60.5$$

Use a 15″ I 45#.

Problems. 1. Design third floor girder *(9-1) Line E.*

2. Design third floor girder *(9-1) Line D.*

3. Design third floor girder *(D-F) Line 5.*

Third-Floor Spandrel Girders (2-3) (3-4) (4-5) (5-6) Lines A and

F. These girders support half a panel of floor and the outside wall.

Gross area of wall, $10' \times 17.34$	$=173$ sq. ft.
Less window area, $6.08 \times 4.375 \times 2$	$=53$ sq. ft.
Net wall area	120 sq. ft.

Total weight of wall on spandrel girder is

$$120 \times 137\# = 16{,}450\#$$

Load per linear foot of girder exclusive of wall load:

Weight of steel girder	55#
Fireproofing	300#
Floor load $158\# \times 7'$	$=1106\#$
Total	1461#

Total load on spandrel girder equals

$$1461 \times 17.34 + 16{,}450\# = 41{,}800\#$$

Use 15″ I 50#.

Framing for Stage. The floor construction under the stage consists of reinforced concrete slabs. The stage floor is also of concrete slab construction with wood surface supported on concrete walls. The space under the stage is used for storing the auditorium seats. The steel framing in these panels must support the dead load of the concrete construction as well as the live load on the stage and in the storage space.

Problem. Design the third-floor spandrel beam (*D-E*) *Line 8*.

Fourth-Floor Interior and Spandrel Girders. As the auditorium occupies the height of two floors, the only fourth floor framing is in back of the balcony. This space is occupied by the foyer, check rooms, toilets, etc. The design of these girders and beams is the same as illustrated for the framing on the other floors.

The fourth-floor spandrel beams are at elevations varying between 35 and 36 feet. The spandrel beams in the auditorium portion do not have any floor load; however, the wall loads are larger than on the spandrels on the other floors. About 12 feet higher than the spandrel beams on the fourth floor, are some that carry the heavy coping and parapet walls.

Balcony Construction. For the balcony construction four main cantilever trusses are connected to columns *B3*, *C3*, *D3*, and *E3*. No framing is allowed between the third and fourth floor girders and between columns on Lines *2* and *3*, otherwise trusses could be provided from columns on Line *3* back to columns on Line *2* to take the necessary uplift.

Girders are provided between the columns on Lines *2* and *3*, framing in at the top and bottom of the cantilever trusses. With this framing, the assumption can be made that both lines of columns resist the bending of the cantilever trusses. The girders framing between the columns also take some of the bending moment; however, to determine this would require a rigid frame analysis, which is beyond the scope of this book.

The balcony floor or banks is constructed of reinforced concrete.

The joists, or vertical portion of the banks, span from truss to truss, supported on seats at each truss. The horizontal, or floor portion of the banks, span from joist to joist.

Lateral bracing is provided in the plane of the lower chord of the trusses to give lateral restraint to the compression chord and to brace the trusses transversely.

Roof Construction. Precast concrete tile is used for the roof construction. On top of this is placed insulating board and composition roofing. The use of I-beam purlins is necessary to provide a flange wide enough to support the roof tile.

Load per square foot equals:

Live load	25#
Suspended ceiling	10#
Dead load of roof slab	10#
Insulation and composition roofing	5#
Total	50#

The spacing of the purlins over the main portion of the building is 4′-4″. The span of the purlins is 17′-4″.

Load per linear foot of purlin equals

50#×4.34	=217#
Weight of purlin	15#
	232#

$$M = \tfrac{1}{8} \times 232 \times (17.34)^2 \times 12$$
$$= 105{,}000''\#$$

The A.R.E.A. Specification (1927) states that the depth of purlins should preferably be not less than $\frac{1}{24}$ of the span, and in no case more than $\frac{1}{30}$ of the span. If 7-inch purlins are used, the depth will be slightly less than $\frac{1}{30}$ of the span; therefore, the lightest 7-inch I beam, which is 7″ I 15.3#, will be investigated.

$$f_s = \frac{105{,}000}{10.4} + 10{,}100\# \text{ per sq. in.}$$

The A.R.E.A. Specifications also require that the compression flange of rolled beams be laterally supported at intervals not exceeding thirty times the flange width. Also, the flange stress shall not exceed $18{,}000 - 200\ \frac{l}{b}$; where l equals the length of the member in inches and b is the flange width in inches.

Flange width is 3.66 inches.

Allowed unsupported length equals

$$30 \times 3.66 \div 12 = 9.14 \text{ feet}$$

It is therefore necessary to brace the purlins at mid-span and this will be done by using sag or tie rods. The allowed stress then will be

$$f_s = 18{,}000 - 200\ \frac{17.34 \times \frac{1}{2} \times 12}{3.66}$$

$$= 12{,}300\# \text{ per sq. in.}$$

Therefore the 7″ I 15.3# is satisfactory.

PRACTICE PROBLEM

Problem. Make the necessary computations to enable selecting an I-beam purlin for a span of 20 feet. The uniform load is 300 pounds per linear foot. Assume that lateral bracing for the flange will be placed at intervals not exceeding 30 times the flange width.

Column Design. The interior columns in the back of the balcony extend to the roof; however, they are not typical columns.

To illustrate the computation of loads and the design of a typical interior column, assume that column $E4$ has the same beam framing around it at all four floors and the roof. We will call this assumed column $E4'$ so as not to confuse it with the actual $E4$ column in the school building.

The floor area tributary to column $E4'$ is

$$17'\text{-}4'' \times \frac{15'4'' + 14'\text{-}7''}{2} = 259 \text{ sq. ft.}$$

For convenience use 260 sq. feet. This area is assumed to apply at all floors and the roof.

The live loads per square foot for the successive floors, after making the reductions described in the section under loads, are:

Roof	25#
Fourth floor, 85%×50#	=$42\frac{1}{2}$#
Third floor, 80%×100#	=80#
Second floor, 75%×50#	=$37\frac{1}{2}$#
First floor, 70%×50#	=35#

The dead loads per square foot, including the weight of the girders and fireproofing, are:

Roof	50#
Fourth floor	90#
Third floor	100#
Second floor	90#
First floor	90#

There has been no load per square foot added to provide for partitions, as their locations are known and provided for. Assume that a 4-inch tile partition, plastered on both sides, is located on the first floor on column Line *4*. The column then supports one half of the partition on each side between adjacent columns. The partition load carried by the column is

$$10' \times 15' \times 25\# = 3750\#$$

Another load carried by the column is the weight of the column and covering. Usually, for this weight, an average amount per foot of length is computed and used for the whole length thus:

Steel	50#
Concrete $(16'' \times 16'' - 15) \times \frac{150}{144}$ = say	250#
Total per foot of column	300#

From the foregoing data the loads on column *E4′* are computed and entered in the Column Load Schedule, Fig. 237. For the column section in any given story, the loads entered are the weight of the col-

	Fourth Floor	Third Floor	Second Floor	First Floor	Basement
Floor live load..........	6,500	11,050	20,800	9,750	9,100
Floor dead load.........	13,000	23,400	26,000	23,400	23,400
Partition load...........	......	......	......	......	3,750
Column and covering....	4,800	3,600	3,600	3,600	3,600
Total for story.........	24,300	38,050	50,400	36,750	39,850
Accumulated total.....	24,300	62,350	112,750	149,500	189,350
Eccentric effect.........	......	......	......	......	......
Total	24,300	62,350	112,750	149,500	189,350
Column section.........	8″ WF 31#			10″ WF 49#	

Fig. 237. Schedule of Loads in Pounds for Column *E4′*. Floor Area, 260 Sq. Ft.

umn in that story, the weight of the floor above, and the weight of the partitions or walls in the story above.

As the loads are entered, the eccentricity if any, is indicated by a small letter "*e*." For column *E4'* there is a small eccentricity due to the small difference in the panel lengths on each side; namely, 15'-4" and 14'-7". As the effect of this eccentricity is small, it is not taken into account.

With the loads computed as given in the schedule, the next step is to determine the required sections. The columns for this school building are designed by the A.R.E.A. (1927) Specification formula; therefore the sections cannot be picked from the tables in the steel handbooks as they could be if A.I.S.C. Specifications were used.

In multistory buildings the columns are usually made in two-story length. The cross section of any length is governed by the stress in the lower of the two stories comprising that length. Fig. 238 shows the erection of columns in two-story lengths in a typical multistory building.

The load on the basement section of column *E4'* is 189,350#. The unsupported length is usually taken as the distance from floor to floor, unless there are deep girders framing into the columns.

$$l = 11'\text{-}8'' = 140''$$

Try a 10" WF 49#.

$$\text{Area} = 14.40 \text{ sq. in.}$$

$$\text{Least radius of gyration, } r = 2.54$$

$$\text{Allowable unit stress} = 18{,}000 - 60\ \frac{l}{r}$$

$$= 18{,}000 - 60\ \frac{140}{2.54}$$

$$= 14{,}690 \text{ lb. per sq. in.}$$

Allowed load on column section

$$14.40 \times 14{,}690 = 211{,}500 \text{ lb.}$$

Therefore this section, 10" WF 49#, is satisfactory.

The load on the second-floor column is 112,750#.

Try an 8" WF 31#.

$$l = 12'\text{-}0'' = 144''$$

$$r = 2.01$$

$$A = 9.12 \text{ sq. in.}$$

$$\text{Allowed unit stress} = 18{,}000 - 60\,\frac{144}{2.01}$$
$$= 13{,}700$$

Allowed column load equals

$$9.12 \times 13{,}700 = 125{,}000\#$$

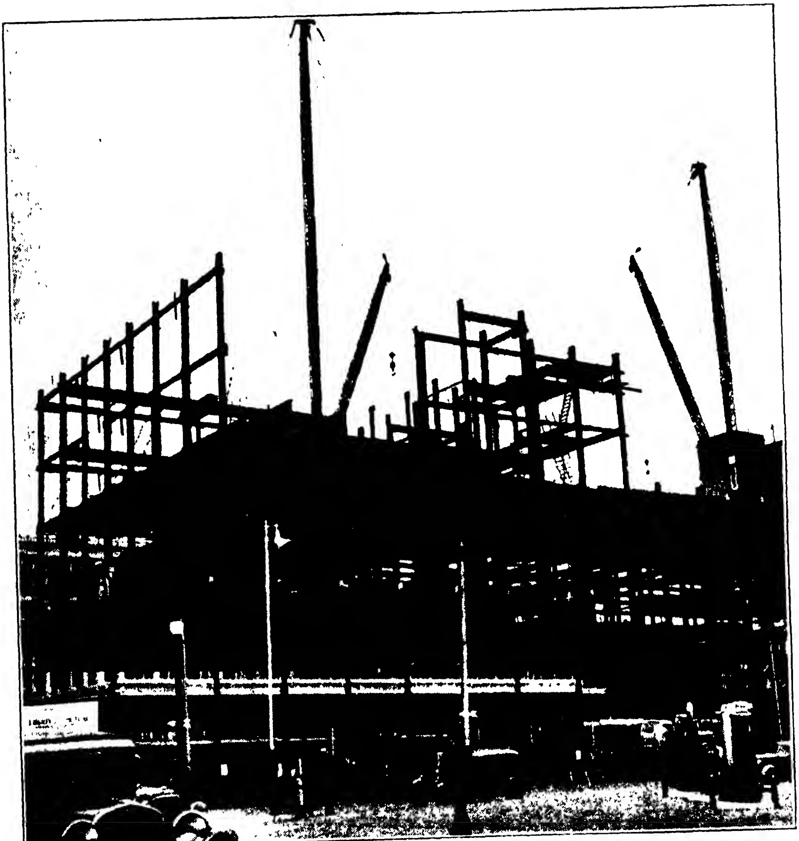


Fig. 238. Steel Erection of a Typical Multistory Building
Courtesy of The American Bridge Company

The section selected is therefore satisfactory. If this column were made two stories long, another section, one story long, would be required. A three-story column is more economical in this case, as the saving in steel by the use of a two-story column would be small and would not equal the cost of a splice.

Design of Column E10. The loads on this column consist of the weight of the column and covering, floor loads, and wall loads. The spandrel girders support one half of the panel of floor, and the outside wall. The spandrel girders are connected to the web of the column 3 inches off center, causing an eccentricity.

The beams framing into the inside flange of the column serve only as struts, except on the roof where the beam supports the roof purlins.

The floor area tributary to this column is

$$\frac{14'\text{-}7''}{2}\times\frac{16'-0''+14'\text{-}0''}{2}=109 \text{ sq. ft.}$$

The live loads per square foot for the successive floors are:

Roof	25#
Third floor,	85%×100=85#
Second floor,	80%×100=80#
First floor,	75%×100=75#

The dead loads per square foot, including the weight of the girders, struts, and their fireproofing, are:

Roof	75#
Third floor	90#
Second floor	90#
First floor	90#

Column *E10* supports one half of the wall between columns *E9* and *E10* and one half between wall of old building and column *E10*. The walls are 13 inches thick and weigh 137# per square foot.

The net area of the wall between columns *E10* and *E9* is

$$13\times10-2(2.83\times6.08)-3\times6.08=77 \text{ sq. ft.}$$

The net area of the wall between column *E10* and the old building is

$$12.5\times10-2(3.12\times6.08)=87 \text{ sq. ft.}$$

The width of the concrete encasement of the column is 3 feet and is included in the weight of the column. The height of the windows on the first floor is slightly greater than on the second and third floors where they are 6'-1"; however, the 6'-1" height is used for all the floors in figuring the net wall area.

From the above data the loads on column *E10* are computed and entered in the schedule, Fig. 239. Where eccentricity exists it is indicated by the small letters "*e*."

On the roof the spandrel beams connect to the column 3 inches off center. The beam supporting the roof purlins connects to the inside flange of the column, which is 6 inches off center if a 12″ WF section is used. The eccentricities of these beams tend to balance each other; however, due to the larger load and eccentricity of beam *EF* it causes a net eccentric effect.

	Third Floor	Second Floor	First Floor	Basement
Floor live load	2,720	9,260	8,720	8,180
Floor dead load	8,180	9,810	9,810	9,810
Wall load	.....	e 24,470	e 22,470	e 22,470
Column and covering	6,600	6,600	6,600	6,600
Total for story	17,500	50,140	47,600	47,060
Accumulated total	17,500	67,640	115,240	162,300
Eccentric effect	5,450	12,500	12,600	12,500
Total	22,950	80,140	127,840	174,800
Column section	12′ WF 25#		12′ WF 40#	

Fig. 239. Schedule of Loads in Pounds for Column E10′. Floor Area, 109 Sq Ft

In the schedule, Fig. 239, on the line marked "Eccentric effect," are given the concentric equivalents of the eccentric loads computed from the following formula:

$$W'_e = W'\frac{ec}{r^2}$$

The equivalent concentric load (W_e') due to the eccentric beams at the roof is

$$\begin{aligned} 7530\# \times 6'' &= 45{,}180''\# \\ 5000\# \times 3'' &= \underline{15{,}000''\#} \\ &\ 30{,}180''\# \end{aligned}$$

$$W'_e = 30{,}180\ \frac{c}{r^2}$$

$$= 30{,}180\ \frac{6}{(4.98)^2}$$

$$= 7280\#$$

Three-fourths of this amount is used and entered in the schedule.

On the first, second and third floors, the spandrel girders are off center 3 inches. The struts framing into the flange of the column carry very little load, therefore the eccentricity from these will be disregarded.

The concentric load at the first, second, and third floors will be

$$W'_e = 24{,}470 \times 3 \times \frac{6}{(5.13)^2}$$
$$= 16{,}700$$

Three-fourths of this amount is entered in the schedule.

The eccentric effect is not cumulative and the equivalent con-

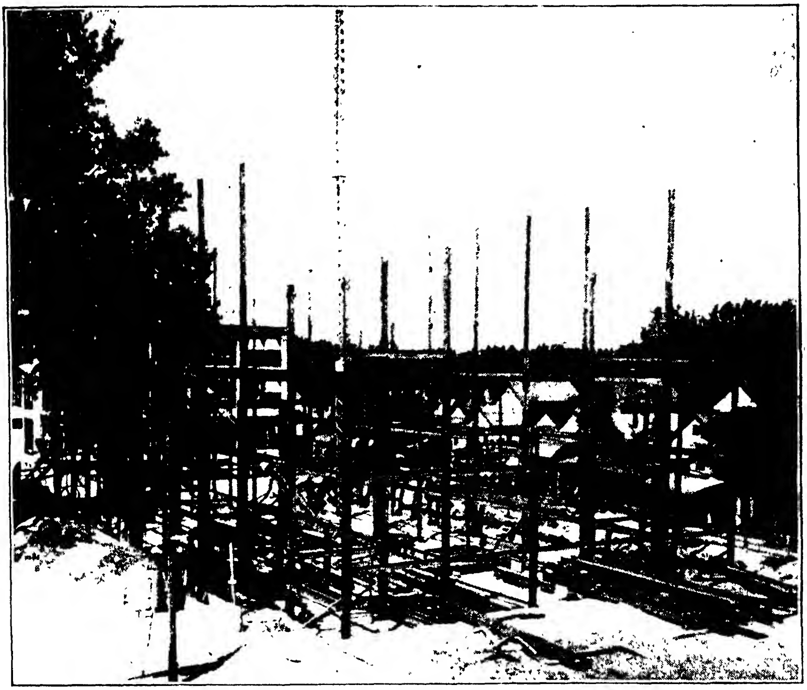


Fig. 240. Steel Frame School Building under Construction
Courtesy of A. T. & S. F. Railway System

centric load is not carried down into the next lower column section, but disappears at the bottom of the section under consideration.

In computing the allowed unit stress, the larger value of radius of gyration can be used, as the column is supported in the weak direction by the 3′-0″ wide concrete encasement.

A 12″ WF 40# section will be investigated for use for the basement and first floor.

Allowed unit stress

$$= 18{,}000 - \left(60 \times \frac{140}{5.13}\right)$$

$$=16{,}360\# \text{ per sq. in.}$$

This exceeds the maximum allowed, therefore 15,000# per sq. in. will be used.

Allowed load equals

$$11.77 \times 15{,}000 = 176{,}500\#$$

The section selected is satisfactory. If the design were according to A.I.S.C. Specifications, it would be necessary, of course, to compute the columns according to their requirements for eccentric loading.

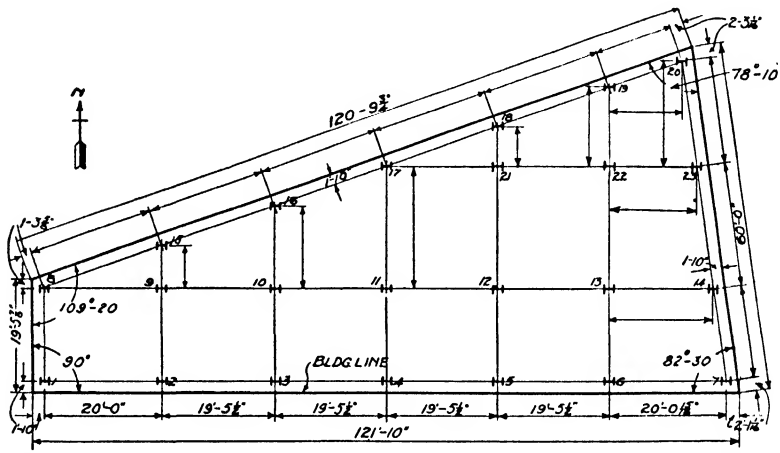


Fig. 241. Diagram Showing Method of Dimensioning Column Centers in an Irregular Building

The lightest 12″ WF section, which weighs 25# per foot, will be investigated for the second- and third-floor section. Allowed load equals

$$7.39 \times 15{,}000 = 111{,}000\#$$

Therefore this section is satisfactory.

Wind Bracing. No special wind bracing is necessary for a building of these proportions. Building codes vary in regard to the limiting height-to-width relation before wind-bracing analysis is required. The usual provision is that where the height of the building is less than two times the least horizontal dimension, it is exempt from wind-pressure design.

Fig. 240 shows this school building under construction.

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. 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. Any irregularity of spacing in any row requires a special line of dimensions in that row.

For 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 coordinates. The only diagonal dimensions needed are those along which, or parallel to which, steel members are placed.

In Fig. 241 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. 242 it is apparent that these distances *a b* and *a′b* are equal to each other and equal to $ca \times \cot 41° 15'$; 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 which are lacking between columns in Fig. 241.

The column center dimensions should be repeated on all the floor plans. If the floor framing plan is crowded, a separate diagram

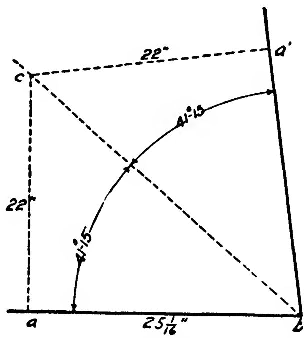


Fig. 242. Construction Diagram for Details of Figure 241

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. 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.

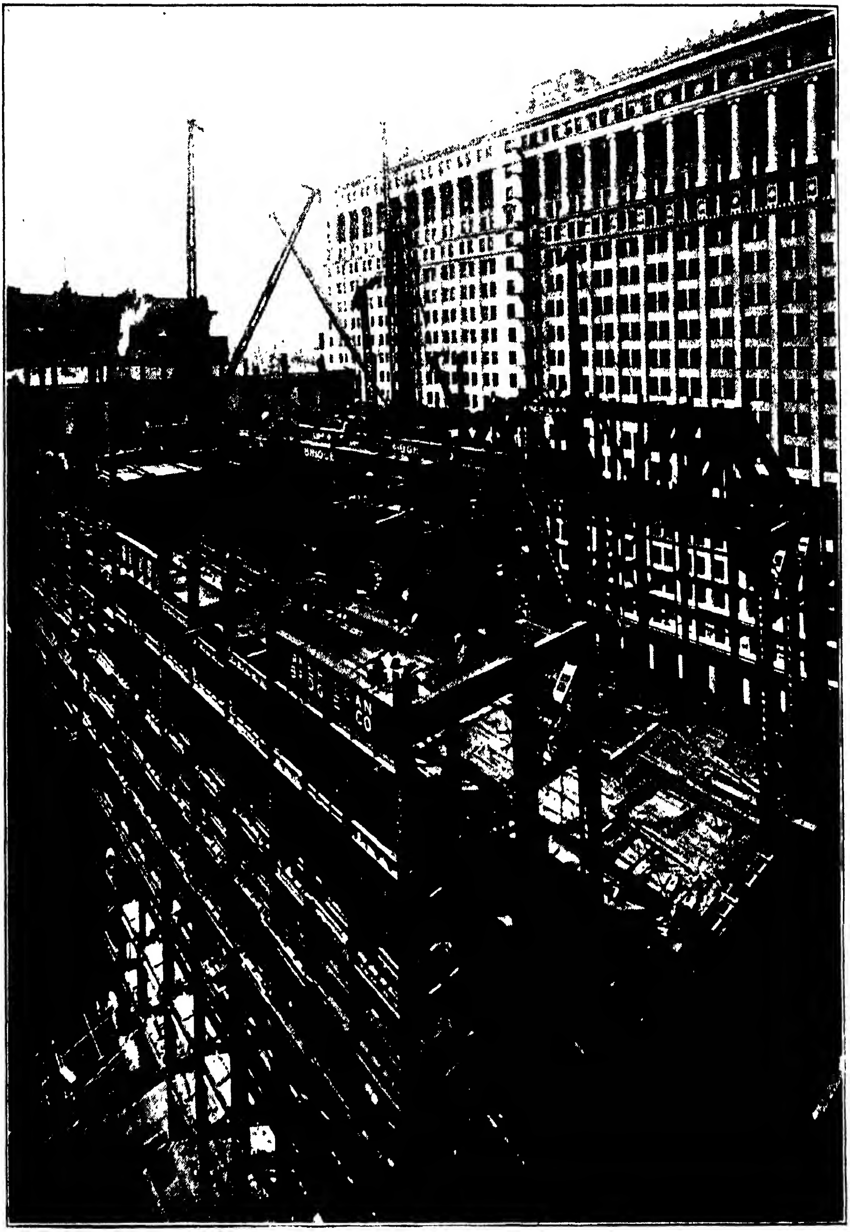
Vertical Dimensions. The vertical dimensions from floor to floor are given in a separate diagram or in connection with the column schedule. At the first floor a reference is made to established sidewalk grade in terms of its elevation above datum. The elevations of beams are given in reference to the finished floor elevations, respectively. Usually the elevation of joists and girders can be covered by a note. 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.

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.



CHICAGO BOARD OF TRADE, CHICAGO
Note Heavy Truss Framing over the Trading Room
Courtesy of Holabird and Root, Architects

CHAPTER XII

PROTECTION OF STRUCTURAL STEEL

The structural steel framework of buildings must be protected against corrosion and fire. In multistory buildings the steel is encased and so is protected against moisture, therefore the protection against fire is the more important of these two problems. In industrial buildings steel is usually exposed and special provisions to prevent rusting are necessary.

PROTECTION FROM CORROSION

Theories of Corrosion. The corrosion of steel is a chemical change in which the metallic iron unites with oxygen and forms oxide of iron, or rust. There are several theories of corrosion, such as: oxidation theory, carbonic acid theory, electrochemical theory, and electrolytic theory. These different theories disagree in regard to the causes of corrosion; however, all accept as a necessary condition the contact of air and water with the steel. While rust is largely or wholly oxide of iron, it is not produced by the contact of iron with oxygen of the air alone; the presence of moisture is essential to its formation.

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 (1) very slowly if the steel is kept in dry air; (2) less slowly if subjected occasionally to moist air; (3) rapidly if exposed to moisture frequently; and (4) 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. The third degree of exposure applies for 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 (*4*) is had in smelters, and in structures where the steel is subjected to the smoke from railroad locomotives.

Rate of Rusting. The American Society of Testing Materials has been making atmospheric exposure tests of iron and steel specimens since 1916 to determine their corrosion resistance. These tests were carried on in different localities, and results have shown that rusting occurred most rapidly in an industrial atmosphere (Pittsburgh, Pa.) where smoke and moisture combine to give a dilute solution of sulphuric acid. Rusting was slower on specimens exposed in a rural or light industrial atmosphere (Ft. Sheridan, Ill.) and slowest on those exposed near the seacoast (Annapolis, Md.).

PROTECTION EFFECTED BY COMPOSITION

The composition of the metal affects the rate of corrosion. This is clearly shown by the results of the A.S.T.M. tests on metal sheets. The 22-gage sheets located at Pittsburgh had the following rating, from the most resistive to the least.

(1) Copper-bearing Bessemer steel
(2) Copper-bearing acid open-hearth steel
(3) Copper-bearing basic open-hearth steel
(4) Copper-bearing pure iron
(5) Copper-bearing wrought iron
(6) Non-copper-bearing wrought iron
(7) Non-copper-bearing pure iron
(8) Non-copper-bearing basic open-hearth steel
(9) Non-copper-bearing Bessemer steel

Committee A5 of the A.S.T.M. has the following to say about the Ft. Sheridan tests.

> At the close of eleven years' observation of these tests, the sub-committee would direct attention to the fact that the failures at the Fort Sheridan location confirm the findings at Pittsburgh location that copper-bearing metal shows marked superiority in rust-resisting properties as compared to non-copper-bearing metal of substantially the same general composition under atmospheric exposure.

Copper-bearing structural steel is commonly used for industrial buildings where the corrosion conditions warrant. The copper content is specified as not less than 0.2 per cent. This steel costs approximately two dollars a ton more than non-copper-bearing.

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 can 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, white lead, blue lead, iron oxide, zinc oxide, graphite, lampblack, basic lead chromate, etc.

Red lead is the red oxide of lead, Pb_3O_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 workable 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.

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. When an inhibitor is used it is necessary to remove only the loose scales and rust.

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 done well 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.

PROTECTIVE COATINGS OTHER THAN LINSEED OIL PAINTS

In recent years there have been developed protective coatings other than the usual linseed oil paints. One of these is a petrolatum coating containing rust inhibitors. These paints are non-drying and should therefore be used where people do not come in contact with the surface. These non-drying petrolatum paints have proven very satisfactory. They can be had in any desired color, including aluminum.

Another method of protecting steel against corrosion is an application of inhibitor for the pre-treatment of the surface, followed by a primer and finish coat of synthetic resin paint.

The first step in this treatment is the cleaning of the steel by wire brushing. The cleaning does not have to be as thorough as for lead and linseed oil paints and this results in a considerable saving. Only enough wire brushing to remove the loose scale and rust is necessary.

On the wire-brushed surface is applied the inhibitor which usually consists of solvents, dispersing agents, phosphoric acid, and a chromium compound. The inhibitor dissolves and disperses grease or oil and it has the ability to penetrate into minute cracks and crevices. It also absorbs and dissolves the small amount of rust always present on the surface, and forms an iron chromium phosphate. After the inhibitor has dried for a couple of hours, the primer coat is applied.

The paint recommended for the primer and finish coat in this method is one having a vehicle consisting of a modified phthalic anhydride glycerol resin fortified with a phenol formaldehyde tung oil varnish. The pigment should be zinc chromate and iron oxide.

Not enough time has elapsed to secure long-time service tests;

however, laboratory and field tests have shown this type of protective coating standing up very well. Several of the best synthetic paints have made better showings than the linseed oil paints.

Surfaces in Contact. Surfaces in contact should be thoroughly cleaned but should not be painted, as the heat from the rivets destroys the paint and reduces the friction between the surfaces. Parts that are inaccessible after assembly should be given two coats of paint in the shop.

CONCRETE ENCASEMENT

Concrete encasement, either of cinder or stone aggregates, provides adequate protection for steel against moisture. It is not necessary to paint steel that is to be encased in concrete.

DESIGN AND DETAILS

Structural steel designers and detailers should avoid construction with inaccessible parts; thus all surfaces can readily be cleaned and painted. Steel members should not be placed directly on concrete floors, but should be placed on raised curbs or pedestals, away from dirt and moisture. Where steel members are incased at the bottom with concrete, the encasement should slope away from the steel surfaces so that dirt and moisture cannot collect against the steel.

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 inch of length is $100 \times 0.0000067 = 0.00067$; for a length of 18 feet, the total increase in length is $0.00067 \times 18 \times 12$, or .14472 inches.

From this it is clear that expansion and contraction due to changes in temperature occur in appreciable amounts. 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.

Loss of Strength. The strength of structural steel varies as its temperature increases above normal. The following table shows the variation in strength with changes in temperature. This table was prepared by the A.I.S.C. and is based on a careful study of all the available literature that has appeared on this subject since about 1890, and is in substantial agreement with the research findings of Dr. Albert Sauveur, Professor of Metallurgy and Metallography of Harvard University.

The tangent elastic limit is the stress per square inch at which the stress-strain curve ceases to be a straight line. It will be noted that, at high temperatures, the spread between the tangent elastic limit and the yield point is greater than at normal temperatures. Steel melts at approximately 2500 degrees Fahrenheit.

Temperature Degrees Fahr.	Ultimate Strength Pounds per Sq. In.	Yield Point Pounds per Sq. In.	Tangent Elastic Limit Pounds per Sq. In.	Modulus of Elasticity Pounds per Sq. In.	Per Cent Elongation	Per Cent Reduction of Area
100	54000	39000	36000	30 700 000	48	68
200	56000	43000	33000	29 000 000	43	66
300	58000	46000	28000	27 300 000	39	65
400	60000	47000	22000	25 600 000	37	64
500	63000	46000	16000	23 900 000	37	64
600	66000	44000	14000	22 200 000	40	66
700	60000	40000	12000	20 500 000	44	68
800	52000	37000	10000	18 800 000	49	76
900	43000	32000	8000	17 100 000	55	84
1000	34000	27000	7500	15 400 000	60	90
1100	25000	21000	5000	13 700 000	64	94
1200	19000	17000	3500	12 000 000	68	96
1300	14000	13000	2000	10 300 000	72	98

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 AND MATERIALS

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.

The following are satisfactory and approved materials for fireproofing structural steel members.

Stone concrete
Cinder concrete
Gunite
Brick of clay, shale, or concrete
Clay tile
Clay tile and concrete
Concrete blocks
Solid or hollow gypsum blocks
Poured gypsum
Metal lath and gypsum or Portland Cement Plaster

City building codes vary regarding the required thickness of the various fireproofing coverings; therefore, local requirements should be followed.

The selection of the type of fireproofing depends somewhat on the type of floor construction used. If the floor is to be tile arch construction, this material will also be used for fireproofing; however, columns may be fireproofed with concrete if desired. When cinder concrete floor slabs are constructed, this material can be used for encasing the steel; or tile or gypsum blocks can be employed.

In Figs. 243 and 244 are shown methods of fireproofing steel beams and columns with tile and gypsum blocks. In Fig. 243 at (*a*) is shown the method of protecting a structural steel girder supporting open web trussed joists. The joists have a concrete slab on top, and metal lath and plaster on the underside. Fig. 243 at (*b*) shows a method of using tile fireproofing with long span concrete joist floor construction. The tile is put in place before the concrete is poured so that the concrete of the joists fills the cells of the tile and bears directly on the flange of the beam. Fig. 243 at (*c*) illustrates the covering of a spandrel girder which also serves as a lintel over a window. Fig. 243 at (*d*) shows a column encased in hollow tile of a minimum thickness of 3 inches. In Fig. 243 at (*e*) is illustrated the covering of double beams. Fig. 243 at (*f*) shows a method of entirely enclosing a strut that is not covered or protected by the floor or other masonry.

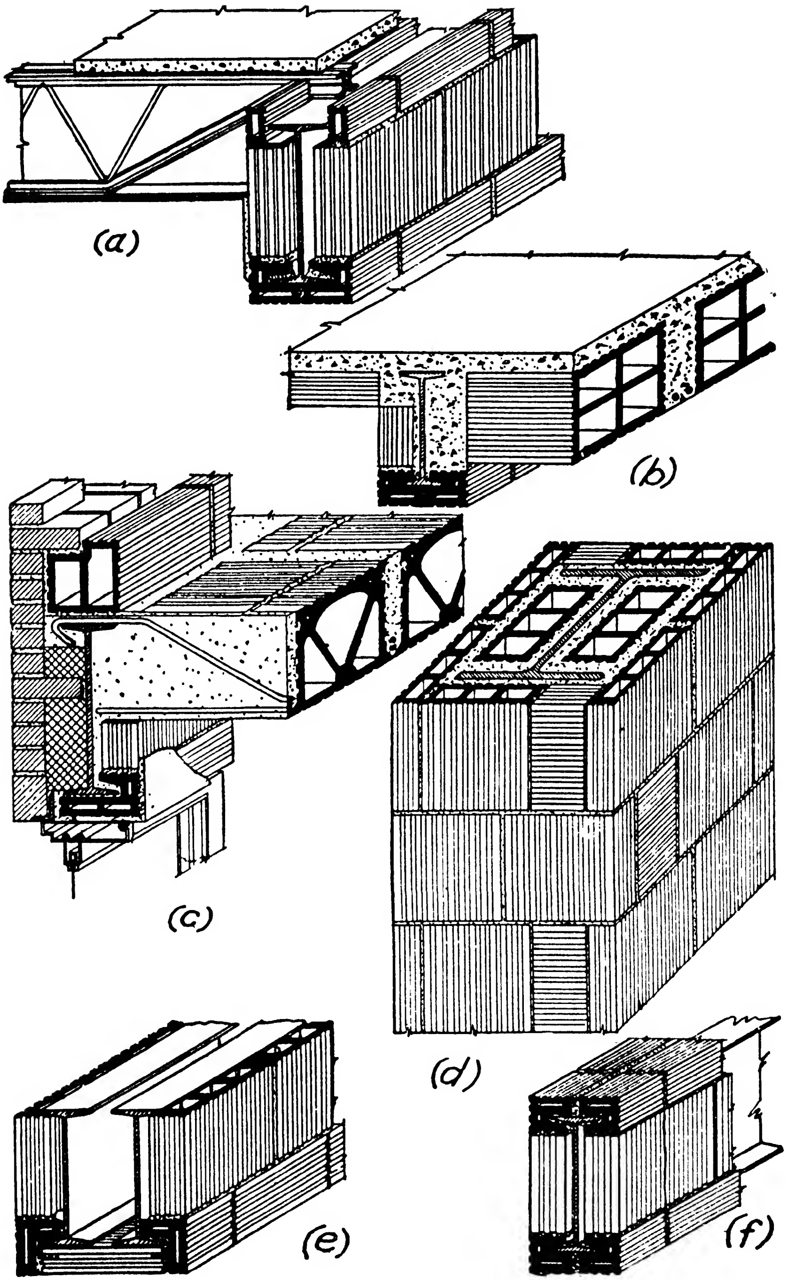


Fig. 243. Fireproofing of Girders and Columns with Hollow Tile
Courtesy of National Fireproofing Corporation

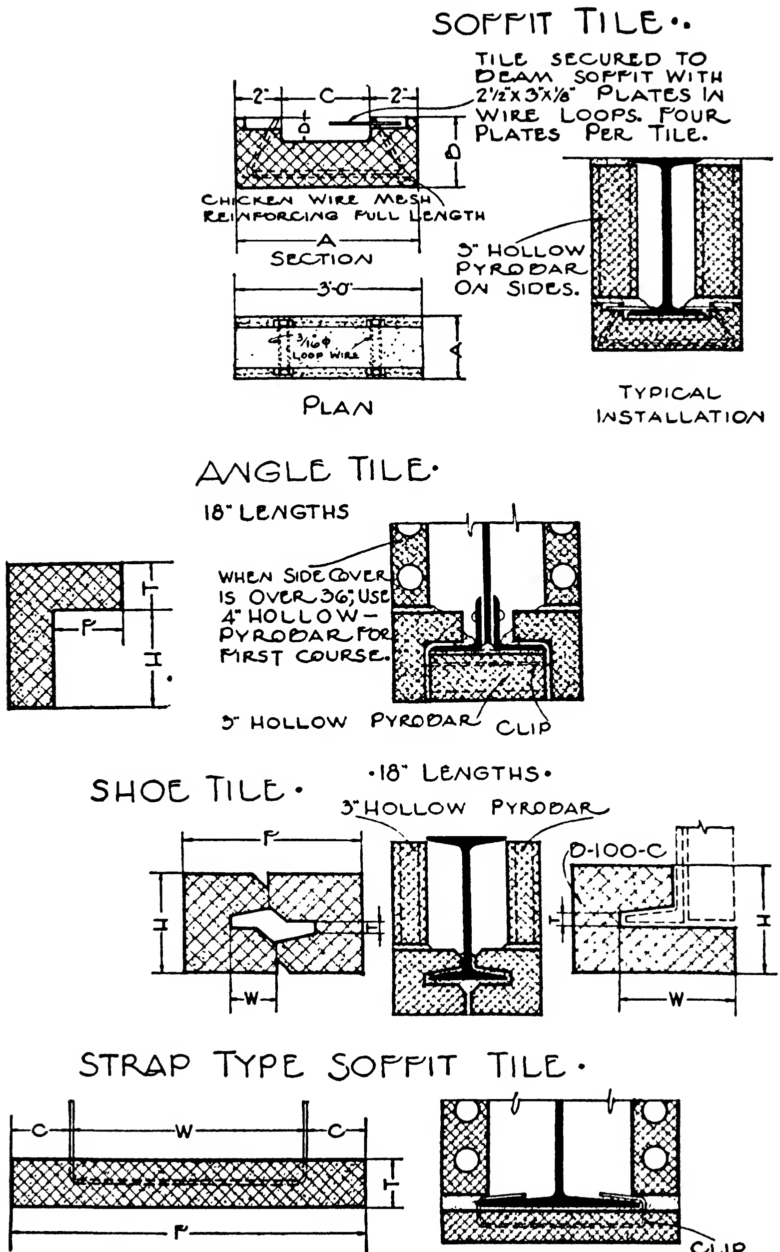


Fig. 244. Fireproofing of Girders with Gypsum Tile
Courtesy of United States Gypsum Company

In Fig. 244 are shown several methods of fireproofing beams and girders with gypsum tile.

Concrete which has been damaged by fire does not lose its property of non-conductivity, consequently it is efficient as a fireproofing so long as it remains in place. In Fig. 245 are shown some methods of providing a mechanical bond for the concrete so that it will not fall off during a fire.

Fireproofing Structural Steel Buildings. Fire hazards are classified in accordance with the nature and combustibility of the materials

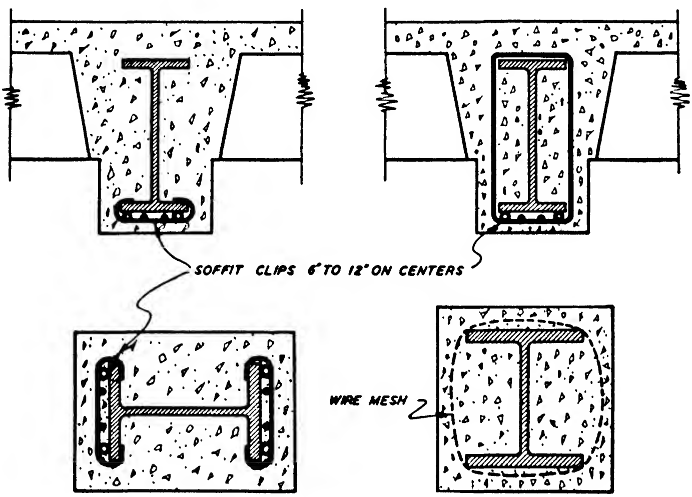


Fig. 245. Reinforcement for Concrete Fireproofing of Beams and Columns

which the building contains. It is not necessary to make buildings containing only ten pounds of combustible material per square foot as fireproof as one having 50 or 60 pounds of burnable contents per square foot.

The A.I.S.C. Specification for Fireproofing Structural Steel Buildings states among other things as follows:

FIRES

(*a*) The intensity and duration of fires is variable, but for the purpose of this specification all fires shall be classified, with regard to their intensity and duration, on the basis of the average time-temperature definitions as set forth in the tentative specification for fire tests on building construction and materials as prepared by the sectional committee on Fire Test Specifications under the joint sponsorship of the United States Bureau of Standards, the American Engineering Standards Committee Fire Test Group, and the American Society for Testing Materials, in accordance with the procedure of the American Engineering Standards Committee, and published by the American Society for Testing Materials under serial designation C 19-26T.

(*b*) The time-temperature definition of the above referred-to specification is given by the accompanying table and shown by the curve in Fig. 1.

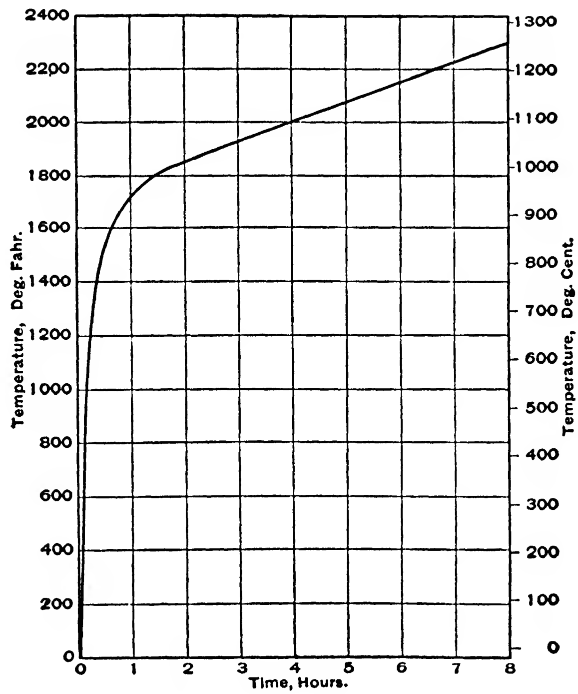


Fig. 1. Time-Temperature Curve
Courtesy of American Institute of Steel Construction

1000°F. at 5 minutes duration	1925°F. at 3 hours duration
1300°F. at 10 minutes duration	2000°F. at 4 hours duration
1550°F. at 30 minutes duration	2075°F. at 5 hours duration
1700°F. at 1 hour duration	2150°F. at 6 hours duration
1792°F. at 1 hour and 30 min.	2225°F. at 7 hours duration
1850°F. at 2 hours duration	2300°F. at 8 hours duration

(*c*) The temperature in the combustion chamber fixed by the curve shall be deemed the average obtained from the readings of several thermo-couples (not less than three) symmetrically located to show the temperature near all parts of the specimen being tested, the thermo-couples being enclosed and sealed in standard porcelain tubes $\frac{3}{4}$-inch outside diameter, and with walls $\frac{1}{8}$-inch thick. The exposed length of the thermo-couples and porcelain tubes shall extend not less than 12 inches into the combustion test chamber. Pyrometers or thermo-couple protecting tubes which are not standard may be used if under test conditions they give readings that are within the limits of accuracy that apply for furnace-temperature measurements. Other conditions defining the control of fire tests shall be as fixed by the above referred-to specifications.

FIRE HAZARDS

(*a*) For the purpose of this specification fire hazards shall be classified in accordance with the nature and combustibility of the materials which the buildings contain.

(*b*) Fire-resistive buildings shall be rated as to fire hazards due to interior fires on the basis of the temperature which their combustible contents will produce, and the time that is necessary to complete the burning of the combustible contents as compared to the standard time-temperature definition above given.

(*c*) The steel in the exterior walls of buildings shall be adequately protected against existing or probable maximum future exterior fire hazards.

(*d*) The occupancy rating of the fire hazards of fire-resistive buildings or parts thereof, shall be based upon the quantity of combustible contents per square foot of floor, and the equivalent intensity of fire which the contents will produce as compared to the standard time-temperature curve.

(*e*) All fire-resistive buildings, together with such parts as are used for the storage or handling of extra quantities of combustible materials, shall be fireproofed to protect them against maximum fire hazards within any section having fire separation from the remainder of the building, in accordance with the measured quantities of contained combustible materials including wood floor covering and wood trim. The burning out tests conducted by the Bureau of Standards with office occupancy and record storage, indicate the following equivalent fire hazard will approximately apply:

10 pounds per square foot constitutes a 1 hour fire hazard
15 pounds per square foot constitutes a $1\frac{1}{2}$ hour fire hazard
20 pounds per square foot constitutes a 2 hour fire hazard
30 pounds per square foot constitutes a 3 hour fire hazard
40 pounds per square foot constitutes a $4\frac{1}{2}$ hour fire hazard
50 pounds per square foot constitutes a 6 hour fire hazard
60 pounds per square foot constitutes a $7\frac{1}{2}$ hour fire hazard

The maximum fire hazard based on the weight of combustible materials shall be determined from the floor area of any one bay of the building or fire division thereof.

The classification, when applied to office equipment, is based upon the use of wood furniture and shelving. Other burning out tests of offices equipped with metal furniture and shelving, and with papers exposed by opened drawers, show a very substantial reduction of the fire hazard.

The combustible contents of some fire-resistive buildings may weigh less than 10 pounds per square foot of floor, but for the purpose of this specification no building shall be considered fire-resistive that is not constructed to resist a fire of at least one hour standard duration with the pertaining safety factor.

Fireproofing Specifications. The Uniform Building Code of the

Pacific Coast Building Officials' Conference has a complete and up-to-date section covering fireproofing requirements. A portion of this section is as follows:

CHAPTER 43—FIRE-RESISTIVE STANDARDS

Protection of Structural Parts

Sec. 4301. The thickness of fire-resistive materials for fire protection of structural parts shall be as shown in Table No I of this Chapter for the respective degree of fire protection shown. The figures shown shall be the net thickness of the protecting materials and shall not include any hollow space or spaces between the fire-protecting materials and the member protected. The thickness of plaster protection shall be measured from the face of the plaster to the plane of the back surface of the metal or wire lath where such lath is used and shall include two-thirds of the thickness of the gypsum plaster board (or lath) where such board (or lath) is used.

Grade A concrete shall mean concrete with a coarse aggregate of limestone, pumice, calcareous pebbles, trap rock, blast furnace slag, burnt clay, burnt shale or other coarse aggregates containing not more than 65 per cent of siliceous material, such as granite, sandstone, chert pebbles, flint, cinders or quartz.

Grade B concrete shall mean concrete with a coarse aggregate other than that allowed in Grade A concrete.

For flat ceilings where the ceiling protection for beams, girders or slabs is suspended to form a free air space of not less than one inch (1") between the member and the protection, the protection thicknesses may be one-half inch (½") less than that required in Table I of this Chapter for flat ceiling protection, but no thickness shall be less than three-fourths inch (¾").

Soffit tile protecting beam and girder flanges shall be tied to the flange with steel or iron ties.

If the structural part is of iron or steel the thickness given in Table No. I of this Chapter shall be measured outside of the extreme edges of the structural shapes, except that projecting edges of lugs and brackets shall be given a minimum protection of one inch (1") thickness. For reinforced concrete members, the thickness given in Table No. I of this Chapter shall be outside of the reinforcement. For purposes of design the protection shall not be considered as carrying load except as permitted for columns in Section 2620.

Plaster protections of over one inch (1") in thickness shall have an additional layer of metal lath, wire or metal mesh embedded not more than three-fourths inch (¾") from the surface and securely tied into the supporting members.

Wire of not less than No. 10 B. and S. gauge wound or tied around members at not more than a six-inch (6") pitch, or wire or expanded metal mesh shall be placed and well embedded in all concrete, poured gypsum and gunite protections.

Wire mesh or other forms of metal ties in concrete protections shall be held away from the structural members and embedded in the protection not less than three-fourths inch (¾") from its outer surface at points of minimum thickness. Hollow tile or gypsum block protections shall have iron or steel ties embedded in each horizontal joint, or have outside iron or steel ties over each unit, the diameter of wire to be 0.18 inch, or of equivalent area in ties of other forms. Wire mesh, where used for tying protections, shall weigh not less than one and one-half pounds per square yard. Where metal lath or wire mesh is used as a plaster base or tie it shall weigh not less than two and two-tenths pounds per square yard, and two and one-half or more meshes per inch or equivalent. Gypsum plaster board not less than three-eighths inch (⅜") thick and having not more than 15 per cent combustible material combined with the gypsum may be substituted for metal lath for resistance periods of not more than two hours, provided the plaster is reinforced with metal or wire mesh weighing not less than one and one-half pounds per square yard, standing away from the board and secured to the supporting members, and two-thirds of the thickness of the plaster-board may be considered as plaster.

Concrete aggregates whose mineral composition is unknown or undetermined shall for the application of these regulations be classed as Grade B aggregates.

Fire-Resistive Walls and Partitions

Sec. 4302. Fire-resistive bearing and non-bearing walls and partitions shall be of not less than the thicknesses and construction specified in this Chapter, to be classed for the respective degrees of protection indicated.

The structural requirements of the masonry and reinforced concrete walls shown in Table No. II of this Chapter are specified in Chapters 24 and 29 for the specific

TABLE No. I—Minimum Protection of Structural Parts Based on Time Periods for Various Incombustible Insulating Materials

Structural Parts to Be Protected	Insulating Material Used	Minimum Thickness of Material in Inches for the Following Fire Resistive Periods			
		4 hr.	3 hr.	2 hr.	1 hr.
Steel or Cast Iron Columns; Projecting Steel Beam or Girder Flanges; All Members of Primary Trusses	Grade A concrete	2	2	1½	1
	Grade B concrete	3	2½	2	1½
	Gunite	2	1½	1	¾
	Brick of clay, shale, concrete or sand-lime	3¾	3¾	2¼	1¼
	Clay tile, clay tile and concrete or concrete block (see Note 2)	4 or 2 pl.	4 or 2 pl.	2	2
	Solid gypsum blocks	2 pl.	2 pl.	2	2
	Hollow gypsum blocks	3 pl.	3	2	2
	Poured gypsum	2	1½	1	1
	Metal lath and gypsum or Portland cement plaster			2½*	1
Webs of Steel Beams and Girders	Grade A concrete	2	1½	1	1
	Grade B concrete	3	2½	1½	1
	Gunite	2	1½	1	¾
	Brick of clay, shale, concrete or sand-lime	3¾	2¼	2¼	2¼
	Clay tile, clay tile and concrete or concrete block	3 or 2 pl.	2	2	2
	Solid gypsum block	2 pl.	2	2	2
	Hollow gypsum block	3 pl.	2	2	2
	Poured gypsum	2	1½	1	1
	Metal lath and gypsum or Portland cement plaster			2	1
Reinforcing Steel in Reinforced Concrete Columns, Beams, Girders and Trusses	Grade A concrete	1½	1½	1½	1
	Grade B concrete	2	1½	1½	1
Reinforcing Steel in Reinforced Concrete Joists	Grade A concrete	1¼	1¼	1	¾
	Grade B concrete	1¾	1½	1	¾
Ceiling Protection for Roof Members including Roof Trusses and Secondary Trusses (See Note 3)	Metal or wire lath and gypsum or cement plaster, concrete, burned clay products or gypsum.	2	1½	1	¾
	Gunite	1½	1	¾	¾
Reinforcing and Tie Rods in Floor and Roof Slabs	Grade A concrete	1	1	¾	¾
	Grade B concrete (Thickness includes gypsum or cement plaster)	1¼	1	1	¾
	Gypsum	1	1	¾	¾

NOTE: (1) pl. in above table shall be not less than ½ in. gypsum or cement plaster.
(2) Reentrant parts of protected members shall be filled solid for 4 and 3 hour protections.
(3) See paragraph 4, Section 4301.
* Two ½-inch layers with ¾-inch air space between.

TABLE No. II—Rated Fire-Resistive Periods for Various Walls and Partitions

MATERIAL	CONSTRUCTION	Minimum Finished Thickness face to face (including plaster where mentioned) in Inches			
		4-hr.	3-hr.	2-hr.	1-hr.
Brick of Clay, Shale, Sand-Lime or Concrete, and Plain Concrete	Solid unplastered	8			4*
	Solid plastered	9		5*	
	Hollow (rowlock) unplastered	12	10	8	
	Hollow (rowlock) plastered	9			
Hollow Clay Tile Wall	End or side construction. One cell in wall thickness. Plastered				3*
	End or side construction. Two cells in 8-inch or less thickness. Unplastered	16	12		6* or 8
	End or side construction. Two cells in 8-inch or less thickness. Plastered	13	9	7*	
Hollow Clay Tile A. S. T. M. Load Bearing	End or side construction. Two cells in wall thickness. Unplastered			6	
	End or side construction. Two cells in wall thickness. Plastered			5*	
	End or side construction. Three cells in 8-inch or less thickness. Unplastered	12			
	End or side construction. Three cells in 8-in. or less thickness. Plastered one side.		8½		
	End or side construction. Three cells in 8-inch or less thickness. Plastered	9			
Combination of Brick and A.S.T.M. Load-Bearing Tile	4-inch brick and 4-inch tile plastered one side (tile side)	9			
"Special" Hollow Concrete Block or Tile	One cell in 8-in. or less thickness. Unplastered	12†	8	6	4*
	One cell in 8-in. or less thickness. Plastered	9†		5*	
Hollow Concrete Block or Tile	One cell in 8-in. or less thickness. Unplastered	16	12	10	6* or 8
	One cell in 8-in. or less thickness. Plastered	13	11	7* or 9	7*
Solid Concrete	Reinforcement not less than 0.2% in each direction	6	5*	4*	2*
Solid Gunite	Reinforcement not less than 0.2% in each direction	5* or 6	4*	3*	2*
Hollow Gypsum Blocks	Unplastered	6*	5*	4*	3*
	Plastered	5*	4*	4*	3*
Hollow Wall of Reinforced Gunite	Outer shell 2-in. thick for 10-in. wall and 1½ in. thick for 8-in. wall	10*	8*		
Solid Gypsum or Portland Cement Plaster	Incombustible studding with metal or wire lath				2*
Hollow Partition with Gypsum or Portland Cement Plaster or Gunite on Each Side	Incombustible studding with metal or wire lath. ¾ in. plaster on each side				3*
	Incombustible studding with metal or wire lath. 1 in. plaster on each side			4½*	
	Wood studs with metal or wire lath. Fire-stopped				3* or 5

*Indicates that such walls and partitions shall be used for non-bearing purposes only.
†This thickness to be given a four-hour rating only after an A. S. T. M. certified fire-test.

location or use of the walls and all walls shall comply with those structural requirements as well as the fire-resistive limitations as specified in this Chapter.

The tabulated thicknesses given in Table No. II of this Chapter are minimum and shall not be broken into; provided that where combustible floor or partition members project into solid masonry or reinforced concrete walls or partitions the required effective thickness of wall shall be measured from two inches (2") back along the member from the end in the wall, to the opposite face of the wall. Where such members project into hollow walls and the space between the members and for not less than four inches (4") above and below them is filled solid with fire-resistive incombustible materials for the full thickness of the wall, or where such hollow walls are constructed of hollow units laid with cells horizontal (side construction) the required thickness shall be measured as specified for solid masonry walls in this paragraph. Where the hollow spaces are not thus filled or where hollow units are laid with cells vertical (end construction) the required effective thickness of wall shall be measured from the end of member in wall to the opposite face of wall.

Plaster, in order that it may be considered as adding to the fire resistance of walls and partitions shall be gypsum or Portland cement plaster applied to an average thickness of not less than one-half inch (½") on each side. Plaster over one inch (1") in thickness, as measured to the plaster base, shall have an additional layer of metal lath, wire or metal mesh embedded not more than three-fourths inch (¾") from the surface and securely tied into the supporting members.

Required fire-resistive plastering or stucco on the outside of exterior masonry walls may be omitted from inaccessible portions of the wall provided the inside plastering opposite the inaccessible portions is doubled in thickness.

Gypsum plaster board (or lath) not less than three-eighths inch (⅜") in thickness and having not more than 15 per cent of combustible material combined with the gypsum may be substituted in interior construction for metal lath for resistance periods of not more than two hours, provided the plaster is reinforced with metal or wire mesh weighing not less than one and one-half pounds per square yard, standing away from the board (or lath) and secured to the supporting studs or joists. Two-thirds of the thickness of the plaster board (or lath) may be considered as plaster.

Gypsum plaster board (or lath) conforming to the specifications contained in the preceding paragraph may be substituted for metal lath and the reinforcement of the plaster omitted, provided that the joints of the plaster board (or lath) are covered with strips of metal fabric not less than three inches (3") in width and the plaster board (or lath) is plastered with not less than one-half inch (½") of fibered gypsum plaster containing not more than 33⅓ per cent by weight of silica.

Metal or wire lath shall weigh not less than two and two-tenths pounds per square yard. Metal or wire mesh where used as ties in concrete shall weigh not less than one and one-half pounds per square yard. Where used as ties for plaster it shall weigh not less than two and two-tenths pounds per square yard and have not less than two and one-half meshes per inch, or equivalent.

Wood studs for bearing partitions or walls shall be not less than the two inch by four inch (2"x4") nominal size and be spaced not more than sixteen inches (16") apart.

Note: The term "plastered" in Table No. II, in this Chapter, shall mean walls plastered with not less than one-half inch (½") of gypsum or Portland cement plaster on each side of wall.

Fire-Resistive Floor Construction

Sec. 4303. Fire-resistive floor construction shall be accepted for the following respective degrees of fire-resistive protection when constructed as specified in this Section. For the structural details of any floor construction, the particular details specified under Part VI of this Code shall govern.

Four-hour, three-hour and two-hour fire-resistive floors as specified in this Section shall be constructed entirely of incombustible materials.

(a) Four-Hour. Four-hour fire-resistive floor construction shall consist of reinforced concrete, gypsum and/or solid masonry slabs or arches not less than four inches (4") in thickness or shall consist of hollow masonry slabs or arches not less than four inches (4") in thickness with a top covering of not less than two inches (2") of solid masonry, or shall consist of steel joists protected with fire-resistive materials of the kind and thickness shown in Table No. III in this Chapter. Except in the case of steel joisted construction, all reinforcing, tie rods and supporting structural members in such floors shall be protected with not less than four-hour fire-resistive construction as specified in Section 4301.

(b) Three-Hour. Three-hour fire-resistive floor construction shall consist of reinforced concrete, gypsum and/or solid masonry slabs or arches not less than three inches (3") in thickness or shall consist of hollow masonry slabs or arches not less than four inches (4") in thickness with a top covering of solid masonry not less than one and one-half inches (1½") in thickness, or shall consist of steel joists protected with fire-resistive materials of the kind and thickness shown in Table No. III in this Chapter. Except in the case of steel joisted construction all reinforcing, tie rods and supporting structural members in such floor construction shall be protected with not less than three-hour fire-resistive construction as specified in Section 4301.

TABLE No. III—Minimum Protection for Metal and Wood Joists Based on Time Periods for Various Insulating Materials

Joists to be Protected	Insulating Material Used	Minimum Thickness of Material in Inches for the Following Fire-Resistive Periods			
		4-hr.	3-hr.	2-hr.	1-hr.
Ceiling protection of Steel Joists, where incombustible slab not less than $2\frac{1}{2}$ in. thick is placed above	Metal or wire lath and gypsum or Portland cement plaster, concrete, burned clay products or gypsum	2	$1\frac{1}{2}$	1	$\frac{3}{4}$
	Gunite	$1\frac{1}{2}$	1	$\frac{3}{4}$	$\frac{3}{4}$
Ceiling Protection of Wood Joists with double floor on top	Metal or wire lath and gypsum or Portland cement plaster				$\frac{3}{4}$

(c) **Two-Hour.** Two-hour fire-resistive floor construction shall consist of reinforced concrete, gypsum and/or solid masonry slabs or arches not less than two and one-half inches ($2\frac{1}{2}''$) in thickness or shall consist of hollow masonry slabs or arches not less than three inches (3") in thickness with a top covering of not less than one inch (1") of solid masonry, or shall consist of steel joists protected with fire-resistive materials of the kind and thickness shown in Table No. III in this Chapter. Except in the case of steel joisted construction all reinforcing, tie rods and supporting structural members in such floor construction shall be protected with not less than two-hour fire-resistive construction as specified in Section 4301.

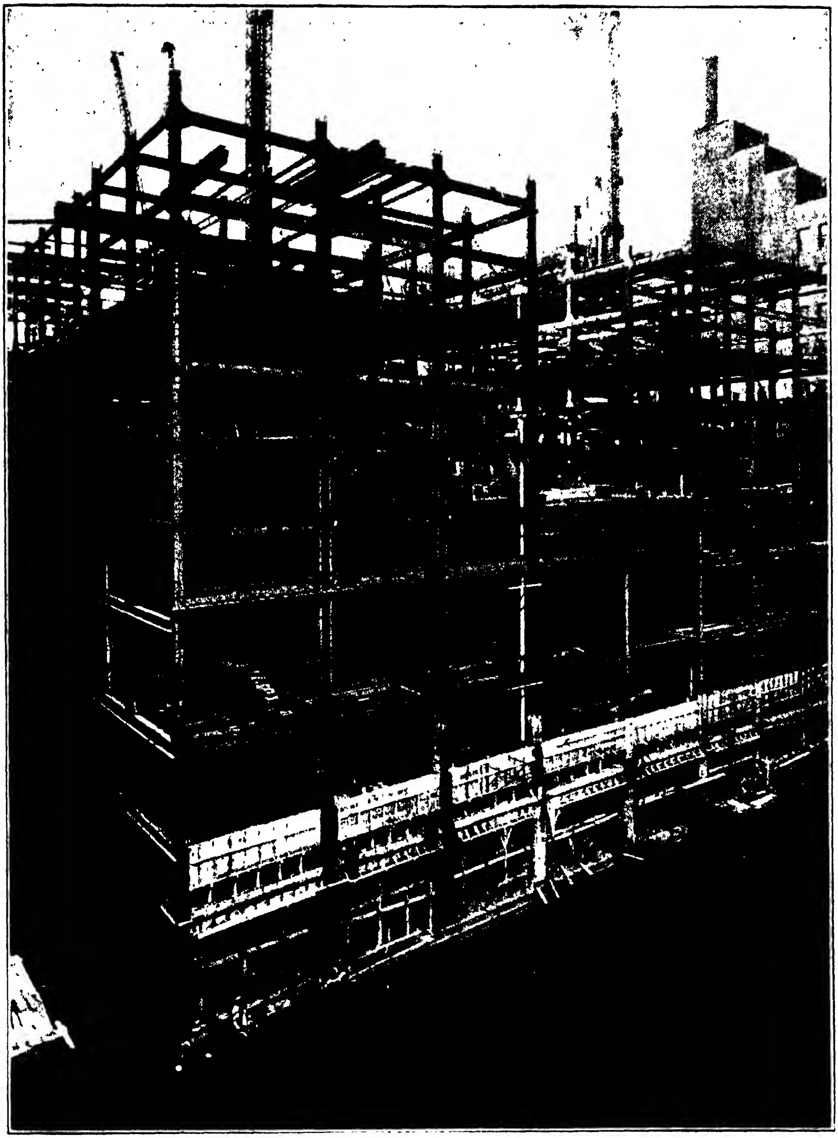
(d) **One-Hour.** One-hour fire-resistive floor construction shall consist of one of the following.

1. Reinforced concrete, gypsum and/or solid masonry slabs or arches not less than two and one-half inches ($2\frac{1}{2}''$) in thickness.
2. Hollow masonry slabs or arches not less than three inches (3") in thickness with all joints in such hollow unit construction thoroughly filled with cement or gypsum mortar.
3. Steel joists protected with fire-resistive materials of the kind and thickness shown in Table No. III in this Chapter.
4. Wood joisted construction with a double wood floor on top (the subfloor not less than three-fourths inch ($\frac{3}{4}''$) thick, and the total thickness of the two layers not less than one and one-fourth inches ($1\frac{1}{4}''$) thick) and with a fire-resistive ceiling, as shown in Table No. III of this Chapter, securely fastened to or suspended from the under side of such joists, except the metal lath and plaster ceiling shall not be required below the lowest floor joist over unusable space.

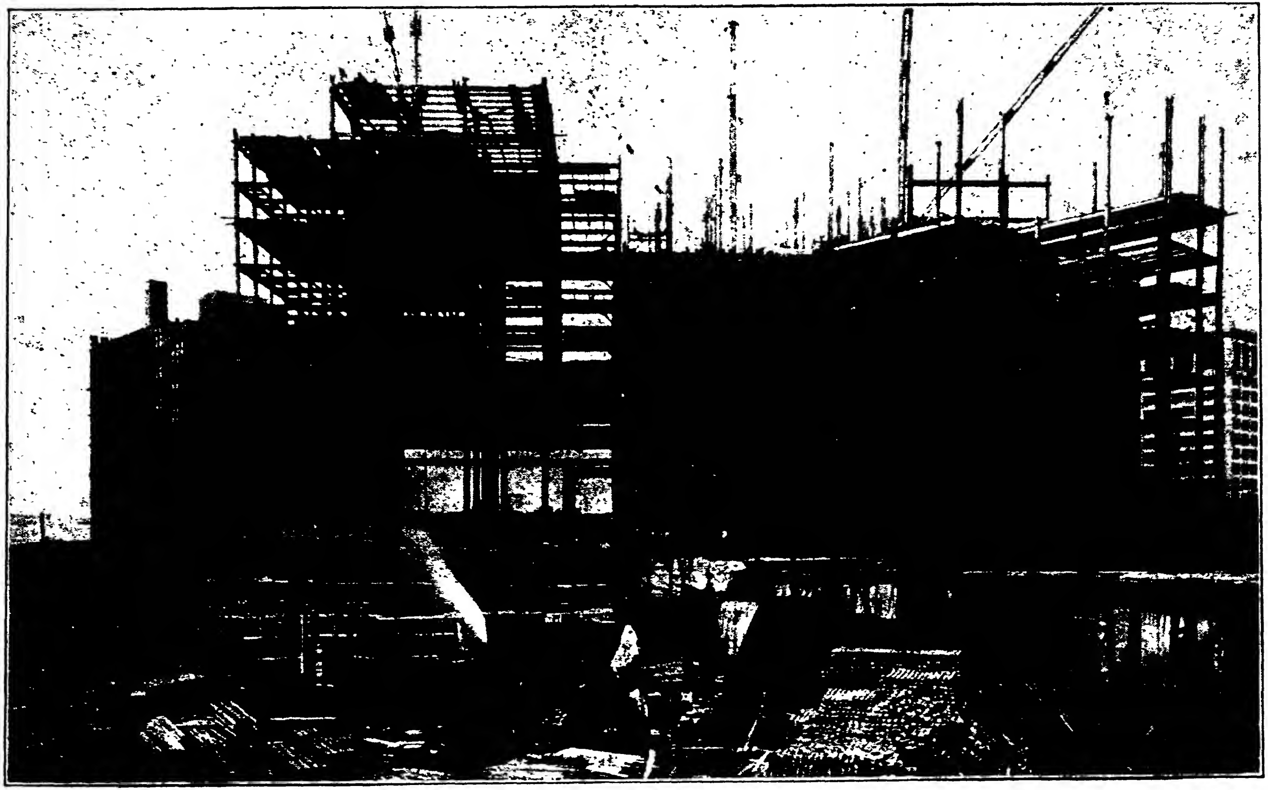
Except in the case of steel joisted construction, all reinforcing, tie rods and supporting structural members shall be protected with not less than one-hour fire-resistive construction as specified in Section 4301.

All flat ceilings where the ceiling protection for beams, girders or slabs is suspended to form a free air space between the member and the protection, the protection thicknesses may be one-half inch ($\frac{1}{2}''$) less than that required in Table No. III in this Chapter for flat ceiling protection, but no thickness shall be less than three-fourths inch ($\frac{3}{4}''$) minimum protection of metal and wood joists based on time periods for various insulating materials.

In any reinforced concrete floor construction which includes a metal lath and cement or gypsum plastered ceiling on the under side, not less than three-fourths inch ($\frac{3}{4}''$) thick, the required slab thickness may be reduced one-half inch ($\frac{1}{2}''$) but in no case shall be less than two and one-half inches ($2\frac{1}{2}''$) thick.



THE PALMER HOUSE, CHICAGO
Note Heavy Truss Framing over One of the Ball Rooms
Courtesy of Holabird and Root, Architects



THE TERMINAL BUILDING, CLEVELAND
Courtesy of Graham, Anderson, Probst and White, Architects

APPENDIX

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 that can be more easily defined than shown on the drawings. They describe the required quality of the materials and workmanship. The specification must cooperate 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, the specification includes certain items related to the business transaction between the purchaser and the contractor.

The specification for a building covers a vast number of materials and processes; therefore it is desirable to refer to some authoritative source for information and assistance. Such a source is the Specification Record of the American Specification Institute. In their *Specification Record*, Volume IV, the following informative discussion regarding specifications in general is given.

A specification as used for buildings and engineering construction and equipment should be a qualitative description of the materials, methods and workmanship, which, when assembled in orderly fashion in accordance with a studied arrangement as expressed in the drawings accompanying it, will produce a building or engineering structure or equipment that is economically constructed for efficient use. A specification must be brief, concise, coherent, comprehensive without verbosity and an explicit statement of all elements necessary to the satisfactory accomplishment of the desired result.

In almost all cases, an engineering or architectural specification is accompanied by drawings that express, in a quantitative manner, the desire of their author in the fabrication, manufacture or construction of the engineering or architectural structure or equipment. Thus the scope of any project is usually outlined by linear measurements and illustrations or architectural, structural or equipment requirements in the drawings and through description by the specifications so that the estimator, the contractor, the owner and all others concerned in the accomplishment of the work have all the necessary data available.

The engineer or architect who is responsible for the design and construction of a building or engineering structure must be intimately acquainted with a vast number of materials and processes and, as his knowledge of these and his skill in their use are increased he finds his object more easy of accomplishment. The person who writes specifications for such structures must have a thorough knowledge of the subject matter of the various subdivisions of his specifications and since the average mortal cannot hope to learn, through personal experience, all he must know in order that he may achieve success, he must look to some source or sources of authoritative information for assistance.

Every specification comprises certain elements that are necessary in order to convey to the mind of the reader all fundamentals that will govern in the execution of the work. Although it is not expressed in so many words, practically all specifications have been prepared with a view to the economic use of materials and methods, consistent with the component parts available to use and with the results that are considered most desirable. Economic values always must be considered, else the wasteful processes that will be encouraged by a disregard of such consideration will bring to the engineer or architect a reputation that will not be to his credit.

Certain standards of excellence in workmanship or materials must be specified so accurately that there will be no question as to the aims of the author of the specifications. Likewise it is oftentimes necessary or desirable to present to the one who is to execute the work a choice of several alternative ways of accomplishing the ends sought. In order that both of these elements may be provided for with scientific precision, it is necessary that the architect or engineer become acquainted with the standards and alternatives that are available for use and that are most desirable for any particular operation. The description of materials, methods and their use cannot be made in a brief, concise, coherent and accurate manner unless the one writing the specification has, at his finger tips, all facts that will be of assistance to him in formulating his judgment.

Another element of specifications is the use made of them as the instrument of instruction to all concerned as to how each step in the work is to be accomplished. Specifications should not only describe materials and methods but they should, in addition, instruct the men engaged in active construction or installation work as to the means by which the materials and methods are to be used in order that results will conform with those conceived in the mind of the engineer or architect. This element is intimately bound in with the general descriptive element of specifications, yet it should not be confused with it.

All specifications for building and engineering structures and for many classes of equipment or co-related operations must be accompanied by general contract conditions and instructions to bidders. It is rare to find these elements missing although they may not have been given sufficient consideration to give them prominence.

Specifications may be divided into four classes insofar as their contents are concerned. These classes are materials, methods, construction and equipment. Many specifications and, in fact, most specifications that are used in building and engineering structures, combine three of these four elements, that is to say, for building and engineering structures the specifications will embrace materials, methods and construction, while for the equipment there will be these same elements with a sub-element of installation after shop construction.

Specifications are essential to the proper and orderly conduct of the business of all those concerned in buildings, engineering structures of all classes and for all manner of equipment of whatever kind which may be placed in or about such works. We have, then, the following classification of users of specifications: Engineers, architects, contractors, manufacturers, vendors, labor, owners, buyers, operators.

Each one of these classes is vitally interested in having at his disposal good specifications, the intent and purpose of which cannot be questioned. To each of them a good specification brings the assurance that there will be smooth operation and cordial relations for all concerned; whereas mediocre specifications mean everything but cordiality and happiness.

Owing to the present lack of means for collecting and distributing information concerning specifications there is a needless duplication of study, research and labor on the part of specification writers.

This condition tends to make the work seem arduous, as it is quite often, if there has been no effort expended toward meeting the conditions present in the individual office. Those specification writers who have had sufficient vision to analyze the problems that they must meet and who have attempted to organize their work in some more or less methodical fashion, have been gratified to find the time so spent has been well spent.

Practically all other professions are so organized that the interchange of knowledge peculiar to their profession, such as the deliberations of committees which formulate proposed standards for basic operations and the results of researches undertaken by scientific laboratories, is effected in such a way as to result in the improvement of the quality of specifications produced and as a direct consequence, has resulted in an improvement in the professional and business standing of their authors.

THE AMERICAN SPECIFICATION INSTITUTE is endeavoring to improve all those conditions surrounding the writing of specifications and to bring to specification writers the benefits that are to be obtained from organized efforts of men accustomed to study and write these essential documents.

The kinds of specifications that are being studied and for the preparation of which informative data is compiled and distributed to members, include those for buildings, engineering structures and all works whatsoever in which materials of construction and labor are used; those for the installation and use of mechanical, electrical and sanitary apparatus and equipment; those for the fabrication and installation of all furnishings and furniture; those for all exterior and interior ornaments and ornamentation; those for road paving, planting, embellishing and improving all landscapes, estates and waterways; and those for all miscellaneous matters and things that are produced and offered for sale under specifications written by the engineer or architect.

The following general specification outline is also in the Specification Record and shows the large number of items that may have to be covered in a building specification.

I. Preliminary to Contract
- Matters Preliminary to Contract

II. Contract Documents
- Agreement and Schedule of Conditions of Contract

III. Preliminary to Construction
- Work Preliminary to Construction
- Construction Plant
- Inspection of Materials
- Wrecking and Clearing Site
- Surveying and Grades

IV. General Excavation
- Excavation

V. Foundations
- Piling
- Caisson Foundations
- Footings and Foundation Walls
- Waterproofing and Dampproofing

VI. Structural Steel
- Manufacture and Fabrication of Structural Steel
- Erection of Structural Steel

VII. Concrete Masonry
- Concrete Masonry
- Finished Cement Floors
- Sidewalk Lights
- Reinforced Steel
- Architectural Cast Concrete
- Terrazzo Work

VIII. Terra Cotta and Tile Work
- Fireproofing (Terra Cotta)
- Cohesive Tile Work
- Architectural Terra Cotta
- Mosaic and Faience Tile Work

IX. Stone Masonry
- Stone Masonry

X. Brick Masonry
- Brick Masonry

XI. Carpentry
- Rough Carpentry Work
- Interior Wood Trim and Finish
- Wood Cabinet Work

XII. Miscellaneous Steel, Iron, Etc.
- Ornamental Bronze and Iron Work
- Miscellaneous Steel Work
- Fire Doors and Shutters
- Solid Metal Windows

XIII. Sheet Metal Work
- Fireproof Hollow Metal Windows
- Steel Partitions
- Sheet Metal Work
- Skylights
- Hollow Metal Trim

XIV. Roofing
- Slate Roofing
- Tile Roofing
- Composition Roofing

XV. Furring, Lathing and Plastering
- Furring and Lathing
- Plain and Ornamental Plastering
- Scagliola

XVI. Models
- Models for All Work

XVII. Structural Slate Work
- Interior Structural Slate Work

XVIII. Marble Work
- Stone Masonry
- Interior Marble Work

XIX. Other Floors
- Composition Floors
- Wood Block Floors
- Special Floors

XX. Glass
- Glass and Glazing
- Structural Glass

XXI. Painting
- Painting

XXII. Decorations and Furnishings
- Decorations
- Draperies, Rugs and Carpets
- Window Shades
- Awnings

XXIII. Sound Control
- Sound Insulation
- Acoustical Correction

XXIV. Hardware, Etc.
- Hardware
- Weather Strips
- Window Screens

XXV. Elevators
- Elevators
- Elevator Signals
- Dumbwaiters
- Escalators

XXVI. Special Installations
- Pneumatic Tubes
- Conveyors
- Mail Chute
- Master and Secondary Clocks
- Vacuum Cleaning

XXVII. Electrical Work
- Express Call System
- Telegraph Call System
- Electrical Recording System
- Telephone Systems
- Electrical Call Systems
- Electrical Signal Systems
- Electrical Conduits and Wiring
- Electrical Machinery and Equipment
- Lighting Fixtures

XXVIII. Plumbing Work
- Sewage and Drainage
- Plumbing
- Pumps and Ejectors
- Gas Fitting
- Water Purification
- Water Softening
- Drinking Water Systems
- Fire Protection Equipment
- Automatic Sprinkler Systems
- Refrigeration Machinery and Piping

XXIX. Insulation
- Cold Storage Insulation
- Heat Insulation

XXX. Heating and Ventilating
- Heating
- Mechanical Ventilation
- Power Plant Work and Equipment
- Smoke Stack

XXXI. Miscellaneous
- Permanent Building Equipment
- Furniture and Fixtures
- Landscape Architecture
- Garden Furniture
- Sculpture
- Walks
- Roads and Streets
- Bridges
- Wells
- Miscellaneous

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

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

PART I. GENERAL

Section 1. Scope.

(a) Scope.

This Specification defines the practice adopted by the American Institute of Steel Construction in the design, fabrication, and erection of structural steel for buildings.

(b) Code.

In the execution of contracts entered into under this Specification, the Code of Standard Practice for Buildings of the American Institute of Steel Construction shall apply unless otherwise specified or required.

Section 2. Plans and Drawings.

(a) Plans.

The plans shall show a complete design with sizes, sections, and the relative location of the various members. Floor levels, column centers, and off-sets shall be dimensioned. Plans shall be drawn to a scale large enough to convey the information adequately.

(b) Shop Drawings.

Shop drawings shall be made in conformity with the best modern practice and with due regard to speed and economy in fabrication and erection.

PART II. MATERIAL

Section 3. Material.

(a) Structural Steel.

Structural steel shall conform to the Standard Specifications of the American Society for Testing Materials for Steel for Buildings, Serial Designation A 9 (or, if so specified by the Buyer, for Steel for Bridges, Serial Designation A 7), as amended to date.

(b) Rivet Steel.

Rivet steel shall conform to the Standard Specifications of the American Society for Testing Materials for structural Rivet Steel, Serial Designation A 141, as amended to date.

(c) Other Metals.

Alloy steels, cast steel, cast iron and other metals shall conform to the applicable Specifications of the American Society for Testing Materials, as amended to date.

(d) Stock Material.

Stock material shall be of a quality equal to that called for by the specifications of the American Society for Testing Materials for the classifications covering its intended use; and mill test reports shall constitute sufficient record as to the quality of material carried in stock.

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

PART III. LOADS AND STRESSES

Section 4. Loads and Forces.

(a) Loads and Forces.

Steel structures shall be designed to sustain the following loads and forces:

1. Dead Load.
2. Live Load.
3. Impact.
4. Wind and other Lateral Forces.
5. Erection Loads.
6. Other Forces.

(b) Dead Load.

The dead load shall consist of the weight of the steelwork and all material fastened thereto or supported thereby.

(c) Live Load.

The live load shall be that stipulated by the Code under which the structure is being designed or that required by the conditions involved. In general, the live loads should not be less than those recommended by the Building Code Committee of the National Bureau of Standards, November, 1924, under the caption "Minimum Live Loads for use in the Design of Buildings."

(d) Impact.

For structures, carrying live loads inducing impact or vibrational forces, the design stresses shall be increased by a percentage of the live load stresses sufficient to suitably provide for such forces.

(e) Wind.

Proper provision shall be made for stresses caused by wind both during erection and after completion of the building. The wind pressure is dependent upon the conditions of exposure and geographical location of the structure. The allowable stresses specified in Sections 6 (c) and 7 are based upon the steel frame being designed to carry a wind pressure of not less than twenty (20) pounds per square foot on the vertical projection of exposed surfaces during erection, and fifteen (15) pounds per square foot on the vertical projection of the finished structure.

(f) Erection.

Proper provision shall be made for temporary stresses caused by erection.

(g) Other Forces.

Structures in localities subject to earthquakes, hurricanes, and other extraordinary conditions shall be designed with due regard for such conditions.

Section 5. Reversal of Stress.

(a) Reversal of Stress.

Members subject to live loads producing alternating tensile and compressive stresses shall be proportioned as follows:

To the net total compressive and tensile stresses add 50 per cent of the smaller of the two and proportion the member to resist either of the increased stresses resulting therefrom.

Connections shall be proportioned to resist the larger of the two increased stresses.

Section 6. Combined Stresses.

(a) Axial and Bending.

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

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

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

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

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

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

(b) **Rivets.**

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

(c) **Wind and Other Forces.**

Members subject to stresses produced by a combination of wind and other loads may be proportioned for unit stresses 33⅓ per cent greater than those specified in Section 10, provided the section thus required is not less than that required for the combination of dead load, live load, and impact (if any).

Section 7. Members Carrying Wind Only.

(a) **Wind Only.**

Members subject only to stresses produced by wind forces may be proportioned for unit stresses 33⅓ per cent greater than those specified in Section 10.

Section 8. Composite Beams.

(a) **Composite Beams.**

The term "composite beam" shall apply to any rolled or fabricated steel floor beam entirely encased in a poured concrete haunch at least four inches wider, at its narrowest point, than the flange of the beam, supporting a concrete slab on each side without openings adjacent to the beam; provided that the top of the beam is at least 1½ inches below the top of the slab and at least 2 inches above the bottom of the slab; provided that a good grade of stone or gravel concrete with Portland cement, is used; and provided that the concrete haunch has adequate mesh, or other reinforcing steel, throughout its whole depth and across its soffit.

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

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

(c) **Composite Beams.**

The total tensile unit stress in the extreme fiber of the steel beam thus computed shall not exceed 20,000 pounds per square inch. [Section 10 (a)].

(d) The maximum stresses in the concrete, and the ratio of Young's moduli for steel and concrete, shall be as prescribed by the specifications governing the design of reinforced concrete for the structure.

(e) The web and the end connections of the steel beam shall be adequate to carry the total dead and live load without exceeding the unit stresses prescribed in this Specification, except as this may be reduced by the provision of other proper support.

Section 9. Effective Span Length.

(a) **Simple Spans.**

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

(b) **End Restraint.**

When designed on the assumption of end restraint, full or partial, based on continuous or cantilever action, beams, girders and trusses, as well as the sections of the members to which they connect, shall be designed to carry the shears and moments so introduced, in addition to all other forces, without exceeding at any point the unit stresses prescribed in Section 10.

PART IV. UNIT STRESSES

Section 10. Allowable Unit Stresses.

(a) **Structural and Rivet Steel.**

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

Tension		
Structural Steel, net section		20 000
Rivets, on area based on nominal diameter		15 000
Compression		
Columns, gross section		
For axially loaded columns with values of l/r not greater than 120		$17\,000-0.485\,\frac{l^2}{r^2}$
For axially loaded columns with values of l/r greater than 120		$\dfrac{18\,000}{1+\dfrac{l^2}{18\,000\,r^2}}$
in which l is the unbraced length of the column, and r is the corresponding radius of gyration of the section, both in inches.		
Plate Girder Stiffeners, gross section		20 000
Webs of Rolled Sections at toe of fillet [Crippling, see Section 19 (h)]		24 000
Bending		
Tension on extreme fibers of rolled sections, plate girders, and built-up members. [See Section 19 (a)]		20 000
Compression on extreme fibers of rolled sections, plate girders, and built-up members, for values of l/b not greater than 40		$\dfrac{22\,000}{1+\dfrac{l^2}{1800\,b^2}}$ with a maximum of 20,000
in which l is the laterally unsupported length of the member, and b is the width of the compression flange, both in inches.		
Stress on extreme fibers of pins		30 000
Shearing		
Rivets		15 000
Pins, and turned bolts in reamed or drilled holes		15 000
Unfinished bolts		10 000
Webs of beams and plate girders, gross section		13 000

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

(b) **Cast Steel.**

Compression and Bearing same as for Structural Steel. Other Unit Stresses, 75 per cent of those for Structural Steel.

(c) **Masonry [Bearing]**

Granite	800
Sandstone and Limestone	400
Concrete, unless otherwise specified	600
Hard Brick in Cement Mortar	250

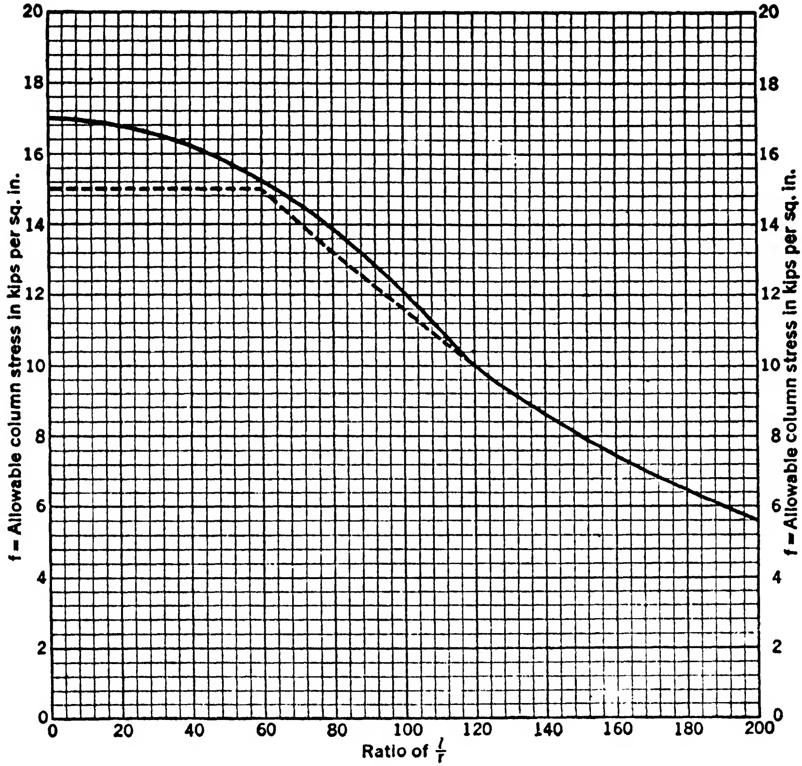
PART V. DESIGN

Section 11. Slenderness Ratio.

(a) The ratio of unbraced length to least radius of gyration $\frac{l}{r}$ for compression members shall not exceed:

For main compression members	120
For bracing and other secondary members in compression	200

Allowable Working Stress for Axially Loaded Columns



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

Section 12. Unsupported Compression Flanges.

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

Section 13. Minimum Thickness of Material

(a) Main Material.

The minimum thickness of steel except for linings, fillers, and the webs of rolled beams and channels, shall be: for exterior construction—5/16 inch; for interior construction—1/4 inch. (These provisions do not apply to light structures such as skylights, marques, fire-escapes, light one-story buildings, or light miscellaneous steelwork.)

(b) Gusset Plates.

Gusset plates for trusses with end reactions greater than 35,000 pounds shall be not less than 3/8 inch thick.

Allowable Stresses Per Square Inch for Beams and Girders Laterally Unsupported

$$f = \frac{22500}{1 + \frac{l^2}{1800\ b^2}}$$

l = Unsupported length in inches
b = Width of flange in inches.

$\frac{l}{b}$	Unit Stress f (Kips)	Ratio	$\frac{l}{b}$	Unit Stress f (Kips)	Ratio	$\frac{l}{b}$	Unit Stress f (Kips)	Ratio
15.0	20.00	1.000	23.5	17.22	.861	32.0	14.34	.717
15.5	19.85	.993	24.0	17.05	.852	32.5	14.18	.709
16.0	19.70	.985	24.5	16.87	.844	33.0	14.02	.701
16.5	19.54	.977	25.0	16.70	.835	33.5	13.86	.693
17.0	19.39	.969	25.5	16.53	.826	34.0	13.70	.685
17.5	19.23	.961	26.0	16.36	.818	34.5	13.54	.677
18.0	19.07	.953	26.5	16.19	.809	35.0	13.39	.669
18.5	18.91	.945	27.0	16.01	.801	35.5	13.23	.662
19.0	18.74	.937	27.5	15.84	.792	36.0	13.08	.654
19.5	18.58	.929	28.0	15.67	.784	36.5	12.93	.647
20.0	18.41	.920	28.5	15.50	.775	37.0	12.78	.639
20.5	18.24	.912	29.0	15.34	.767	37.5	12.63	.632
21.0	18.07	.904	29.5	15.17	.758	38.0	12.49	.624
21.5	17.90	.895	30.0	15.00	.750	38.5	12.34	.617
22.0	17.73	.887	30.5	14.83	.742	39.0	12.20	.610
22.5	17.56	.878	31.0	14.67	.733	39.5	12.05	.603
23.0	17.39	.869	31.5	14.50	.725	40.0	11.91	.596

(c) Angles.

The widths of the outstanding legs of angles in compression (except where reinforced by plates) shall not exceed 12 times the thickness for girder flanges and 16 times the thickness for other members.

(d) Compression Members.

In compression members consisting of segments connected by cover plates or lacing, or segments connected by webs, the thickness of the webs of the segments shall be not less than 1/32 of the unsupported distance between the nearest rivet lines, or the roots of the flanges in case of rolled sections. The thickness of the cover plates or webs connecting the segments shall be not less than 1/40 of the unsupported distance between the nearest lines of their connecting rivets, or the roots of their flanges in case of rolled sections.

(e) Corrosion.

Provision shall be made for parts subject to corrosive agents, either by increasing the thickness of material or by effective protection.

Section 14. Gross and Net Sections.

(a) Definitions.

The gross section of a member at any point shall be determined by summing the products of the thickness and the gross width of each element as measured normal to the axis of the member. The net section shall be determined by substituting for the gross width the net width computed in accordance with paragraphs (c) to (g) of this Section.

(b) Application.

Unless otherwise specified, tension members shall be designed on the basis of net section. Columns shall be designed on the basis of gross section. Beams and girders shall be designed in accordance with Section 19 (a).

(c) Net Width.

In the case of a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain, and adding, for each gage space in the chain, the quantity

$\frac{s^2}{4g}$, where

s = longitudinal spacing (pitch) in inches of any two successive holes.
g = transverse spacing (gage) in inches of the same two holes.
The critical net section of the part is obtained from that chain which gives the least net width.

(d) **Angles.**

For angles, the gross width shall be the sum of the widths of the legs less the thickness. The gage for holes in opposite legs shall be the sum of the gages from back of angle less the thickness.

(e) **Splice Members.**

For splice members, the thickness considered shall be only that part of the thickness of the member which has been developed by rivets beyond the section considered.

(f) **Size of Holes.**

In computing net area the diameter of a rivet hole shall be taken as ⅛ inch greater than the nominal diameter of the rivet.

(g) **Pin Holes.**

In pin connected tension members, the net section across the pin hole, transverse to the axis of the member, shall be not less than 140 per cent, and the net section beyond the pin hole, parallel with the axis of the member, not less than 100 per cent, of the net section of the body of the member.

In all pin-connected riveted members the net width across the pin hole, transverse to the axis of the member, shall preferably not exceed 12 times the thickness of the member at the pin.

Section 15. Expansion.

(a) **Expansion.**

Proper provision shall be made for expansion and contraction.

Section 16. Connections.

(a) **Minimum Connections.**

Connections carrying calculated stresses, except for lacing, sag bars, and girts, shall have not fewer than 2 rivets.

(b) **Eccentric Connections.**

Members meeting at a point shall have their gravity axes meet at a point if practicable; if not, provision shall be made for their eccentricity.

(c) **Rivets.**

The rivets at the ends of any member transmitting stresses into that member should preferably have their centers of gravity on the gravity axis of the member: otherwise, provision shall be made for the effect of the resulting eccentricity. Pins may be so placed as to counteract the effect of bending due to dead load.

(d) **Unrestrained Members.**

When beams, girders or trusses are designed on the basis of simple spans in accordance with Section 9 (a), their end connections may ordinarily be designed for the reaction shears only. If, however, the eccentricity of the connection is excessive, provision shall be made for the resulting moment.

(e) **Restrained Members.**

When beams, girders or trusses are subject both to reaction shear and end moment, due to the restraint specified in Section 9 (b), their connections shall be specially designed to carry both shear and moment without exceeding at any point the unit stresses prescribed in Section 10. Ordinary end connections comprising only a pair of web angles, with not more than a nominal seat and top angle, shall not be assumed to provide for this kind of end moment.

(f) **Fillers.**

In truss construction when rivets carrying computed stress pass through fillers, the fillers shall be extended beyond the connected member and the extension secured by sufficient rivets to develop the stress in the filler.

Fillers under plate girder stiffeners at end bearings or points of concentrated loads shall be secured by sufficient rivets to prevent excessive bending and bearing stresses.

(g) **Splices.**

Compression members when faced for bearing shall be spliced sufficiently to hold the connecting members accurately in place. Other joints in riveted work, whether in tension or compression, shall be spliced so as to transfer the stress to which the member is subject.

Section 17. Rivets and Bolts.

(a) **Diameter.**

In proportioning and spacing rivets, the nominal diameter of the undriven rivet shall be used.

(b) **Effective Bearing Area.**

The effective bearing area of pins, bolts, and rivets shall be the diameter multiplied by the length in bearing; except, that for countersunk rivets, half the depth of the countersink shall be deducted.

(c) **Long Grips.**

Rivets carrying calculated stress, and whose grip exceeds five diameters, shall have their number increased 1 per cent for each additional $\frac{1}{16}$ inch in the rivet grip. Special care shall be used in heating and driving such rivets.

(d) **Use of Unfinished Bolts.**

All field connections may be made with unfinished bolts, except as provided in Par. (e) hereof.

(e) **Use of Rivets.**

Rivets shall be used for the following connections:

Column splices in all tier structures 200 feet or more in height.

Column splices in tier structures 100 to 200 feet in height, if the least horizontal dimension is less than 40 per cent of the height.

Column splices in tier structures less than 100 feet in height, if the least horizontal dimension is less than 25 per cent of the height.

Connections of all beams and girders to columns, and of any other beams and girders on which the bracing of columns is dependent, in structures over 125 feet in height.

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

Connections for supports for running machinery, or of other live loads which produce impact or reversal.

Any other connections stipulated on the design plans.

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

(f) **Use of Turned Bolts.**

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

(g) **Main Members.**

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

Section 18. Rivet Spacing.

(a) **Minimum Pitch.**

The preferable minimum distance between centers of rivet holes shall be not less than $4\frac{1}{2}$ inches for $1\frac{1}{4}$ inch rivets, 4 inches for $1\frac{1}{8}$ inch rivets, $3\frac{1}{2}$ inches for 1 inch rivets, 3 inches for $\frac{7}{8}$ inch rivets, $2\frac{1}{2}$ inches for $\frac{3}{4}$ inch rivets, 2 inches for $\frac{5}{8}$ inch rivets, and $1\frac{3}{4}$ inches for $\frac{1}{2}$ inch rivets, but in no case shall it be less than three times the diameter of the rivet.

(b) **Maximum Pitch Compression Members.**

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

(c) **End Pitch Compression Members.**

The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets for a length equal to 1½ times the maximum width of the member.

(d) **Two-Angle Members.**

In tension members composed of two angles, a pitch of 3′-6″ will be allowed, and in compression members, 2′-0″, but the ratio l-r for each angle between rivets shall be not more than ¾ of that for the whole member.

(e) **Minimum Edge Distance.**

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

Table I

Rivet Diameter, Inches	Minimum Edge Distance (Inches) for Punched Holes		
	In Sheared Edge	In Rolled Edge of Plates and Sections with Parallel Flanges	In Rolled Edge of Sections with Sloping Flanges
½	1	⅞	¾*
⅝	1⅛	1	⅞*
¾	1¼	1⅛	1*
⅞	1½	1¼	1⅛*
1	1¾	1½	1¼*
1⅛	2	1¾	1½*
1¼	2¼	2	1¾*

*May be decreased ⅛ inch when holes are near end of beam.

(f) **Minimum Edge Distance in Line of Stress.**

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

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

(g) **Maximum Edge Distance.**

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

Section 19. Plate Girders and Rolled Beams

(a) **Proportioning.**

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

(b) **Web.**

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

(c) **Flanges.**

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

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

The total cross-sectional area of cover plates shall not exceed 70 per cent of the total flange area.

(d) **Rivets.**

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

(e) **Stiffeners.**

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

If h/t is equal to or greater than 70, intermediate stiffeners shall be required at all points where h/t exceeds $\frac{8{,}000}{\sqrt{v}}$, in which

h = the clear depth between flanges, in inches
t = the thickness of the web, in inches
v = greatest unit shear in panel, in pounds per square inch under any condition of complete or partial loading.

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

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

d = the clear distance between stiffeners, in inches.

Intermediate stiffeners may be crimped over the flange angles.

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

(f) **Splices.**

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

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

(g) **Lateral Forces.**

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

(h) **Web Crippling of Beams.**

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

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

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

R = concentrated interior load or end reactions, in pounds.
t = thickness of web, in inches.
N = length of bearing, in inches.
k = distance from outer face of flange to web toe of fillet, in inches.

Section 20. Tie Plates.

(a) Compression Members.

The open sides of compression members shall be provided with lacing having tie plates at each end, and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In main members carrying calculated stresses the end tie plates shall have a length of not less than the distance between the lines of rivets connecting them to the segments of the member, and intermediate ones of not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between the lines of rivets connecting them to the segments of the members, and the rivet pitch shall be not more than four diameters. Tie plates shall be connected to each segment by at least three rivets.

(b) Tension Members.

Tie plates shall be used to secure the parts of tension members composed of shapes. They shall have a length not less than two-thirds of the length specified for tie plates in compression members. The thickness shall be not less than one-fiftieth of the distance between the lines of rivets connecting them to the segments of the member and they shall be connected to each segment by at least three rivets.

Section 21. Lacing.

(a) Spacing.

Lacing bars of compression members shall be so spaced that the ratio l/r of the flange included between their connections shall be not over ¾ of that of the member as a whole.

(b) Proportioning.

Lacing bars shall be proportioned to resist a shearing stress normal to the axis of the member equal to two per cent of the total compressive stress in the member. In determining the section required the compression formula shall be used, l being taken as the length of the bar between the outside rivets connecting it to the segment for single lacing and 70 per cent of that distance for double lacing. The ratio l/r shall not exceed 140 for single lacing nor 200 for double lacing.

(c) Minimum Proportions.

The thickness of lacing bars shall be not less than one-fortieth for single lacing, and one-sixtieth for double lacing, of the distance between end rivets; their minimum width shall be three times the diameter of the rivets connecting them to the segments.

(d) Inclination.

The inclination of lacing bars to the axis of the members shall preferably be not less than 45 degrees for double lacing and 60 degrees for single lacing. When the distance between the rivet lines in the flanges is more than 15 inches, the lacing shall be double and riveted at the intersection if bars are used, or else shall be made of angles.

Section 22. Adjustable Members.

(a) Initial Stress.

The total initial stress in any adjustable member shall be assumed as not less than 5,000 pounds.

Section 23. Column Bases.

(a) Loads.

Proper provision shall be made to transfer the column loads, and moments if any, to the footings and foundations.

(b) Alignment.

Column bases shall be set level and to correct elevation with full bearing on the masonry.

(c) Finishing.

Column bases shall be finished to accord with the following requirements:

1. Rolled steel bearing plates, 2" or less in thickness, may be used without planing, provided a satisfactory contact bearing is obtained; rolled steel bearing plates, over 2", but 4" or less in thickness, may be straightened

by pressing (planed on all bearing surfaces if presses are not available) to obtain a satisfactory contact bearing; rolled steel bearing plates, over 4" in thickness, shall be planed on all bearing surfaces (except as noted under 3).

2. Column bases other than rolled steel bearing plates shall be planed on all bearing surfaces (except as noted under 3).
3. The bottom surfaces of bearing plates and column bases which rest on masonry foundations and are grouted to insure full bearing contact need not be planed.

Section 24. Anchor Bolts.

(a) Anchor Bolts.

Anchor bolts shall be of sufficient size and number to develop the computed stress.

PART VI. FABRICATION

Section 25. Workmanship.

(a) General.

All workmanship shall be equal to the best practice in modern structural shops.

(b) Straightening.

All material shall be clean and straight. If straightening or flattening is necessary, it shall be done by a process and in a manner that will not injure the material. Sharp kinks or bends shall be cause for rejection.

(c) Heating.

Rolled sections, except for minor details, shall preferably not be heated or, if heated, shall be annealed, but this restriction does not apply to gas cutting, Section 25 (1).

(d) Holes.

Holes for rivets or unfinished bolts shall be 1/16 inch larger than the nominal diameter of the rivet or bolt. If the thickness of the material is not greater than the nominal diameter of the rivet or bolt plus 1/8 inch, the holes may be punched. If the thickness of the material is greater than the nominal diameter of the rivet or bolt plus 1/8 inch, the holes shall be either drilled from the solid, or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be 1/16 inch smaller than the nominal diameter of the rivet or bolt.

Holes for turned bolts shall be 1/50 inch larger than the external diameter of the bolt. If the bolts are to be inserted in the shop, the holes may be either drilled from the solid, or sub-punched and reamed. If the bolts are to be inserted in the field, the holes shall be sub-punched in the shop and reamed in the field. All drilling or reaming for turned bolts shall be done after the parts to be connected are assembled.

Drifting to enlarge unfair holes shall not be permitted. Holes that must be enlarged to admit the rivets shall be reamed. Poor matching of holes shall be cause for rejection.

(e) Planing.

Planing or finishing of sheared plates or shapes will not be required unless specifically called for on the drawings.

(f) Assembling.

All parts of riveted members shall be well pinned or bolted and rigidly held together while riveting. Drifting done during assembling shall not distort the metal or enlarge the holes.

(g) Riveting.

All rivets are to be power-driven hot. Rivets driven by pneumatically or electrically operated hammers are considered power-driven. Standard rivet heads shall be of approximately hemispherical shape and of uniform size throughout the work for the same size rivet, full, neatly finished, and concentric with the holes. Rivets, after driving, shall be tight, completely filling the holes, and with heads in full contact with the surface. Rivets shall be heated uniformly to a temperature not exceeding 1950° F.; they shall not be driven after their temperature is below 1000° F.

Loose, burned, or otherwise defective rivets shall be replaced.

(h) **Finishing.**

Compression joints depending upon contact bearing shall have the bearing surfaces truly machined to a common plane after the members are riveted. All other joints shall be cut straight.

(i) **Lacing Bars.**

The ends of lacing bars shall be neat and free from burrs.

(j) **Tolerances.**

Finished members shall be true to line and free from twists, bends, and open joints.

Compression members may have a lateral variation not greater than 1/1000 of the axial length between points which are to be laterally supported.

A variation of 1/16 inch is permissible in the overall length of members with both ends milled.

Members without milled ends which are to be framed to other steel parts of the structure may have a variation from the detailed length not greater than 1/16 inch for members 30 feet or less in length, and not greater than 1/8 inch for members over 30 feet in length.

(k) **Castings.**

All steel castings shall be annealed.

(l) **Gas Cutting.**

The use of a cutting torch is permissible if the metal being cut is not carrying stresses during the operation. To determine the effective width of members so cut, 1/8 inch shall be deducted from each gas cut edge. The radius of re-entrant gas cut fillets shall be as large as practicable, but never less than 1 inch.

Section 26. Shop Painting.

(a) **Shop Coat.**

Before leaving the shop, all steel work shall be thoroughly cleaned, by effective means, of all loose mill scale, rust and foreign matter. Except where encased in concrete, all steel work shall be given one coat of approved metal protection, applied thoroughly and evenly and well worked into the joints and other open spaces. All paint shall be applied to dry surfaces.

(b) **Inaccessible Parts.**

Parts inaccessible after assembly shall be given two coats of shop paint, preferably of different colors.

(c) **Contact Surfaces.**

Contact surfaces shall be cleaned, by effective means, before assembly, but not painted.

(d) **Finished Surfaces.**

Machine finished surfaces shall be protected against corrosion by a suitable coating.

PART VII. ERECTION

Section 27. Erection.

(a) **Bracing.**

The frame of all steel skeleton buildings shall be carried up true and plumb, and temporary bracing shall be introduced wherever necessary to take care of all loads to which the structure may be subjected, including erection equipment, and the operation of same. Such bracing shall be left in place as long as may be required for safety.

(b) **Bolting Up.**

As erection progresses the work shall be securely bolted up to take care of all dead load, wind and erection stresses.

(c) **Erection Stresses.**

Wherever piles of material, erection equipment or other loads are carried during erection, proper provision shall be made to take care of stresses resulting from the same.

(d) **Alignment.**

No riveting shall be done until the structure has been properly aligned.

(e) **Riveting.**

Rivets driven in the field shall be heated and driven with the same care as those driven in the shop.

(f) **Turned Bolts.**

Holes for turned bolts to be inserted in the field shall be reamed in the field as specified in Section 25 (d).

(g) **Field Painting.**

All field rivets and bolts, also all serious abrasions to the shop coat, shall be spot painted with the material used for the shop coat, or an equivalent, and all mud and other firmly attached and objectionable foreign materials shall be removed, before general field painting.

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

PART VIII. INSPECTION

Section 28. Inspection.

(a) **General.**

Material and workmanship at all times shall be subject to the inspection of experienced engineers representing the purchaser.

(b) **Cooperation.**

All inspection as far as possible shall be made at the place of manufacture, and the Contractor or Manufacturer shall cooperate with the Inspector, permitting access for inspection to all places where work is being done.

(c) **Rejections.**

Material or workmanship not conforming to the provisions of this Specification may be rejected at any time defects are found during the progress of the work.

CODE FOR† FUSION WELDING AND GAS CUTTING IN BUILDING CONSTRUCTION PART A—STRUCTURAL STEEL

PREFACE TO 1937 EDITION

Since the Code was first issued in 1928, revisions have been made from time to time which have reflected advances in the field of structural fusion welding. The present 1937 edition incorporates the following important revisions:

1. While retaining unchanged the existing unit stresses for welds made with filler metal of Grades 20, 30 or 40 (bare or lightly coated electrodes and mild steel gas welding rods), a new set of unit stresses for welds in shear and tension are added which provide for an increase of 20 per cent when made with filler metal of Grades 2, 4, 10 or 15 (heavily coated electrodes or high test gas welding rods). These increased unit stresses reflect the higher tensile strength, superior ductility, and resistance to fatigue afforded by these grades of filler metal.
2. Qualification strength requirements for operators of welding equipment using these higher grades of filler metal are correspondingly increased 20 per cent.
3. The current specifications relating to filler metal are included.

The following is the personnel of the Committee on Building Codes as constituted October 18, 1937: G. D. Fish, Chairman, Consulting Engineer; Milton Male, Secretary, United States Steel Corporation; J. L. Anderson, Air Reduction Company; A. G. Bissell, Bureau of Construction & Repair, U. S. Navy; C. T. Bissell, National Board of Fire Underwriters; E. A. Doyle, Linde Air Products Company; F. Eder, R. W. Hunt Company; J. L. Edwards, H. G. Balcom Associates; A. R. Ellis, Pittsburgh Testing Laboratory; F. H. Frankland, American Institute of Steel Construction; W. S. Hoar, Bethlehem Steel Company; G. A. Hughes, Truscon Steel Company; Jonathan Jones, Bethlehem Steel Co. Inc.; E. F. Kenney, Bethlehem Steel Company; Martin P. Korn, Consulting Engineer; J. C. Lincoln, The Lincoln Electric Company; C. W. Obert, Union Carbide & Carbon Research Laboratories; J. W. Owens, Fairbanks-Morse Company; H. M. Priest, Railroad Research Bureau; E. W. P. Smith, Lincoln Electric Company; F. D. Sturges, Jones & Laughlin Steel Corporation; Andrew Vogel, General Electric Company; E. Vom Steeg, Jr., General Electric Company; Aubrey Weymouth, Post & McCord, Inc.

(Signed) P. G. Lang, President,
American Welding Society

SECTION 1—GENERAL APPLICATION

1. Fusion welding may be substituted for or used in combination with riveting· bolting or other connecting means specified in the Building Code,** for connecting to one another or assembling the component parts of steel beams, girders, lintels, trusses, columns and other structural steel members of buildings, or for connecting steel to wrought-iron members of existing buildings, provided that such work be designed and executed in accordance with Sections 3 to 7, inclusive, of this Code.

2. Gas cutting may be substituted for shearing, sawing or other cutting means, in fabricating or altering structural steel or wrought iron members of buildings, subject to the limitations of Section 8 of this Code.

SECTION 2—DEFINITIONS

Definitions published by the American Welding Society apply to welding terms used in this Code, subject to special restrictions in the use of some of the terms. A few definitions are reproduced herein, with the modifications required in some of them to suit the uses of this Code.

† The Welding Society recommends that in the future, wherever the term "Fusion Welding" occurs, the more accurate term "Non-Pressure Welding" be adopted.

** *Building Code*, wherever the expression occurs in this code, refers to building law or specifications or other construction regulations in conjunction with which this code is applied.

1. *Fusion Welding.*—The process of joining metal parts in the molten (or molten and vapor) state, without application of mechanical pressure or blows. Under this code, fusion welding is restricted to the electric-arc and gas-welding processes.

2. *Root.*—The zone at the bottom (or base) of the cross-sectional space provided to contain a fusion weld.

3. *Throat.*—The minimum thickness of a weld along a straight line passing through the root.

4. *Throat Dimension.*—The thickness of throat assumed for purposes of design. Under this code the throat dimension of a *fillet weld* is the altitude from the root to the opposite side of the largest isosceles triangle which can be constructed within the cross section of the weld, the equal legs lying in the fused faces. In the case of a *butt weld*, the throat dimension is the thickness of the thinner of the parts joined.

5. *Fillet Weld.*—A weld of approximately triangular cross section, whose throat lies in a plane disposed (inclined) approximately 45 degrees with respect to the surfaces of the parts joined. The size of a fillet weld shall be expressed as the dimensions of the equal legs of the isosceles triangle described in paragraph 4.

6. *Butt Weld.*—A weld whose throat lies in a plane disposed approximately 90 degrees with respect to the surfaces of at least one of the parts joined. The size of a butt weld shall be expressed as the throat dimension.

7. *Weld Length.*—The unbroken over-all length of the full cross section of the weld exclusive of the length of any craters. Under this code the length of the full cross section is termed the *effective length*, and shall be used in all specifications, calculations, and drawings. In determining the effective length of a fillet weld, ¼ inch shall be deducted from the over-all length of the weld as an allowance for the rounded ends and the crater.

8. *Weld Dimensions.*—The dimensions of a weld shall be expressed in terms of its size (Pars. 5 and 6) and length (Par. 7).

9. *Gas Cutting.*—The process of severing ferrous metals by means of the chemical behavior of oxygen, in the presence of ferrous metals at high temperatures, to produce a kerf or cut of uniform width without burning the edges of the kerf or cut.

SECTION 3—MATERIALS

1. Structural steel to be welded under this code shall conform to Serial Designation A-9 (Steel for Buildings) of the current Standard Specifications of the American Society for Testing Materials.

2. Filler metal (arc-welding electrodes and gas-welding rods) shall conform to all general requirements, and to all special requirements for at least one of the grades of filler metal, provided by Serial Designation A205-37T (Iron and Steel Filler Metal), as amended to date, issued jointly by the American Society for Testing Materials and the AMERICAN WELDING SOCIETY. (See Appendix I.)

SECTION 4—PERMISSIBLE UNIT STRESSES

1. Welded joints shall be proportioned so that the stresses caused therein by loads specified in the Building Code shall not exceed the following values, expressed in kips (thousands of pounds) per square inch:

Kind of Stress	For Welds Made with Filler Metal of	
	Grade 2, 4, 10 or 15	Grade 20, 30 or 40
Shear on section through weld throat	13.6	11.3
Tension on section through weld throat	15.6	13.0
Compression (crushing) on section through throat of butt weld	18.0	18.0

Fiber stresses due to bending shall not exceed the values prescribed above for tension and compression, respectively. Stress in a fillet weld shall be considered as shear, for any direction of the applied stress.

2. In designing welded joints, adequate provision shall be made for bending stresses due to eccentricity, if any, in the disposition or sections of base-metal parts.

SECTION 5—DESIGN

1. The architect or engineer designing or supervising welded construction shall be experienced and skilled in such work.

2. *Plate Girders.*—Girders shall be proportioned either by their moments of inertia or by the flange-area method; in the latter method, when applied to a welded girder

having no holes in the web, one-sixth of the web area may be considered a part of the area of each flange. Stiffeners may be either angles or flat plates, welded to the web and flanges by intermittent or continuous fillet welds designed to transmit the stresses. Connection of component parts of flanges to each other and of flanges to web shall be by means of intermittent or continuous fillet welds designed to transmit the stresses.

3. *Beams.*—The use of continuous beams and girders, designed in accordance with accepted engineering principles, shall be permitted provided that their welded connections be designed to transmit the stresses involved in continuous beam construction. At the ends of non-continuous beams, the connections shall be designed to avoid excessive secondary stresses due to bending.

4. *Columns.*—Adjacent component parts of a built-up column shall be joined by two or more lines of continuous or intermittent welding in the direction of stress, such lines to be not further apart than 30 times the thickness of metal in the thinner part. In any line of intermittent welding, the clear distance between welds shall nowhere be more than 12 inches, or more than 16 times the thickness of metal in the thinner part, or more than one inch for each kip of designed strength in either adjoining weld. Fillet welding within a distance from either end of the column equal to the least width of column shall be continuous. Sufficient weld strength shall be provided to transmit the shearing stresses between joined parts caused by flexure due to long-column action, applied bending moments, and any beam reactions or other loads tending to compress the parts unequally.

5. *Butt Joints.*—One or both edges of base-metal parts to be joined by a butt weld shall be beveled if the throat dimension exceeds ¼ inch, except that beveling may be omitted if the weld is to transmit only compressive stress and if the space between the parts be made wide enough to permit sound welding and if the opening be backed up by a base-metal part or by sheet metal on the side further from the welding operator. For single and double-V joints, the angle of bevel of each part shall be not less than 30 degrees, and for single and double-bevel joints not less than 45 degrees. The clearance between parts at the root of a beveled joint shall be from 1/16 to 3/16 inch, except that when welding joints with heavily coated electrode, the clearance at the root may be made equal to the diameter of the electrode. Butt welds required to be beveled shall also be reinforced by making the thickness greater than the throat dimension defined in Sec. 2, Par. 4; the exposed face of a single-V or single-bevel weld shall be reinforced at least 20 per cent of the throat dimension and each exposed face of a double-V or double-bevel weld shall be reinforced at least 12½ per cent of the throat dimension. A butt weld intended to transmit tensile stress shall be made only when one of the parts to be joined is free or is flexible enough to permit contraction of the weld metal.

6. *Fillet Welds.*—The length of any fillet weld shall be made not less than four times the weld size or else the size of the weld shall be considered not to exceed ¼ of the length for purposes of calculating strength under this code.

7. *Welds in Slots or Holes.*—When welding inside a slot or hole in a plate or other part, in order to join same to an underlying part, fillet welding may be used along the wall or walls of the slot or hole, but the latter shall not be filled with weld metal or partially filled in such manner as to form a direct weld-metal connection between opposite walls, except that fillet welds along opposite walls may overlap each other for a distance of one-fourth of their size. No slot or hole shall be less in width or diameter than 1½ times its depth.

SECTION 6—WORKMANSHIP

1. Contractors for welded construction shall give evidence of their ability to handle the required work in a competent manner. They shall prove the ability of their welding operators to produce welded connections of the required strength by having them prepare strength-test specimens conforming to the specifications in Appendix II.

2. Welding, whether in shop or in field, shall be performed in accordance with the practices recommended in Appendix III.

3. Surfaces to be welded shall be free from loose mill scale, rust, paint or other foreign matter, except that a thin coat of linseed oil, if present, need not be removed. This clause applies not only to new structures but also to cases where new steel is to be welded to members of existing structures.

4. Component parts of built-up members shall be firmly secured together, by adequate clamps or other means, in preparation for assembly welding.

5. Where parts to be joined by a fillet weld are separated more than 1/16-inch, the excess above 1/16-inch in amount of separation shall be added to the weld size required by the design.

6. Structural steel shall not be painted on any areas where shop or field welding is later to be performed, except that a coat of linseed oil without pigment may be used for temporary protection. However, this clause shall not prohibit welding of steel

which has been painted, provided that the paint be first completely removed from the areas to be welded.

SECTION 7—ERECTION

1. In erecting a welded structure, adequate means shall be employed for temporarily fastening the members together and bracing the framework until the joints are welded; such means shall consist of erection bolts or other positive devices imparting sufficient strength and stiffness to resist all temporary weights and lateral forces, including wind. Owing to the small number of bolts ordinarily employed for joints which are to be welded, the temporary support of heavy girders carrying columns should receive special attention.

2. In tier-building erection, members shall not be erected more than four tiers or more than two-column lengths above any column connections yet unwelded. Light structures under 30 feet high may be erected without the use of temporary joint fastenings, provided that the members be welded together sufficiently for temporary security at the time they are erected.

SECTION 8—GAS CUTTING

1. Contractors shall give evidence of their ability to produce satisfactory gas cuts.

2. Gas-cut edges shall be smooth and regular in contour.

3. Gas cutting may be used in preparing base-metal parts for welding, provided that the edges so cut be thoroughly cleaned so as to expose a clean surface.

4. Gas cutting shall not be used to replace milling for bearing surfaces.

5. Gas cutting shall not be performed on a member while under stress, except that detail cutting to correct minor fabrication errors may be done if the removal of metal does not reduce the strength below design requirements.

6. Gas cutting of members to form openings or to remove portions of flanges shall not be done unless the design provides therefor.

. . . .

APPENDIX II—QUALIFICATION TEST FOR OPERATORS OF WELDING EQUIPMENT

1. The minimum qualification requirement shall be the welding of test specimens of the type shown in Fig. 2 and specified in Par. 2, each specimen to be tested to failure in a reliable tension-testing machine and to develop ultimate resistance not less than

80 kips if welded with filler metal of
Grade 2, 4, 10 or 15

67 kips if welded with filler metal of
Grade 20, 30 or 40

Two specimens shall be welded in the horizontal welding position, two in the vertical welding position, and (in case the operator is to perform overhead welding as part of his structural work) two in the overhead welding position.

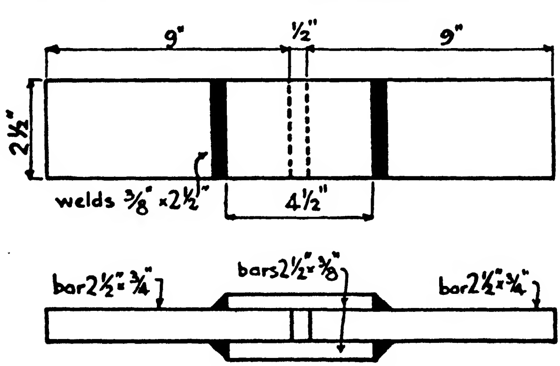


Fig 2

2. The form of test specimen shall be that which is shown in Fig. 2, consisting of four steel bars or plates connected by approximately equal fillet welds. This specimen is similar to that embodied in the previous edition of this Code, except that its width is reduced from 3 inches to 2½ inches. The bar steel shall conform to A. S. T. M. Specification A-9. The welding edges of the splice bars shall be cut flat and square and any burrs shall be removed. The bars shall be clamped securely together and tack welded in proper position with the splice bars centered on the main bars and the faces of the main bars in true planes, in preparation for the test welding. Welding shall be performed with the same process (arc or gas), the same welding procedure, and the same grade of electrode or welding rod to be used in the work for which the operator is being tested; the diameter of electrodes or rods shall be the largest to be used in the proposed work. Welds shall be made as nearly as possible to the designed dimensions, and in conformity with the practices set forth in Appendix III hereto; at every cross section the weld size in each direction shall be within 1/16-inch of the prescribed 3/4-inch.

3. The foregoing provisions are not intended to bar an operator from further trials in case of one or more failures, or to require an operator who has passed the tests to qualify anew for every job he may undertake. Requalification is advisable after any period of several months without practice, or on changing to an unfamiliar type or size of electrode or welding rod, or at any time if decrease of skill becomes apparent. Qualification for arc welding shall not constitute qualification for gas welding, and vice versa. Qualification for welding with bare electrodes shall not constitute qualification for welding with flux-coated electrodes, and vice versa.

CODE 2—FOR RESISTANCE WELDING OF STRUCTURAL STEEL IN BUILDING CONSTRUCTION

Formulated by American Welding Society Committee on Building Codes February 1935*

PREFACE

The increasing use in building construction of resistance welded steel members, such as joists, light trusses, stair stringers and similar parts, has developed standardized shop practices for their multiple production. Such building parts shop-fabricated in accordance with these practices have been used throughout the United States for a sufficient length of time to establish their safety in service.

The present code is based on the belief that, in building construction, the application of resistance welding should be restricted to those members which, by reason of their shop fabrication in multiple quantity, can be subjected to definite qualification tests. These tests aim to insure that each member shall develop full strength by requiring that the welded joints of one or more of each product, fabricated by any given setting of a particular welding machine, shall be tested to destruction, and that authoritative records of these tests shall be readily available. Such procedure, it is believed, will lessen the expense and delay of field testing by each local building inspector.

This Code for Resistance Welding of Structural Steel in Building Construction has been so arranged that it may be cited by, or embodied in, existing municipal or other Building Codes. The Committee is prepared to assist in the interpretation of the Code or its applications. Communications should be addressed to the American Welding Society, 33 West 39th Street, New York, N. Y.

Sub-Committee, Code for Resistance Welding of Structural Steel in Building Construction: Frank Burton, Chairman, Steel Joist Institute; W. A. Hoar, McClintic-Marshall Corp.; G. A. Hughes, Truscon Steel Company; H. A. Woofter, Swift Electric Welder Company.

Sub-Committee on Scope, Code for Resistance Welding of Structural Steel in Building Construction: J. L. Edwards, Chairman; H. G. Balcom, Consulting Engineer; Gilbert D. Fish, Consulting Engineer; F. P. McKibben, Consulting Engineer.

The following is the personnel of the Committee on Building Codes as constituted February 1, 1935: H. H. Moss, Chairman, Linde Air Products Company; Milton Male, Secretary, U. S. Steel Corporation; J. L. Anderson, Air Reduction Company; H. G. Balcom, Consulting Engineer; A. G. Bissell, Consulting Engineer; C. T. Bissell, National Board of Fire Underwriters; Frank Burton, Consulting Engineer; E. A. Doyle, Linde Air Products Company; F. Eder, R. W. Hunt Company; A. R. Ellis, Pittsburgh Testing Laboratory; G. D. Fish, Consulting Engineer; J. M. Frankland, Engineer; W. S. Hoar, McClintic-Marshall Corporation; G. A. Hughes, Truscon Steel Company; J. C. Lincoln, The Lincoln Electric Company; F. T. Llewellyn, U. S. Steel Corporation;

*Reprinted from May 1935 issue of Journal of American Welding Society.

F. P. McKibben, Consulting Engineer; J. W. Owens, Fairbanks, Morse & Company; H. M. Priest, Railroad Research Bureau; Andrew Vogel, General Electric Company; E. Vom Steeg, Jr., General Electric Company; C. C. Whittier, R. W. Hunt Company; H. A. Woofter, Swift Electric Welder Company.

(Signed) D. S. Jacobus, President,
American Welding Society

SECTION 1—APPLICATION

1. Resistance welding may be used for the multiple shop fabrication of truss-shaped or beam-shaped members for floors and roofs and for the shop fabrication of steel stringers and sheet steel parts of stairs and other similar members.

SECTION 2—DEFINITIONS

1. Pressure Welding—A process of welding metals in the highly plastic and/or fluid states by the aid of mechanical pressure.
2. Resistance Welding—A pressure welding process wherein the welding heat is obtained by passing an electric current between the contact areas to be welded.
3. Spot Welding—A resistance welding process wherein a weld is made in one or more spots by the localization of the electric current between contact points.
4. Projection Weld—A spot weld wherein projections to localize the heat are provided in one or both of the overlapping parts to be joined.
5. Resistance Butt Welding—A resistance welding process wherein a butt joint is employed.
6. Seam Welding—A resistance welding process wherein the weld is made linearly between and by means of two contact rollers or a contact roller and a contact bar.

SECTION 3—MATERIALS

1. The parts joined shall be steel conforming to the Standard Specifications of the American Society for Testing Materials, Serial Designation A9, or A15 Structural Grade, as amended to date.
2. Surfaces to be welded shall be free from rust, loose scale, dirt and paint, but pickling shall not be required.

SECTION 4—PERMISSIBLE UNIT STRESSES

1. Welds shall be proportioned so that the unit stresses therein shall not exceed 11,300 lb. per sq. in. in shear, or 13,000 lb. per sq. in. in tension. Stresses due to eccentricity of loading, if any, shall be combined with the primary stresses; and the combined unit stresses shall not exceed the values given above.
2. In plates or sheets joined by spot welding, the unit stress at any weld shall not exceed 11,300 lb. per sq. in. In computing this stress, the area used shall be the lesser value of either the actual area of the weld or the product of the thickness of the thinnest piece of metal being welded multiplied by the perimeter of the weld.

SECTION 5—DESIGN

1. Parts to be joined shall be designed to permit sufficient flow of plastic metal to produce proper contact and to extrude any foreign materials, and in such manner that the center lines of electrodes shall pass approximately through the center of the weld.

SECTION 6—MACHINES

1. Resistance welding machines shall be provided with automatic devices for regulating the total available energy and pressure. Machines shall produce uniform welds, uniform in physical properties, and strong enough to provide a factor of safety of at least three in respect to the working load of the weakest member connected.
2. Resistance welding machines should be qualified for the work which they are to produce by actually testing to destruction the welded joints of one or two of each product to be made, the machines to be adjusted until the destructive tests show the machine to be developing the strength factor of safety specified in paragraph 1. Destructive tests of individual joints shall be repeated at least once on every shift and also every time the combination of material being welded is changed. Destructive tests in which the welded joint fails outside the weld shall be accepted as showing the efficiency of the weld, in which case that fact shall be recorded rather than the actual load.

SECTION 7—OPERATORS

1. Operators shall be thoroughly experienced in the use of the welding machines employed. If the operators adjust the machines for making various types of weld they shall correspondingly adjust the machine for making the sample welds. If, for any reason, after the machine has been adjusted and qualified it is found necessary to make further adjustments, a new qualification test shall be made.

SECTION 8—TESTING

1. Contractors for electrical resistance welded parts of buildings shall satisfy the Superintendents of Buildings as to the adequacy and strength of such joints as are resistance welded. When tests are required they shall be performed in accordance with the provisions of Section 6.

A.S.T.M. STANDARD SPECIFICATIONS FOR STEEL FOR BUILDINGS

A.S.T.M. Designation: A-9-36

These specifications are issued under the fixed designation A-9; the final number indicates the year of original adoption as standard or, in the case of revision, the year of last revision.

Adopted, 1901; Revised, 1909, 1913, 1914, 1916, 1921, 1924, 1929, 1933, 1934, 1936.

Structural Rivet Steel.

1. Unless otherwise specified, the Standard Specifications for Structural Rivet Steel (A.S.T.M. Designation: A 141) of the American Society for Testing Materials, shall govern the purchase of rivets for use with steel purchased under these specifications.

Rolled Base Plates.

2. Rolled base plates over 2 in. in thickness for bearing purposes shall be open hearth or electric furnace steel containing 0.20 to 0.35 per cent carbon. The chemical composition shall also conform to the requirements specified in Section 4. A sufficient discard shall be made from each ingot to secure sound plates. Physical tests shall not be required for this material.

MANUFACTURE

Process.

3. (*a*) The steel, except as specified in Paragraph (*b*) shall be made by one or more of the following processes: open hearth, electric furnace or acid bessemer.

(*b*) Steel for plates and shapes over $\frac{7}{16}$ in. in thickness which are to be punched shall be made by either or both of the following processes: open hearth or electric furnace.

CHEMICAL PROPERTIES AND TESTS

Chemical Composition.

4. The steel shall conform to the following requirements as to chemical composition:

Phosphorus, max., per cent	Bessemer	0.10
	Open hearth or electric furnace	0.06
Copper, when copper steel is specified, min., per cent		0.20

Ladle Analyses.

5. (*a*) A carbon determination, and a copper determination when copper steel is specified shall be made of each melt of bessemer steel, and determinations for manganese, phosphorus and sulphur representing the average of the melts applied for each 8 hr. period.

(*b*) An analysis of each melt of open hearth or electric furnace steel shall be made to determine carbon, manganese, phosphorus and sulphur; also copper when copper steel is specified.

(*c*) The analyses prescribed in Paragraphs (*a*) and (*b*) shall be made by the manufacturer from test ingots taken during the pouring of each melt. The chemical composition thus determined shall be reported to the purchaser or his representative and the percentage of phosphorus, and also copper when copper steel is specified, shall conform to the requirements specified in Section 4.

Check Analyses.

6. Analyses may be made by the purchaser from finished material representing each melt. The phosphorus content thus determined shall not exceed that specified in Section 4 by more than 25 per cent.

PHYSICAL PROPERTIES AND TESTS

Tension Tests.

7. (*a*) The material, except as specified in Section 2 and Paragraph (*b*) of this section, shall conform to the following requirements as to tensile properties:

Tensile strength, lb. per sq. in.	60,000 to 72,000
Yield point, min., lb. per sq. in.	0.5 tens. str.
but in no case less than	33,000
Elongation in 8 in., min., per cent	$\frac{1{,}400{,}000^a}{\text{Tens. str.}}$
Elongation in 2 in., min., per cent	22^b

[a] See Section 8.

[b] When specimen shown in Fig. 2 is used; see Section 10 (*e*) and (*g*).

(*b*) Flat rolled steel $\frac{5}{16}$ in. and under in thickness, shapes less than 1 sq. in. in cross section and bars, other than flats, less than ½ in. in thickness or diameter need not be subjected to tension tests.

(*c*) The yield point shall be determined by the drop of the beam or halt in the gage of the testing machine.

Modifications in Elongation.

8. (*a*) For material over ¾ in. in thickness or diameter, a deduction from the percentage of elongation in 8 in. specified in Section 7 (*a*) of 0.25 per cent shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness or diameter above ¾ in., to a minimum of 16 per cent.

(*b*) For material under $\frac{5}{16}$ in. in thickness or diameter, a deduction from the percentage of elongation in 8 in. specified in Section 7 (*a*) of 1.25 per cent shall be made for each decrease of $\frac{1}{32}$ in. of the specified thickness or diameter below $\frac{5}{16}$ in

Bend Tests.

9. Bend test specimens shall stand being bent cold through 180 deg. without cracking on the outside of the bent portion, around a pin, the diameter of which shall have the following relation to the thickness of the specimen:

Thickness of Material	Ratio: $\frac{\text{Pin Diameter}}{\text{Thickness of Specimen}}$
Up to ¾ inch, inclusive	1
Over ¾ to 1 inch, inclusive	1½
Over 1 to 1½ inch, inclusive	2
Over 1½ to 2 inch, inclusive	2½
Over 2 inches	3

Test Specimens.

10. (*a*) Test specimens shall be prepared for testing from the material in its rolled or forged condition, except as specified in Paragraph (*b*).

(*b*) Test specimens for annealed material shall be prepared from the material as annealed for use, or from a short length of a full section from the same melt similarly treated.

(*c*) Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*e*), (*f*) and (*g*), shall be of the full thickness or section of material as rolled.

(*d*) Test specimens for plates, shapes and flats may be machined to the form and dimensions shown in Fig. 1, or with both edges parallel.

(*e*) Tension test specimens for material over 1½ in. in thickness or diameter, except pins and rollers, may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in., or they may conform to the dimensions shown in Fig. 2.

(*f*) Bend test specimens for material over 1½ in. in thickness or diameter, except pins and rollers, may be machined to a thickness or diameter of at least ¾ in. or to 1 by ½ in. in section.

(*g*) Tension test specimens for pins and rollers shall conform to the dimensions shown in Fig. 2, and bend test specimens shall be 1 by ½ in. in section.

(*h*) Test specimens for pins and rollers shall be taken so that the axis is 1 in. from the surface.

(*i*) The machined sides of rectangular bend test specimens may have the corners rounded to a radius not over $\frac{1}{16}$ in.

Number of Tests.

11. (*a*) Two tension and two bend tests shall be made from each melt, unless the finished material from a melt is less than 30 tons when one tension and one bend test will be sufficient. If however, material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and thinnest material rolled regardless of the weight represented.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 7 (*a*) and any part of the fracture is more than ¾ in. from the center of the gage length of a 2 in. specimen or is outside the middle third of the gage length of an 8 in. specimen, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

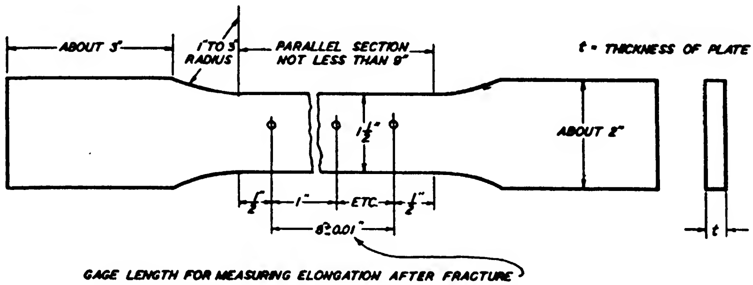


Fig. 1. Standard 8-in. Gage Length Test Specimen

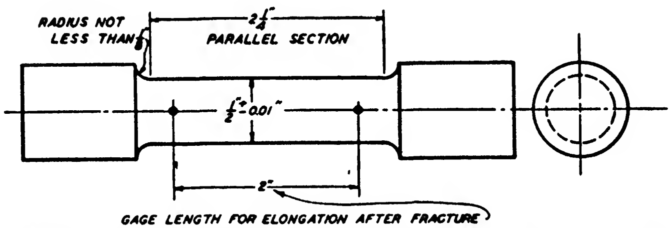


Fig. 2. Standard 2-in. Gage Length Tension Test Specimen. NOTE: The Gage Length, Parallel Section, and Fillets shall be as shown, but the ends may be of any shape to fit the holders of the Testing Machine in such a way that the Load shall be axial.

PERMISSIBLE VARIATIONS IN WEIGHT AND THICKNESS

Permissible Variations.

12. (*a*) One cubic inch of rolled steel is assumed to weigh 0.2833 lb. The cross sectional area or weight of each structural size shape and of each universal mill plate up to and including 36 in. in width and 2 in. in thickness shall not vary more than 2.5 per cent from the theoretical or specified amounts. The thickness or weights of rectangular sheared mill plates and of rectangular universal mill plates over 36 in. in width or 2 in. in thickness shall conform to the requirements of Paragraphs (*b*), (*c*) and (*d*).

(*b*) *Plates, When Ordered to Weight per Square Foot.*—The weight of each lot[1] of plates in each shipment shall not vary from the weight ordered more than the amounts given in Table I.

(*c*) *Plates, When Ordered to Thickness.*—The thickness of each plate shall not vary more than 0.01 in. under that ordered.

[1] The term "lot" as applied to Table I means all of the plates of each group width and group weight. As applied to Table II, it means all the plates of each group width and group thickness.

The overweight in each lot[1] of plates in each shipment shall not exceed the amount given in Table II.

(*d*) *Plates Over 2 In. in Thickness*—All plates over 2 in. in thickness shall conform to the permissible variations over ordered thickness given in Table III.

TABLE I. Permissible Variations of Rectangular Plates Ordered to Weight

Ordered Weight, Pounds per Square Foot	Permissible Variations in Average Weight per Square Foot of Plates for Widths Given, Expressed in Percentages of Ordered Weights																	
	Under 48"		48" to 60" excl.		60" to 72" excl.		72" to 84" excl.		84" to 96" excl.		96" to 108" excl.		108" to 120" excl.		120" to 132" excl.		132" or Over	
	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under
Under 5 ...	5	3	5 5	3	6	3	7	3										
5 to 7.5 excl.	4 5	3	5	3	5 5	3	6	3										
7.5 to 10 excl.	4	3	4 5	3	5	3	5 5	3	6	3	7	3	8	3				
10 to 12.5 excl.	3 5	2 5	4	3	4 5	3	5	3	5 5	3	6	3	7	3	8	3	9	3
12.5 to 15 excl.	3	2 5	3 5	2 5	4	3	4 5	3	5	3	5 5	3	6	3	7	3	8	3
15 to 17.5 excl.	2 5	2 5	3	2 5	3 5	2 5	4	3	4 5	3	5	3	5.5	3	6	3	7	3
17.5 to 20 excl.	2 5	2	2 5	2 5	3	2 5	3 5	2 5	4	3	4 5	3	5	3	5 5	3	6	3
20 to 25 excl.....	2	2	2 5	2	2 5	2 5	3	2 5	3 5	2 5	4	3	4.5	3	5	3	5 5	3
25 to 30 excl.. .	2	2	2	2	2 5	2	2 5	2 5	3	2 5	3 5	3	4	3	4 5	3	5	3
30 to 40 excl. .	2	2	2	2	2	2	2 5	2	2 5	2 5	3	2 5	3.5	3	4	3	4 5	3
40 to 81.6, incl.	2	2	2	2	2	2	2	2	2 5	2	2 5	2 5	3	2 5	3.5	3	4	3

Note: The weight per square foot of individual plates shall not vary from the ordered weight by more than one and one-third the amount given in this table.

TABLE II. Permissible Overweights of Rectangular Plates Ordered to Thickness

Ordered Thickness, Inches	Permissible Excess in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Nominal Weights								
	Under 48"	48" to 60" excl.	60" to 72" excl.	72" to 84" excl.	84" to 96" excl.	96" to 108" excl.	108" to 120" excl.	120" to 132" excl.	132" or Over
Under 1/8	9	10	12	14					
1/8 to 3/16 excl....	8	9	10	12					
3/16 to 1/4 excl....	7	8	9	10	12				
1/4 to 5/16 excl....	6	7	8	9	10	12	14	16	19
5/16 to 3/8 excl....	5	6	7	8	9	10	12	14	17
3/8 to 7/16 excl. ..	4 5	5	6	7	8	9	10	12	15
7/16 to 1/2 excl....	4	4.5	5	6	7	8	9	10	13
1/2 to 5/8 excl...	3 5	4	4.5	5	6	7	8	9	11
5/8 to 3/4 excl....	3	3 5	4	4 5	5	6	7	8	9
3/4 to 1 excl.	2 5	3	3 5	4	4 5	5	6	7	8
1 to 2, incl.	2 5	2 5	3	3 5	4	4 5	5	6	7

Note: The weight of individual plates ordered to thickness shall not exceed the nominal weight by more than one and one-third the amount given in this table.

TABLE III. Permissible Variations over Ordered Thickness of Plates Over 2 inches in Thickness

Note: Each plate shall not vary more than 0.01 in. under the ordered thickness.

Ordered Thickness, Inches	Permissible Excess in Thickness, Inches. Width, Inches					
	Up to 36 excl.	36 to 60 excl.	60 to 84 excl.	84 to 120 excl.	120 to 132 excl.	132 to 144 excl.
Over 2 to 3, excl.	0 062 (1/16)	0.094 (3/32)	0 109 (7/64)	0 125 (1/8)	0 125 (1/8)	0 141 (9/64)
3 to 4, excl.	0.078 (5/64)	0.094 (3/32)	0 109 (7/64)	0 125 (1/8)	0.125 (1/8)	0 141 (9/64)
4 to 6, excl.	0.094 (3/32)	0 125 (1/8)	0 141 (9/64)	0.141 (9/64)	0.156 (5/32)	0 172 (11/64)
6 to 8, excl.	0.109 (7/64)	0.125 (1/8)	0 156 (5/32)	0 156 (5/32)	0.172 (11/64)	
8 to 10, excl.		0 172 (11/64)	0 187 (3/16)	0 187 (3/16)		
10 to 12, excl.		0.187 (3/16)	0.234 (15/64)	0 234 (15/64)		
12 to 15, excl.		0.219 (7/32)	0 250 (1/4)			

[1] The term "lot" as applied to Table I means all of the plates of each group width and group weight. As applied to Table II, it means all the plates of each group width and group thickness.

FINISH

13. (*a*) The finished material shall be free from injurious defects and shall have a workmanlike finish.

(*b*) Surface imperfections that do not affect the full utility of the pieces, shall not be considered as injurious defects in structural shapes, 3/8 in. or more in thickness. Such pieces may be processed by the following methods in order to give them a workmanlike finish:

(1) When the surface imperfections are less than 1/16 in. in depth, they may be removed by grinding.

(2) When the surface imperfections are 1/16 in. or more in depth, the pieces may be subjected to chipping and welding under limiting conditions as follows:

The cross sectional area of any piece shall not be reduced more than 1.5 per cent at any point, nor shall the total area of the chipped surface of any piece exceed 2 per cent of the total surface area of that piece.

After any imperfection has been completely removed, the maximum depth of depression shall not exceed the following:

Thickness of Material, Inches	Depth of Depression, Maximum, Inches
3/8 up to 1/2	1/16
1/2 up to 1	1/8
1 up to 1 1/4	3/16
1 1/4 up to 2 1/4	1/4
2 1/4 up to 3 1/2	3/8

An experienced mill inspector shall inspect the work after the chipping operation to see that the defects have been completely removed and that the limitations specified above have not been exceeded. The inspector representing the purchaser shall be given full opportunity to make this same inspection. All welding shall be done by qualified welders using suitable coated welding rods. The welds shall be sound; the weld metal being thoroughly fused on all surfaces and edges without undercutting or overlap. Weld metal shall project at least 1/16 in. above the rolled surface after welding, and the projecting metal shall be removed by chipping or grinding to make it flush with the rolled surface and produce a workmanlike finish.

MARKING

14. The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that lattice bars and other small sections shall, when loaded for shipment, be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping, if practicable, on each test specimen.

INSPECTION AND REJECTION

Inspection.

15. The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, without charge, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

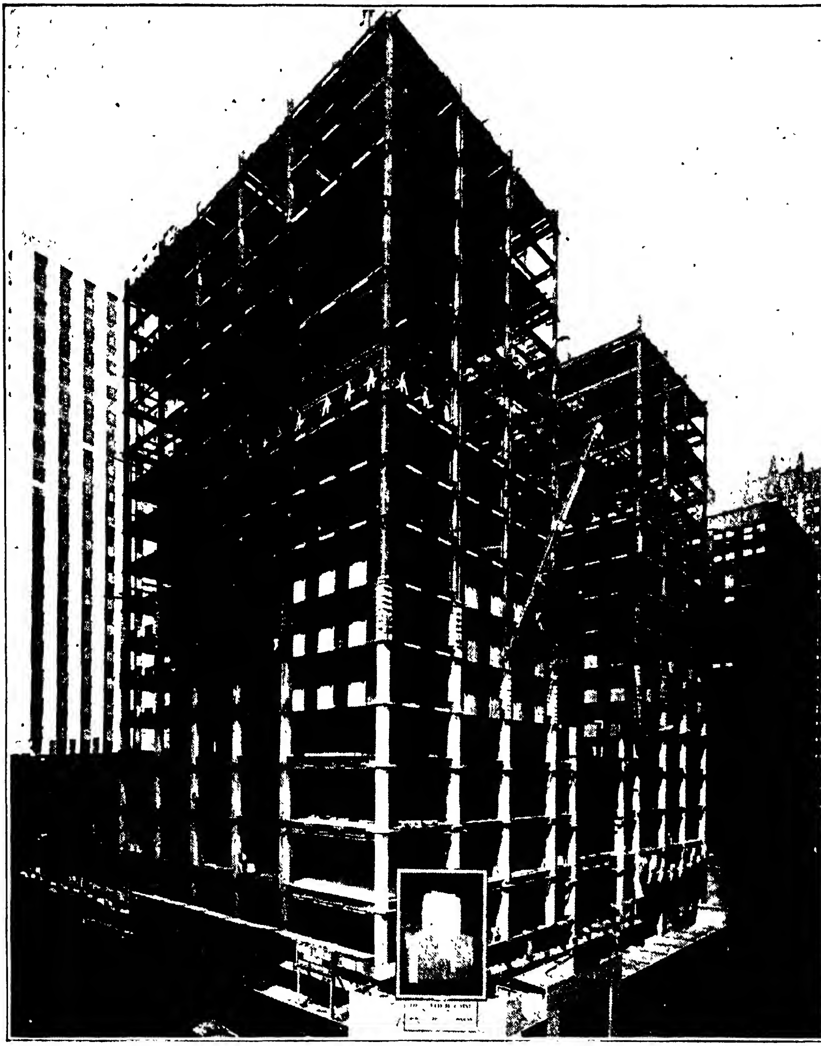
Rejection.

16. (*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 6 shall be reported within five working days from the receipt of samples.

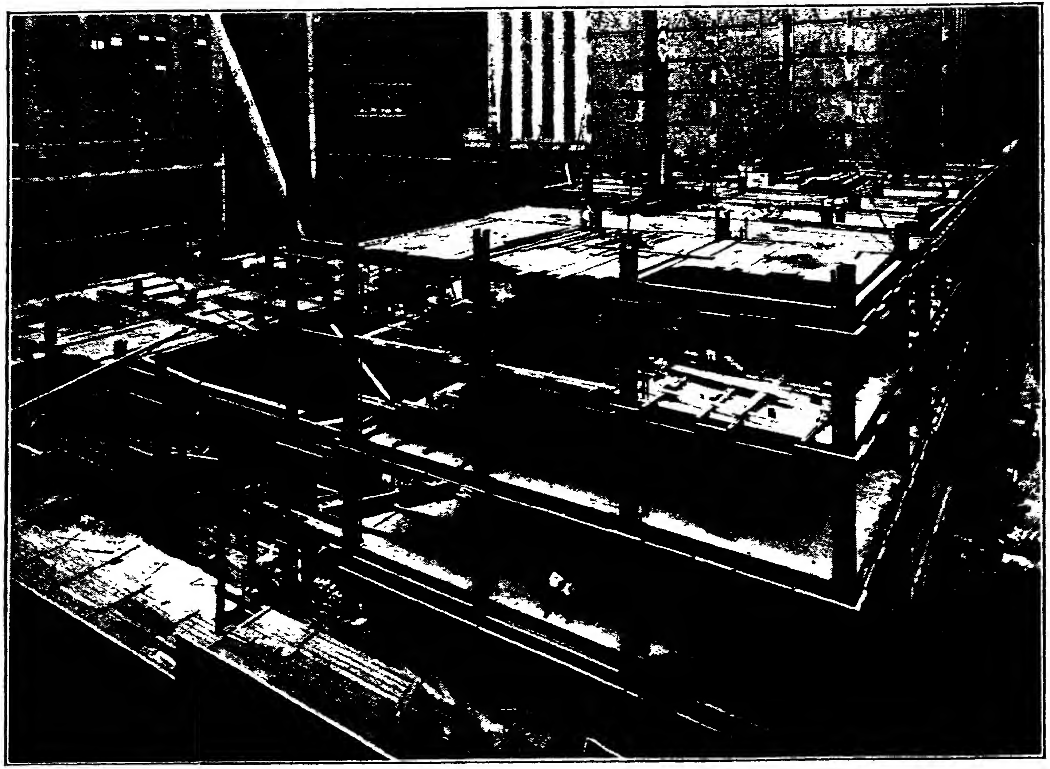
(*b*) Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

Rehearing.

17. Samples tested in accordance with Section 6, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.



THE FIELD BUILDING, CHICAGO
Courtesy of Graham, Anderson, Probst and White, Architects



THE FIELD BUILDING, CHICAGO
Note the Shelf Angle Supports on the Spandrel Beams
Courtesy of Graham, Anderson, Probst and White, Architects

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