PRACTICAL STEEL CONSTRUCTION

Dealing with all phases in the construction of our modern steel buildings : : : :

With Illustrations, Drawings and valuable Tables.

Compiled, Revised and Edited

By

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Steel Construction

INTRODUCTION

The introduction of steel in construction has become universal in this country during the last decade. Its extended use is evidently due, not only to its merits as a constructive material and the exceedingly low price at which it may be obtained, but also to the rapidity with which it may be assembled in the field. Without such a material the skeleton construction adopted for high office buildings and similar structures, which are being erected so generally in the principal American cities, would be impracticable; bridges would become more cumbersome and unsightly affairs, and the successful construction of graceful and symmetrical roof trusses, and arched ribs of great span would be almost impossible.

Since structural steel has become such an important factor in all building operations, its proper adaptation in the design of structures should be thoroughly understood by architects, contractors, and others engaged in the building trades. Not only will such an understanding enable them to design the safest building with the least expenditure of material, and consequently capital to their client, but it will enable them to erect such structures as to preclude the possible loss of life by the collapse of unstable structures, caused by their improper design and through ignorance of good constructive details on the part of their designer.

The intention of this essay is to set forth the usual practice adopted in America by designers of steel construction, and to treat and describe in detail such features of construction, as enter into the design of modern offices and buildings, where the elements of strength depend upon a framework of steel.

In this treatise the wording usually employed in writing specifications has been adhered to as closely as practicable; for by the adoption of such a style the facts can be expressed more concisely.

DEAD LOAD

The dead load shall consist of the static load due to the weight of all the materials entering into the construction of the building, and shall include in its estimate the weight of permanent machinery, water tanks, and large safes or vaults built as a part of the structure. The weight of the fireproof construction shall be calculated for in each case. The dead load of the floors shall consist of the weight of the arches, concrete filling, flooring, plaster, ceiling, and steel construction. Where partitions are permanent the floor beams immediately beneath them shall be calculated to carry the weight of the partitions in addition to the regular floor load. Where the partitions are not fixed—that is, are liable to be changed from time to time as circumstances may arise—a load of 20 lb. per square foot of floor area shall be added to the weight of the floor construction, in order to cover the possibility of an imposed load, at any point on the floor, due to the weight of the partitions.

In figuring the dead load due to the fireproof floor construction, the weight of the several characters of arches used shall be obtained from the catalogues of their respective manufactures. The weight of the cinder concrete filling shall be assumed at 72 lb. per cubic foot. The finished floor line shall be at least 3 in. above the top of the steel construction, and the finished ceiling line 2 in. below the under side of the beams, brick arches excepted.

The general construction shall be first decided upon, and the approximate dimensions of the members composing the frame assumed; from which their weights are calculated and the dead load estimated. After the structure has been designed, and the dimensions of the beams, columns, and girders, together with the rolled sections comprising them have been definitely decided upon, the dead load as previously figured shall be accurately gone over and checked. Should the beams, columns, or girders be found inadequate, they must be strengthened by increasing the weight of the rolled sections composing them. This shall be accomplished, wherever possible, by thickening the rolled section rather than by changing their principal dimensions.

In designing the footings and foundation piers the entire dead load shall be figured; great care being exercised to see that the load per unit of surface for like soils is the same throughout the entire foundation area, so that any tendency towards unequal settlement will be avoided.

LIVE LOAD

The live load shall comprise the transitory loads upon the floors of the building such as people, merchandise, small safes, movable machinery, and varying loads of any character whatsoever, including the snow load upon the roof. This load shall be added to the dead load in obtaining the total or maximum load upon floor or roof.

The maximum live loads for the several characters of buildings shall be, for each square foot of floor area as enumerated in the following table, based upon the subscribed data:—

For City Dwellings, 70 lb.
For County Dwellings, 40 lb.
For Theatres, Churches, and School-rooms, 80 lb.
For Office Buildings, 50 to 70 lb.
For Assembly Halls, Ball-rooms, and Drill Halls, 120 to 150 lb.
For Factories, Light Work, 150 lb.
For Factories, Heavy Work, 200 to 450 lb.
For Warehouses, etc., from 250 lb. up.

The weight of a crowd of people seldom attains more than 80 lb. per square foot of floor surface, though when densely packed the weight may be as high as 120 lb. for like area. City houses are liable to other uses than that of a dwelling, hence a somewhat heavier load shall be figured on than would ordinarily obtain in dwellings.

All buildings containing furniture fixed to the floor, such as the seats in theatres, pews in churches, and the desks or benches in schools, cannot be densely crowded, and consequently a somewhat lighter load than 120 lb. per square foot of floor surface shall be assumed as the live load in such buildings. In buildings occupied as offices dense crowds seldom collect except on the lower floors devoted to stores, either wholesale or retail, banking purposes, show-rooms, or business exchanges. Ascertained data, obtained by investigating 210 Boston offices, shows the average live load for the entire number of offices to be approximately 17 lb. per square foot of floor surface, while the greatest live load in any one office was 40 lb. per square foot; the average of the ten highest being 33 lb. per square foot.

In so much as the live load is liable to be concentrated, to attempt to use an average load would be dangerous. Therefore the maximum live load likely to occur shall be used in figuring the loads upon the floors of office buildings. This load shall be at least 50 lb. per square foot of floor surface, for all floors used only as general offices. For lower floors, used as places of exchange or mercantile business, liable at times to be densely crowded, at least 120 lb. per square foot shall be estimated on.

In designing large buildings of the skeleton construction type where great sums of money are invested, provision shall be made for all possibilities of extreme, either present or future. Since the likelihood of the full live load occurring on all of the floors of an office building, hotel, or apartment house, is remote, the footings and foundation piers shall be proportioned to sustain only a portion of the live load, and can in some cases be neglected entirely. Where this is done, however, great care must be exercised in ascertaining the bearing value of the foundation soil, and the allowable bearing value must be taken well within the safe limits.

In designing buildings many stories in height, the floor beams supporting individual floors must be designed to sustain the full live load. All girders supporting such beams shall be designed to sustain at least 90 per cent of the live floor load, and those supporting two or more floors from 75 to 85 per cent of the live load upon its dependent floors, according to the number and the possibilities of loading. All columns must be designed to sustain the reaction of all girders secured to them and depending upon them for support.

In this character of building, especially where its height exceeds six to eight stories, the live load accumulated at each floor, and transmitted through the tiers of columns, may be reduced a reasonable percentage from roof to foundation. This percentage shall be carefully considered in each case, the percentage to be deducted at each floor depending upon the height of the building and the possibilities as to the full live load being realized. The following shall be an example of the percentage of reduction existing from the roof to the basement of a thirteen story office building, erected according to good practice and economical construction. The live load upon the floor beams, from the second to the thirteenth floor, inclusive, was taken at 70 lb. per square foot. An additional load of 20 lb. per square foot was added to the dead load to take care of the possibility of changes in the partitions which was likely to occur at any time, and the location of which might be anywhere upon the plan of the floors.

	Live Load on Floor in lbs. per square foot of Floor Surface.	Live Load on Columns from Floors above in lbs. per square foot of Floor Surface.	Total Load on Columns in lbs. per square foot of Floor Surface.
Roof	40	—	—
13th Floor	50	40	40
12th "	—	45	85
11th "	—	41	126
10th "	—	35	161
9th "	—	31	192
8th "	—	25	217
7th "	—	21	238
6th "	—	15	253
5th "	—	11	264
4th "	—	5	269
3rd "	—	1	270
2nd "	—	0	270
1st "	125	0	270
Basement	—	50	320

The live load in warehouses depends upon the merchandise to be stored. The weight of such merchandise shall be carefully ascertained for each particular case in hand, and the structure designed for the maximum live load. Where standard articles of merchandise are stored the following table, which gives measurements, floor space occupied, and weights of one case, box, cask, crate, barrel, bale, or bag, etc., together with the load in pounds per square foot of floor surface, shall be used:—

Material.	Measurements. Floor Space Occupied. Sq. Ft.	Cu. Ft.	Weight. Lbs. per Cu. Ft.	Lbs. per Sq. Ft.
Cassimers, woollen, in cases	10.5	28.0	20	52
Cement, American, in barrels	3.8	5.5	59	86
Cement, English, in barrels	3.8	5.5	73	105
Cheese	—	—	30	—
Corn, in bags	3.6	3.6	31	31
Cotton, in bales	8.1	44.2	12	64
Cotton, extra compressed, in bales	1.25	3.13	40	100
Crockery, in casks	13.4	42.5	14	52
Crockery, in crates	9.9	36.6	40	162
Dress goods, woollen, in cases	5.5	22.0	21	84
Flannels, heavy woollen, in cases	7.1	15.2	22	46
Flour, in barrels	4.1	5.4	40	53
Glass, in boxes	—	—	60	—
Hay, in bales	5.0	20.0	14	57
Hay, extra compressed, in bales	1.75	5.25	24	72
Hides, raw, in bales	6.0	30.0	23	117
Leather, sole, in bales	12.6	8.9	16	22
Leather, sole, in piles	—	—	17	—
Lime, in barrels	3.6	4.5	50	63
Oats, in bags	3.3	3.6	27	29
Oil, lard, in barrels	4.3	12.3	34	98
Paper, Manila	—	—	37	—
Paper, news	—	—	38	—
Paper, super-calendered	—	—	69	—
Paper, wrapping	—	—	10	—
Paper, writing	—	—	64	—
Prints, cotton, in cases	4.5	13.4	31	93
Rags, jute butts, in bales	2.8	11.0	36	143
Rags, woollen, in bales	7.5	30.0	20	80
Rags, white cotton, in bales	9.2	40.0	18	78
Rags, white linen, in bales	8.5	39.5	23	107
Sheetings, bleached cotton, in cases	4.8	11.4	30	69
Starch, in barrels	3.0	10.5	23	83
Straw, extra compressed, in bales	1.75	5.25	19	57

Material	Measurements. Floor Space. Occupied.		Weight.	
	Sq. Ft.	Cu. Ft.	Lbs. per Cu. Ft.	Lbs per. Sq. Ft.
Sugar, brown, in barrels...........	3.0	7.5	45	113
Tickings, cotton, in bales...........	3.3	8.8	37	99
Tin, in boxes.....................	2.7	0.5	278	99
Wheat, in bags...................	4.2	4.2	.39	39
Wheat, in bulk...................	—	—	41	—
Wool, Australian, in bales.........	5.8	26.0	15	66
Wool, Californian, in bales........	7.5	33.0	17	73
Wool, South American, in bales....	7.0	34.0	29	143

Assembly Halls, Ball Rooms, and Drill Halls are subjected to vibrations caused by rapidly moving live loads, due to walking, dancing, or the rhythmic tread of drilling soldiers, and consequently a somewhat greater live load shall be assumed for buildings of this character than for theatres or churches, etc. All such floors shall be designed for a live load of at least 120 lb. per square foot; although a live load of 150 lb. shall not be considered excessive, and is required in many cases.

Factories are also subjected to vibrations due to moving machinery. Extreme vibrations are likely to occur in buildings where large printing presses are at work, and great caution shall be exercised in designing the floors and supporting members of such buildings. In all such cases, wherever practicable, the location of the presses shall be decided upon, and special provision made in the floor construction for same.

A live floor load of from 100 to 150 lb. per square foot shall be considered sufficient in buildings as ordinarily used for manufacturing purposes, and shall be ample to include the effects of vibrations caused by moving machinery. The pull of large belts or rope drivers are a source of considerable vibration and must be investigated and provided for. If the vibrations are likely to be excessive, and their possible effect not ascertainable the live load shall be taken as double that usually adopted for buildings of like character not subjected to similar causes of vibration.

For machine shops and manufacturing plants, where the work is extremely heavy, a live floor load of 450 lb. per square foot of floor surface shall not be considered excessive. In dealing with this class of buildings the designer shall acquaint himself with the construction and action of the machinery likely to be employed, also the processes used by the manufacture, and thus guard against contingencies that may arise in the future.

In factories and warehouses the footings and foundation piers shall be designed to sustain the full live load, on all floors, and differ in this respect from office buildings.

FOUNDATIONS

Foundations must be properly proportioned, and great care exercised in their design. They shall be required to sustain the maximum load upon them, which shall consist of the dead and live load, subjected to exceptions and considerations as previously set forth. The accumulated dead and live load shall be considered, and the area of the footings and foundation piers shall be such that the greatest pressure per square foot does not exceed the safe values given for the various soils in the following table:—

Material	Safe Bearing Value in Tons per Sq. Ft.
Compact bed rock, of granite	30
Compact bed rock, of limestone	25
Compact bed rock, of sandstone	18
Soft friable rock	5 to 10
Clay, in thick beds, absolutely dry	4
Clay, in thick beds, moderately dry	2
Soft clay	1
Dry coarse gravel, well packed and confined	6 to 8
Compact dry sand, well cemented and confined	4
Clean dry sand, in natural beds and confined	2 to 4
Good solid dry natural earth	4 to 6

Where there is liability of the foundation soil being loosened by water, or by adjacent building operations, due precautions shall be taken to prevent serious results from attending such contingencies, poor soils, wherever practicable, shall be improved so that they are better adapted for foundations. If the soil is wet or saturated an effort shall be made to improve it by drainage. Loose soils must be rammed or otherwise compacted, and piles shall be driven at intervals to accomplish such results where required. All foundations shall be protected against frost, which must be accomplished by excavation, for the foundation, at least 6 ft. below the surface of the surrounding soil.

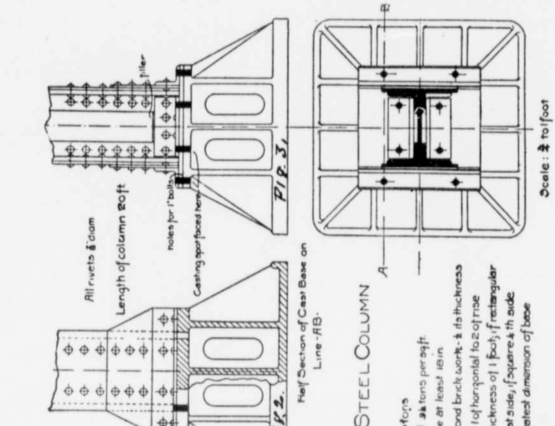
Since foundations upon yielding material will settle, more or less care must be exercised to see that this settlement be uniform. In order to obtain uniform settlement the pressure on a unit of area must be the same under all foundation footings and piers; providing the nature of the soil is the same throughout the area covered by the building. If the soil under different portions of the building is likely to have different bearing values, it shall be carefully tested, the safe bearing values being ascertained at several points throughout the foundation area, and the foundation footings and pier foundations so proportioned that the settlement shall be as uniform as possible.

Where the actual live load is variable, and seldom approaches the load originally assumed, as in office buildings, equal settlement of the foundations shall be obtained by proportioning the areas of the footings, so that the dead loads produce equal pressure. For example, the heaviest loaded foundation pier in a building supports a dead load of 200 tons, and a live load is also of 200 tons; while another foundation supports a dead load of 150 tons, and a live load of 100 tons. The first pier sustains a total load of 400 tons, and, assuming the soil to safely sustain a load of 4 tons per square foot of bearing area, the area of footing required for the heaviest pier, to carry both the dead and live load, equals $400 \div 4$, or 100 square feet. This area gives, when estimating on the dead load alone, a bearing pressure of 2 tons per square foot. Using this unit bearing value of 2 tons for the second pier, the area of base required, figuring on the dead load alone, would be 75 square feet; while, if both the dead and live loads were considered in proportioning the foundation, the required bearing area would have been 62.5 square feet, and the dead load would cause a pressure upon the soil under the second pier of only 1.6 tons, which would have a tendency to cause unequal settlement.

FOUNDATION PIERS

All foundation piers resting upon soils of good dry natural earth, having a safe bearing value of from 3 to 4 tons per square foot, shall be designed as shown on the accompanying detail, see Figs. 1, 2, and 3.

The concrete base shall be composed of one part cement, two parts sand and five parts of broken stone, small enough to pass through a 2-in. diameter ring. It shall have such an area that the pressure per square foot upon the soil shall not exceed its safe bearing value. The offset a of the concrete shall not be less than one-half its depth or thickness b as shown on the accompanying detail. The thickness b of the concrete shall never be less than 18 in. The body of the pier shall be of good quality hard bricks laid in Portland cement mortar, and the area of its base shall be so proportioned that the pressure upon each square inch of concrete shall not be more than 200 lb. The latter of the sides of the brick pier shall never exceed a 6-in. offset to a



FIGS. 1, 2 AND 3

12-in. rise; that is, the rise shall be in ratio to the offset as two to one. The area of the capstone in juxtaposition to the brickwork shall be such that the safe bearing value of the brickwork is not exceeded. Should the load upon the cap be great a heavy cast-iron pedestal shall be provided in order to distribute the pressure equally over the capstone. The area of the cast-iron base bearing on the capstone shall be such that the allowable crushing strength of the stone is not exceeded. The maximum pressure allowed on the materials employed in the construction of foundation piers of the aforesaid character shall be as follows:—

Granite	350 lb. per sq. in.
Limestone	300 lb. per sq. in.
Sandstone	250 lb. per sq. in.
Brickwork in Portland Cement	200 lb. per sq. in.
Concrete	200 lb. per sq. in.

Capstones shall be of granite, limestone, or sandstone, and shall have a minimum thickness of 1 ft.; though its thickness in any case shall never be less than one-fifth its greatest dimensions, if rectangular, and one-fourth the length of its side if square. The cast-iron pedestal shall have a height of one-half the greatest dimension of its base, and shall be so proportioned that the pressure upon each square inch of its cross-section does not exceed 14,000 lb.

The following is an example of the application of the foregoing information. It shall be required to design a square foundation pier of hard bricks with concrete footing, all in Portland cement mortar. The capstone is of granite and supports a cast-iron pedestal, which in turn sustains a structural steel plate and angle column. The load upon the column is 600,000 lb., and the safe bearing value of the soil has been ascertained to be $3\frac{1}{2}$ tons per square foot.

The size of the cast-iron pedestal is dependent upon the unit allowable bearing value of the capstone, taken from the preceding table as 350 lb. per square inch; then $600{,}000 \div 350 = 1{,}714$ sq. in. required, and the dimensions of the base will equal $\sqrt{1{,}714}$, or approximately 42 in. on a side. The size of the granite capstone will be determined by limiting its pressure on the brickwork to 200 lb. per square inch, and the area required on the bed of the stone will equal $600{,}000 \div 200$ or 300 sq. in.; the length of its sides being $\sqrt{3{,}000}$, or 55 in. Its thickness will be one-fourth its base, or 14 in. The area required for the concrete footing is equal to $600{,}000 \div 7{,}000$, or 86 sq. ft. and consequently it shall be 9 ft. 3 in. square, its thickness being 18 in.

Since the sides of the brickwork are battered 1 to 2, and the dimensions of its top and bottom are known, its height shall be found as follows: One-half the difference between the top and bottom dimensions equals

$$\frac{(111 - 18) - 55}{2}$$ or 19 in. The 18 in the calculation is twice the offset of the concrete beyond the brickwork, which, as previously stated, shall not be less than one-half the thickness of the concrete, it is in this case 9 in. The latter is 1 to 2, therefore the height of the brickwork is equal to 19×2, or 38 in.

From these calculations the design of the foundation pier cap pedestal, and foot of column shall be as shown on the accompanying illustrations.

PILE FOUNDATION.

Pile foundations shall be used where the soil is clayey and insufficient to sustain the desired load, or where there is a stratum of clay with firmer soil beneath.

Timber piles, if properly driven, and their location allows them to be submerged under water, shall be considered as satisfactory and permanent foundation. All timber piles shall be spaced from 2 ft. to 3 ft. between centres, and shall be driven to an equal bearing, which shall be determined by the distance penetrated under the last blow of the hammer. The bearing load on piles driven to bed rock through stiff soil, capable of providing lateral support, shall be considered as equal to the safe direct compressive strength of the timber taken at the least cross section. If the surrounding soil is plastic and incapable of furnishing lateral support, then the pile shall be considered as a timber column of the same length as the pile. Its safe strength per sq. inch of cross section shall then be determined by the formula

$S - \frac{SL}{8.33\,D}$, where S equals the allowable compressive strength of the timber in pounds per sq. inch, L the length of the pile in feet, and D the diameter of the column in inches. The allowable compressive values of the several kinds of timber used as piles shall be as follows:

White Oak	800 lb. per sq. in.
Southern, Long Leaf, or Georgia Yellow Pine	700 lb. per sq. in.
Cypress	800 lb. per sq. in.
Chestnut	1000 lb. per sq. in.
Cedar	800 lb. per sq. in.
Norway Pine	800 lb. per sq. in.
Oregon Pine	1200 lb. per sq. in.

In locations where bed rock or hard pan cannot be reached, and the piles are driven through yielding material, they shall be so loaded that the weight upon them does not exceed the value of L in the following formula:

$L = \frac{2\,WH}{P + I}$, where W is the weight of the hammer in tons; H the drop of the hammer in feet, and P is the distances in inches penetrated by the pile under the last blow of the hammer. The value P shall be considered only when the burred splinters have been removed from the top, previously to the last blow being struck.

Under no conditions, however, shall the maximum load upon any pile exceed 20 tons, and the least allowable diameter of any pile shall be 5 in.

After the piles shall have been properly driven the tops shall be cut off to a level and capped with granite or provided with oak grillage, as thought expedient.

In constructing pile foundations great care shall be exercised to prevent the disturbing of adjacent building foundations. If it is found necessary to drive them close to existing foundations, especially if such foundations support heavy buildings, and the surrounding earth is liable to considerable compression, due to the super-imposed load, such construction shall be abandoned and the foundations designed in such a manner as to prevent the danger attending the disturbance of adjacent footings.

STEEL BEAM GRILLAGE.

Steel beam grillage is the form of foundation constructed as shown on the accompanying design, Fig. 4, and shall be resorted to where the loads to be supported are heavy and the soil exceedingly plastic. Its use is especially recommended where a thin and compact stratum overlies another of a more yielding nature, and where in consequence the available height of the foundation is limited, for if possible the requisite area for the foundation shall be obtained without penetrating the firm stratum, and without incroaching too much upon it.

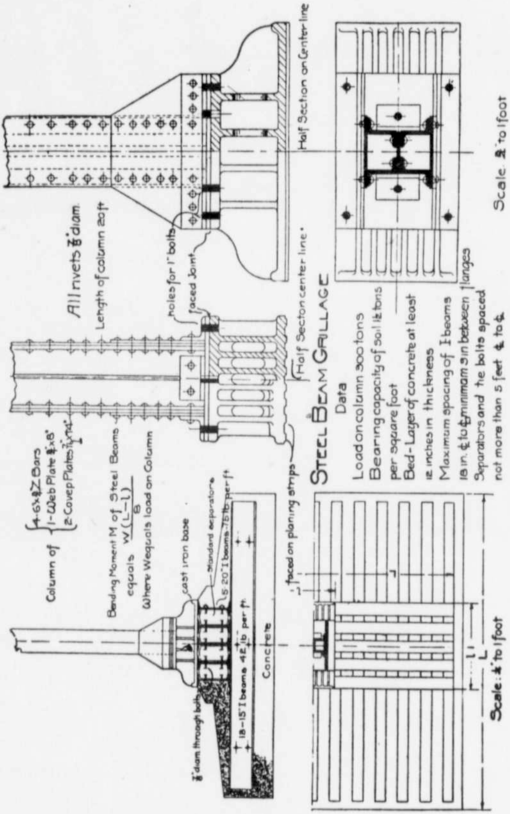


FIG. 4

This type of foundation shall consist of two or more layers of steel I beams, or, if occasion requires it, steel rails may be substituted. In constructing such a foundation a bed of concrete not less than 12 in. and of such a quality as heretofore specified shall be first laid upon this, the steel I beams shall be placed side by side of sufficient number and dimensions to distribute the load over the desired area. These beams shall be thoroughly embedded in concrete by filling between, and well-ramming. The second tier of beams supporting the cast base shall be laid across the first layer, reaching to the extreme outer edge. They shall likewise be filled with concrete. Separators must be provided between the I beams and the distance between the separators shall not exceed 5 ft. The tie bolts holding the separators in place shall pass through the entire tier of beams. The I beams shall never be spaced closer than 3 in. at the flanges, as less space would interfere with proper ramming of the concrete. Neither shall they be spaced at a greater distance apart than 18 in. between centres.

Should the load upon the foundation be so great as to require more beams in the second tier than can be conveniently spanned by the cast-iron base a third tier of beams may be introduced, or box girders may be used instead of the beams.

In determining the strength of steel grillage beams the following formula shall be used to obtain the maximum bending moment M, which is considered as existing at the centre of the length of the beam $\frac{M = W\ (L - l)}{8}$.

Where W is the superimposed load in pounds on the foundation; l is the length over which the superimposed load is applied; and L is the length of the beam. The bending moment shall thus be obtained in inch-pounds or in foot-pounds, according as the length represented by L. and l in the formula are taken in inches or feet. The distance L and l for each tire of beams are clearly shown on the accompanying detail. The allowable fibre stress employed in figuring the strength of steel beams used as grillage shall be 15,000 lb. per square inch.

From the following table the values of I for the usual steel I beams may be obtained.

Depth of Beam	Weight per foot	Area in Sq. inches	Thickness of Web in inches	Width of Flange	Value of I, the neutral axis square, to Web at center
20	90	26.4	.78	6.75	1506
20	80	23.5	.69	6.38	1345
20	75	22.1	.66	6.16	1246
20	65	19.1	.50	6.00	1148
15	75	22.1	.81	6.29	720
15	66⅔	19.7	.65	6.13	676
15	60	17.6	.52	6.00	637
15	50	14.7	.45	5.76	529
15	42	12.4	.40	5.50	429
12	55	16.1	.63	6.00	358
12	40	11.8	.39	5.50	281
12	31½	9.2	.35	5.13	220
10	40	11.8	.58	5.21	178
10	33	9.7	.37	5.00	161
10	30	8.8	.45	4.89	134
10	25	7.3	.31	4.75	122
9	27	7.9	.31	4.75	110
9	23½	6.9	.35	4.58	89
9	21	6.2	.27	4.50	84
8	27	7.9	.49	4.56	77
8	22	6.4	.29	4.38	69
8	18	5.2	.25	4.13	56
7	20	5.7	.28	4.09	47
7	15	4.4	.23	3.88	37
6	15	4.3	.25	3.52	26
6	12	3.6	.22	3.38	21.7
5	13	3.8	.26	3.13	15.7
5	9¾	2.9	.21	3.00	12.1
4	10	2.9	.39	2.69	6.84
4	7½	2.2	.20	2.50	5.86
4	6	1.8	.18	2.19	4.59

The following shall be an example in the design of a steel beam grillage foundation. The soil safely sustains a load of $1\frac{1}{2}$ tons per square foot of surface, and the load upon the column is 300 tons.

The area required in the footing will be $300 - 1\frac{1}{2} =$ 200 sq. ft., and if square shall be 14 ft. on a side, which, while somewhat scant, will be ample. The accompanying plate (Figs. 5 and 6), shows the layout of the beams as assumed in the example. The size of the beams in the upper tier will be determined first. Since they are five in number and are 4 ft. across, this being as many as can be placed at the minimum distance apart that can be conveniently spanned by the cast pedestal—the load carried by each beam will be one-fifth the total load, or 60 tons. By substituting in the formula previously given, the maximum bending on each of these beams is found to be equal $\frac{120{,}000\ (14 - 4)}{8}$, or 150,000 foot-pounds or 1,800,000 inch-pounds.

The resisting moment in inch-pounds of any beam equals $\frac{IS}{C}$. Where I is the moment of inertia of the section and C is one-half the depth of the beam in inches; S being the safe unit fibre stress of the material.

The value of I for a 20 in. 75 lb. beam is 1,246, and by substitution the formula $\frac{IS}{C}$ equals $\frac{1{,}246 \times 15{,}000}{10}$ or, 1,869,000 the resisting moment of the beam in inch-pounds, and, since the maximum bending moment is only 1,800,000, this size beam shall be considered amply strong for the top tier.

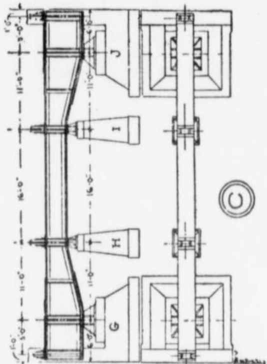

G
H
I
J
C
11'-0"
16'-0"
5'-0"
1'-0"

FIG. 6

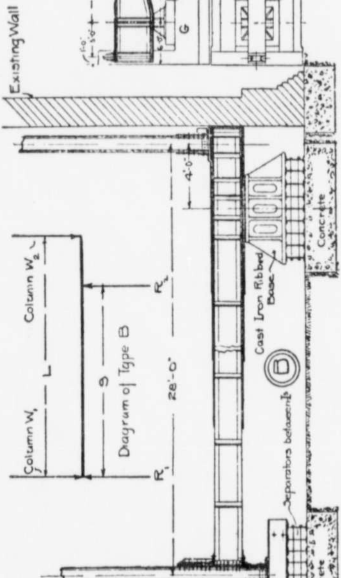

Existing Wall
Column W_1
L
Column W_2
S
Diagram of Type B
R_1
R_2
28'-0"
4'-0"
Cast Iron Ribbed Base
Separators between
B
Concrete
Concrete

FIG. 5

The bottom tier of beams are thirteen in number, and will each have to sustain a maximum bending moment of approximately $\frac{46,000\ (14 - 4)}{8}$, or 57,500 foot-pounds, which equals in inch-pounds 690,000. From the above table the value of I for a 15 in. 42 lb. beam is 402,000, its resisting moment, according to the formula $\frac{IS}{C}$ equals 858,000 inch-pounds, which is considerably in excess of that required; but as a 12-in. beam of sufficient weight to sustain the load is heavier than the selected 15-in. beam, it shall be considered economy to use the deeper beam.

The design of the foundation, as calculated above, shall be as shown in the accompanying design, see Figs. 5 and 6.

CANTILEVER FOUNDATIONS

Cantilever foundations shall be used where it is deemed advisable to undermine existing walls on adjoining property, and shall also be employed where it is impracticable to locate the exterior columns on the centre of the wall or wall footings. In all such cases the wall columns shall be supported upon the end of an overhanging girder, which in turn is sustained by a foundation pier well within the building line. The several types of cantilever foundations that may be used are as shown at L, B, and C on the accompanying designs, Figs. 5 and 6.

Type L shall be used on narrow buildings where the foundation supporting the cantilever are small, and in consequence the overhang of the cantilever is comparative short. The bending moment on this character of cantilever girder is small, as the cast-iron base practically transfers the load upon the column to the foundation direct.

Type B may be adopted to advantage in many buildings, and the cantilever beam can be made of heavy I beams, or it can be composed of plates and rolled shapes forming plain or box girders.

The foundation shall be placed as near the wall as possible, the wall column resting upon the over-hanging portion of the girder, while the other end of the girder is securely rivited to the interior column. Great care shall be exercised in the design of such a foundation to ascertain that the moment due to the load upon the wall column acting about the outer pier as a fulcrum is not sufficient to raise the interior column with its sustaining load. Sufficient rivets shall also be placed in the connection between the girder and interior column so that their safe resistance to shear is equal to the upper tendency of the girder at this point.

The type of cantilever foundation C shall be adopted in extremely heavy buildings requiring large foundations and consequently considerably overhang. In this type the central span L can be omitted, though by so doing the continuity of the girder is destroyed, and although the calculations involved in figuring the reactions upon the foundations are greatly simplified, the increased rigidity and safety of the continuous system shall be greatly preferred.

The following shall be an example of the calculations employed in determining the reactions, and the bending moment on the cantilever girder used in the type of foundation shown at B on the accompanying plate, see

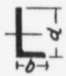
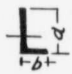
Shape of Section	Moment of Inertia	Section Modulus	Distance of Base from Centre of Gravity	Least Radius of Gyration
	$\frac{Ad^2}{6.1}$	$\frac{Ad}{3.0}$	$\frac{d}{2}$*	$\frac{b}{5.2}$
	$\frac{Ad^2}{6.73}$	$\frac{Ad}{3.3}$	$\frac{d}{2}$*	$\frac{b}{3.56}$
	$\frac{Ad^2}{10.9}$	$\frac{Ad}{7.6}$	$\frac{d}{3.3}$	$\frac{b}{4.00}$
	$\frac{Ad^2}{10.4}$	$\frac{Ad}{7.4}$	$\frac{d}{3.5}$	$\frac{b}{5}$
	$\frac{Ad^2}{9.9}$	$\frac{Ad}{6.7}$	$\frac{d}{3.1}$	$\frac{bd}{2.6(d+b)}$
	$\frac{bd^3}{12}$*	$\frac{bd^2}{6}$*	$\frac{d}{2}$*	$\frac{\text{Least Side}}{3.46}$*

Fig. 7

diagrams, Fig. 7, which also gives a diagram of the loads and the supporting reactions. The load upon the column W_1 is 100 tons, while that upon column W_2 is 150 tons. The distances L and S are 28 ft. and 24 ft. respectively, taken to the centres of the columns and the bearings.

The reaction R_2, then, equals $\frac{W_2L}{S}$, which gives $\frac{200,000 \times 28}{24}$, or 233,333 lb. The sum of the reaction must equal the sum of the loads, and hence R_1 equals $(200,000 \div 300,000) - 233,333$, or 266,667 lb.

The bending moment on the cantilever girder is equal to W_2 $(L - S)$ or $200,000 \times 4 = 800,000$ foot pounds. Assuming that the girder is composed of a single web, plate and angle girder 2 ft. in depth.

The net flange area required can be found by the formula $\frac{M}{D\,S}$, where M is the maximum bending moment in foot pounds, D the depth of the girder in feet, and S the safe allowable unit fibre stress of the material in pounds. By substituting the values in the preceding formula the net flange area required is found to equal $\frac{15,000 \times 2}{800,000}$, or approximately 27 square inches. Therefore, making allowance for the area cut out for rivet holes, the flange of the girder shall be composed of 3 — $\frac{7}{16}$ in. × 14 in. flange plates and 2 — 6 in. × 4 in. × ¾ in. angles.

The details of the design shall be as shown on the accompanying plates.

In calculating the reactions of the type C, the cantilever girder, it shall be considered as inverted, the four supports or the reactions being the columns, and the pier reactions regarded as the loads. The diagram of the girder C is shown on the accompanying detail, see Figures, based upon these conditions. The reactions may be calculated from the following formula:—

$$R_1 = R_4 = P \, \frac{4 + 8x + 3x^2 - 6y - 9xy - 3\,x^2y + 2y^3 + xy^3}{4 + 8x + 3x^2}$$

$$R_2 = R_3 = P \, \frac{6y + 9xy + 3x^2y - 2y^3 - xy^3}{4 + 8x + 3x^2}$$

In the problem under consideration, L = 16 ft., $x = 1$, and $y = \frac{5}{16}$.

The maximum bending moment at the pier G is equal to the load on the exterior column multiplied by the distance between its centre and the centre of the bearing. The bending moment at the pier H is equal to the load on the exterior column multiplied by the distance between its centre and the centre of the bearing G minus the product of the reaction at the pier G by its distance from the pier H.

Before considering the design of structural steel columns, their connections and wind bracing, the strength of rolled steel beams, and the construction of plate or built-up girders, it is necessary that the properties of such sections be determined.

The important elementary value of any section is the *moment of inertia* represented by I. This property I of any section having been obtained, the other properties of the section, which are the *radius of gyration* and the *sectional modules*, are readily ascertainable. These two values shall be represented by the letters R and Q respectively.

The moment of inertia of any plane section shall be considered as equal to the sum of the areas of the particle composing the section multiplied by the square of the distances they are located from the neutral axis. The *neutral axis* shall be taken as a line passing through the centre of gravity of the section, and perpendicular to the direction in which the above named distances are measured. This rule shall be expressed in formula as $I = \sum ad^2$, in which $\sum$ signifies sum of, a the area of each particle constituting the section, and d the distance from the centre of gravity of each particle to the neutral axis, measured in a perpendicular line.

The following formulae for the usual rolled steel sections shall be deemed sufficiently accurate for all practical purposes, and will be used in subsequent calculations, though when recourse can be had to manufacturers' tables of the properties of rolled sections, they can be used more conveniently and accurately. Such tables are given in connection with this article.

FORMULAE FOR DETERMINING THE PROPERTIES OF USUAL ROLLED STEEL SECTIONS

Note.—A = area of section. Values for radius of gyration in flanged beams apply to standard minimum sections only. All the terms are approximate except those marked with an asterisk, which are correct.

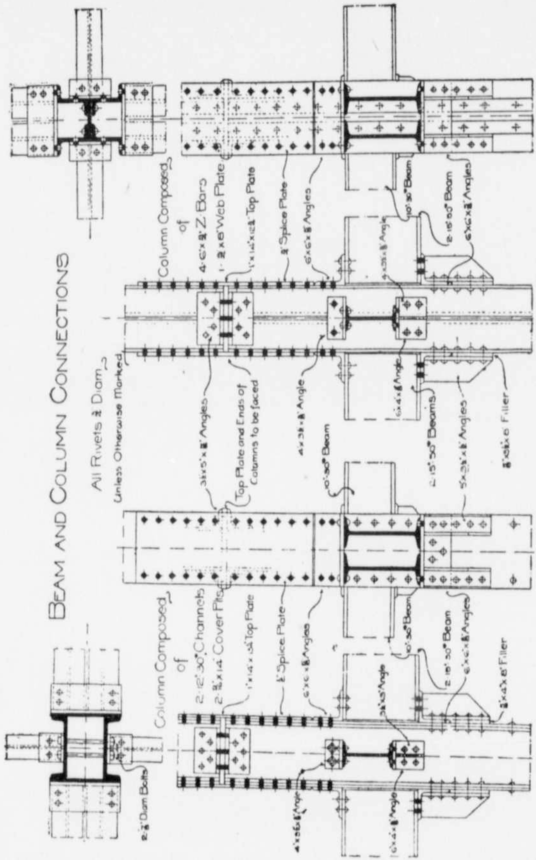


FIG. 8

The moment of inertia of a built-up section, such as is commonly made of rolled steel shapes and plates, and used for steel columns, plate girders, etc., shall be determined in the following manner: Find or locate the neutral axis around which the moment of inertia is desired, by multiplying the area of each simple elementary section, is from a line or origin of moments drawn outing through the centre of gravity of each elementary section, is from a line or origin of moments drawn outside of the figure parallel with the neutral axis, and dividing this product by the total area of the figure. For instance, in the accompanying figure 9, let a, b, represent any line or origin of moments drawn outside of the figure, then the calculation for this section will be as follows:—

Elementary Section.	Area of Elementary Section square in.	Distance of Center of Gravity of Elementary Section from Line a, b.	Products
Top plate.........	6.00	1 n	1.50
Two upper angles..	12.22	.25	22.60
Web plate	7.00	1.85	52.50
Two lower angles..	7.12	7.50	95.90
Total...........	32.34	13.47	172.50

Note.—In the above figure the distance, d, from the back of the angles to their centre of gravity is obtained from the Tables giving the properties of rolled steel sections.

Dividing the sum of the products 172.50, by the total area of the section 32.34, gives 5.33 in., or the distance c, in the figure from the line a, b., or origin of moments, thus locating the neutral axis. Where the figure is symmetrical the above calculation is not necessary, as the neutral axis shall be considered as passing through the centre of the figure. The neutral axis being determined, the moment of inertia, or, I, of any built-up section, shall be taken as equal to the sum of the products of the areas of each elementary section by the square of the perpendicular distance from a line passing through its centre of gravity to the neutral axis, plus the moment of inertia of each elementary section. That is, let a, equal the area of each elementary figure or section, and d, the perpendicular distance from the neutral axis to a line passing through the centre of gravity of the elementary figure or section; and let i, equal the moment of inertia of each elementary section. Then the moment of the inertia

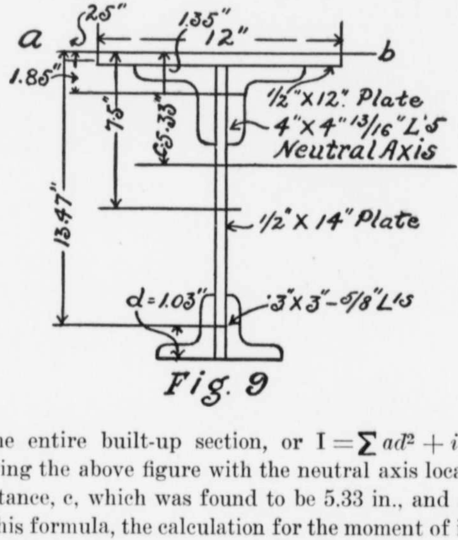


Fig. 9

of the entire built-up section, or $I = \sum ad^2 + i$. Redrawing the above figure with the neutral axis located at a distance, c, which was found to be 5.33 in., and applying this formula, the calculation for the moment of inertia is as follows:—

Elementary	a d^2	i.	$ad^2 + i$
Section.	600 sq. in. × 25.80 = 154.80	.125	154.92
Top plate 1222	sq. in. × 12.11 = 147.98	18.90	166.88
Two upper angles.. 700	sq. in. × 4.70 = 32.90	114.33	147.23
Web plate 712	sq. in. × 66.25 = 471.70	6.40	478.10
Two lower angles..$I=\sum ad^2 + i$ or	....	....	947.13

In the above calculation the values of i for the top and web plates are determined from the formula given in the table of formulas for figuring the properties of elementary sections, while the same values for the angles are taken from the table giving the properties of usual sections.

The radius of gyration of any section shall be obtained when the moment of inertia is known by dividing that value or I by the total area of the section or A, and extracting the square root of the quotient. Thus, by

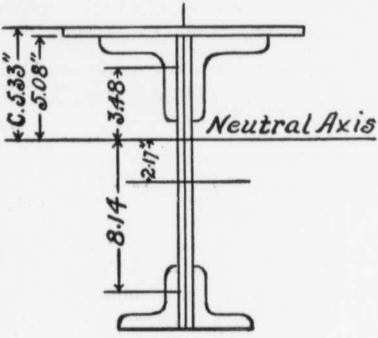


Fig. 10

formula, $R = \sqrt{\frac{I}{A}}$, or $R^2 = \frac{I}{A}$; in which R equals the radius of gyration. For instance, in the preceding example, the moment of inertia of the section was found to be 947.13, and the total area 32.34 square inches, then the square of the radius of gyration or $R^2 = \frac{947.13}{32.34}$ or 29.28, and $R = \sqrt{29.28}$ or 5.41.

The section modules of any section or figure shall be obtained, when the moment of inertia is known, by dividing this value by the greatest distance the neutral axis, see Fig. 10, or line passing through the centre of gravity, of the figure is located from the outside fibres of the section.

Expressed in formula, the section modulus or $Q = \dfrac{I}{C}$;

in which c equals the greatest distance from the neutral axis to the outside fibre. To demonstrate, consider the preceding example, where the moment of inertia is 947.13, and the greatest distance from the neutral axis to the outside of the section is 9.17 in.; Q in this case equals $\dfrac{947.13}{9.17}$, or 103.2. Again, if the moment of inertia or I of a 12-in. I beam is 281.3, and the distance c, which, when the section is symmetrical, equals one-half the depth, is 6 in.; the section modulus or $Q = \dfrac{281.3}{6}$, or 46.9.

The following tables give the moment of inertia, radius of gyration, and section modulus, together with other properties of the usual rolled steel sections employed in structural work.

Structural steel columns can be of varied cross-section depending upon the requirements. The following sections are in general use, and shall be given preference; the first four forms being the usual marketed columns,

PROPERTIES OF ANGLES

EVEN LEGS.

Maximum and Minimum Thicknesses and Weights.

Size of Angle.	Thickness	Weight per foot.	Area of Section.	Distance from Centre of Gravity to Back of Angle	Moment of Inertia, Axis Parallel to Flange.	Section Modulus Axis the same.	Radius of Gyration, Axis the same.	Least Radius of Gyration, axis Diagonal.
in.	in.	lb.	sq. in	in.	I	Q	R	R_2
6 × 6	7/8	34·0	10·03	1·87	35·3	8·17	1·87	1·20
6 × 6	3/8	14·8	4·36	1·64	15·4	3·52	1·88	1·20
5 × 5	3/4	24·2	7·11	1·56	17·0	4·78	1·55	1·00
5 × 5	3/8	12·3	3·61	1·39	8·74	2·42	1·56	1·00
4 × 4	13/16	20·8	6·11	1·35	9·45	3·32	1·24	·80
4 × 4	5/16	8·16	2·40	1·12	3·72	1·29	1·24	·80
3½ × 3½	5/8	13·5	3·98	1·10	4·33	1·81	1·04	·70
3½ × 3½	5/16	7·11	2·09	·99	2·45	·98	1·08	·70
3 × 3	5/8	12·1	3·56	1·03	3·20	1·48	·94	·60
3 × 3	1/4	4·9	1·44	·84	1·24	·58	·93	·60
2½ × 2½	1/2	7·35	2·31	·82	1·33	·76	·76	·50
2½ × 2½	1/4	4·05	1·19	·72	·70	·40	·77	·50
2¼ × 2¼	1/2	7·17	2·11	·78	1·04	·65	·70	·45
2¼ × 2¼	3/16	2·75	0·81	·63	·39	·24	·69	·45
2 × 2	1/2	6·32	1·86	·72	·72	·51	·62	·40
2 × 2	3/16	2·41	·71	·57	·28	·19	·62	·40
1¾ × 1¾	7/16	4·72	1·39	·61	·39	·32	·52	·35
1¾ × 1¾	3/16	2·11	·62	·51	·18	·14	·54	·35
1½ × 1½	3/8	3·33	·98	·51	·19	·19	·44	·30
1½ × 1½	3/16	1·80	·53	·44	·110	·104	·46	·30
1¼ × 1¼	5/16	2·55	·75	·46	·123	·134	·40	·25
1¼ × 1¼	1/8	1·02	·30	·35	·044	·049	·38	·25
1 × 1	1/4	1·57	·46	·36	·045	·064	·31	·20
1 × 1	1/8	·78	·23	·30	·022	·031	·31	·20
7/8 × 7/8	3/16	·99	·29	·29	·019	·033	·26	·175
7/8 × 7/8	1/8	·68	·20	·25	·014	·022	·27	·175
3/4 × 3/4	3/16	·85	·25	·26	·012	·024	·22	·15
3/4 × 3/4	1/8	·58	·17	·23	·009	·017	·23	·15

the latter two being patented sections in more or less demand.

In selecting the column section best adapted to building construction the following considerations shall be carefully studied.

First.—Economy in both material and cost of construc-

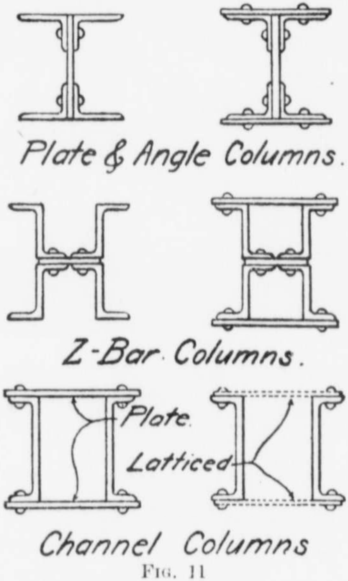


FIG. 11

tion. Economy shall be attained when the metal in the column section is distributed at a maximum distance from the central axis of the column, which will also fulfil the condition that the radius of gyration shall be the same, or nearly so, on all the axis of the column section; for the square of the *least* radius of gyration shall be used in all subsequent formulas for the determination of the supporting strength of columns.

Economy in cost of construction is principally accomplished by so designing the column section that it can readily be built up of stock-rolled shapes, arranged in

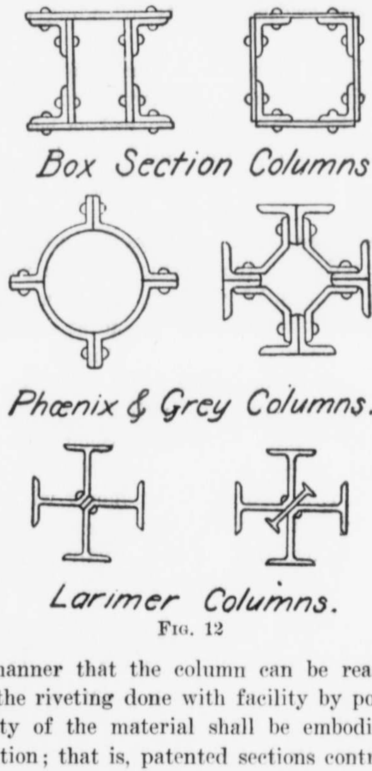


FIG. 12

such a manner that the column can be readily fitted up, and the riveting done with facility by power. The availability of the material shall be embodied in this consideration; that is, patented sections controlled by a distant manufacture shall be avoided, unless prompt shipment is assured, as the delay occasioned by failure to ship, and the impossibility to get the material elsewhere, would be likely to cause pecuniary loss. (See Fig. 18.)

Second.—That section shall be considered the best constructively where it is so arranged that the loads may be transmitted directly to the centre of the column. Thus eccentric loading and its attending dangers are avoided.

Third.—The column section shall be such, that all beam and girder connections thereto can be made conveniently without the use of swaged or bent plates of peculiar shape, and so that all fitting in the field is avoided, and the work can be rapidly and securely assembled.

Fourth.—Where, as in buildings of many stories, the columns extend from the basement to the roof in one continuous length, and for reasons of economy it is necessary to reduce the column section at every several stories; that section shall be used that will allow the reduction of the sectional area of the rolled steel shapes employed, without materially reducing the general dimensions of the column section. Should the general dimensions of the column be materially reduced, at the several floors, it is evident that economy will require a column of such small dimensions in the upper tiers that the difficulty of making good constructive details of the beam and girder connections will be manifest.

Fifth.—The column section shall be of such a form that it is easily accessible for inspection and painting; though when completely fireproofed this consideration is of less importance, as the covering will tend to prevent the accumulation of moisture on the column and lessen the liability of deterioration.

The strength of a structural steel column depends upon the crushing strength of the material, the resistance of the column to lateral flexture, which is influenced by its length, the method in which the ends of the column are secured, and the radius of gyration of the column sections. Long columns have less strength than short

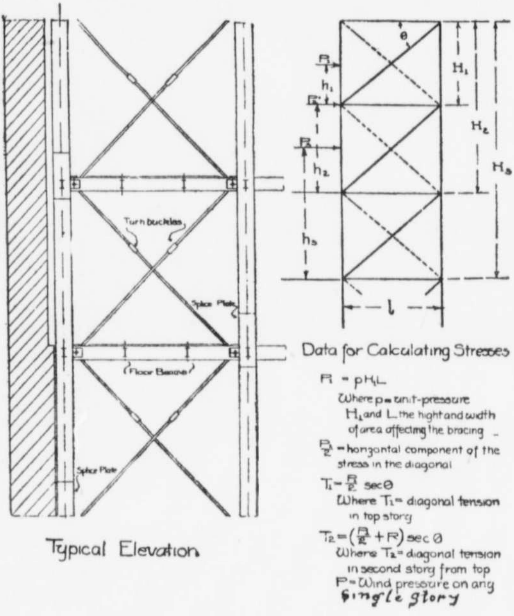


FIG. 13

columns of the same cross-sections, for the reason that they are liable to fail by lateral deflection; while if the columns were of the same length and cross-sectional area, the one in which the metal is distributed, the furthest from the centre of the section, will develop the greatest strength. The manner in which the ends of a column are secured greatly influence its bearing capacity, by diminishing or increasing its resistance to lateral flexture. Thus, a column with its ends firmly fixed or secured, so that it is liable to fail in its length before giving away at the ends, will develop more strength than columns secured with a hinge at the ends, or, as it is called, "pin-connected." Columns with flat ends will develop more strength than columns with hinged ends, but not so much as if the ends are fixed.

The formulas commonly used in figuring the strength of columns may be either the "Gordon," or what is known as the straight line formulas.

The Gordon formulas may be expressed as follows:—

$$\text{Hinged Ends; } S = \frac{U}{1 + \left(\dfrac{6^2}{18{,}000\ R^2}\right)}$$

$$\text{Square Ends; } S = \frac{U}{1 + \left(\dfrac{l^2}{24{,}000\ R^2}\right)}$$

$$\text{Fined Ends; } S = \frac{U}{1 + \left(\dfrac{l^2}{36{,}000\ R^2}\right)}$$

In which S equals the allowable unit fibre stress on the column section; U equals the allowable unit fibre stress of the material composing the column; l equals the length of the column in inches, and R equals the least radius of gyration of the column section. The following shall be considered as an example in the application of one of the above formulas: A structural steel plate and angle column, flat or square ended, composed of one 8 in. plate and four 4 in. $\times$ 3 in. angles, all 7-6 in. thick, has a least radius of gyration of 1.72 and a sectional area of 14.98 sq. in.; its length being 16 ft. Provided the ultimate unit crushing strength of the material is 60,000 lb. and a factor of safety of four is desired, what will be the safe load this column will sustain. Using the formula in which

$$S \frac{U}{\left(1 + \frac{l^2}{24{,}000\ R^2}\right)}$$

by substituting

$$S = \frac{15{,}000}{1 + \left(\frac{36{,}864}{24{,}000 \times 2.95}\right)}, \text{ or } 9 - 869,$$

which is the safe load in pounds the column will sustain per square inch of section and the entire allowable load on the column is equal to 9,869 multiplied by the area 14.98 or 147,837 lbs.

The straight line formulas are, however, much more convenient to handle and may be used to determine the *ultimate* strength in pounds per square inch, of columns whose lengths are between 50 and 150 radii of gyration. These formulas are as follows, and the results obtained by them are found to approximate closely those ascertained by actual tests upon full size specimens.

	Medium Steel.	Soft Steel.
Hinger Ends;	$S = 60{,}000 - 210\frac{l}{R}$;	$S = 54{,}000 - 185\frac{l}{R}$.
Square Ends;	$S = 60{,}000 - 230\frac{l}{R}$;	$S = 54{,}000 - 200\frac{l}{R}$.
Pin Ends;	$S = 60{,}000 - 260\frac{l}{R}$;	$S = 54{,}000 - 225\frac{l}{R}$.

In which S equals the ultimate strength of the column in pounds per square inch of section; l equals the length of the column in inches, and R equals the least radius of gyration of the section. Consider the preceding problem and apply the formula for square ends; that is $S = 60{,}000 - 230\frac{l}{R}$. By substituting $S = 60{,}000 - 230\frac{192}{1.72}$ or 34,300; which, divided by the factor of safety four, gives 8575, the allowable unit stress on the column in pounds. The area of the column being 14.98 square inch, the safe load that it will sustain equals 8575 + 14.98 or 128,453 pounds. This result differs considerably from the value obtained by the Gordon formula, but, as can be seen, it is on the side of safety. Where the length of the column is under 50 radii of gyration these formulas need not be applied, as within that limit columns of both medium and soft steel shall be considered as having a uniform ultimate strength of 48,000 pounds per square inch.

Square or flat-ended columns are mostly used in building construction.

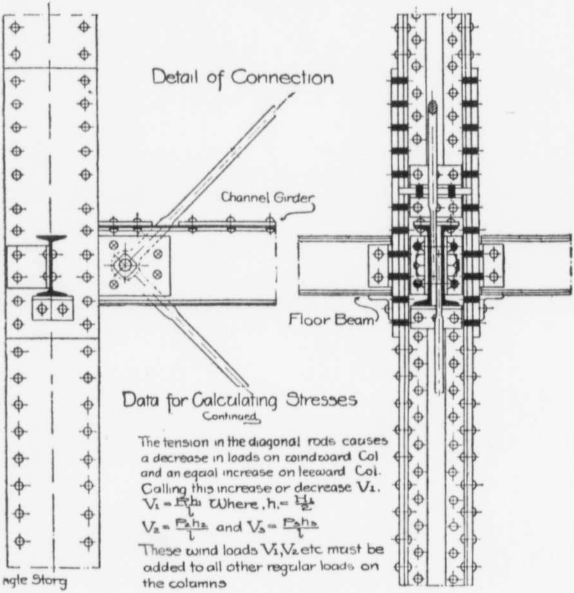


FIG. 14

Hinged or pin-connected columns, or struts being used, especially in framed structures, such as roof trusses. Hence, as it is usual to adopt a factor of safety of four, the following formulas will be found convenient in calculating the allowable unit stress for flat or square-ended columns, whose lengths are within 50 and 150 radii of gyration.

$$\text{Medium Steel, } S = 15{,}000 - 57\frac{l}{R}.$$

$$\text{Soft Steel, } S = 13{,}500 - 50\frac{l}{R}.$$

In which l and R represent the length of the column in inches and the least radius of gyration respectively, while S equals the allowable or safe load the column will sustain per square inch of section. Where the length of the column is under 50 radii of gyration the above formulas do not apply, and for these columns a safe bearing value of 12,000 pounds per square inch of section may be figured on. The use of any column having a length greater than 150 radii of gyration, or whose length exceeds forty-five times the least dimension of the column shall be condemned.

Columns subjected to bending stresses, due to eccentric loading or lateral forces, together with direct compressive stress, shall be proportioned to withstand the direct stress; the plates and rolled shapes composing the section shall then be increased in thickness, so as to supply ample section of metal to take care of the bending moment created in the column.

The bending moment due to an eccentric load shall be considered as equal to the amount of the load multiplied by its distance of application from the axis of the column. See Figs. 14, 21. That is $M = Wl$; where M equals the bending moment in inch-pounds; W equals the load in pounds, and l equals the distance from the

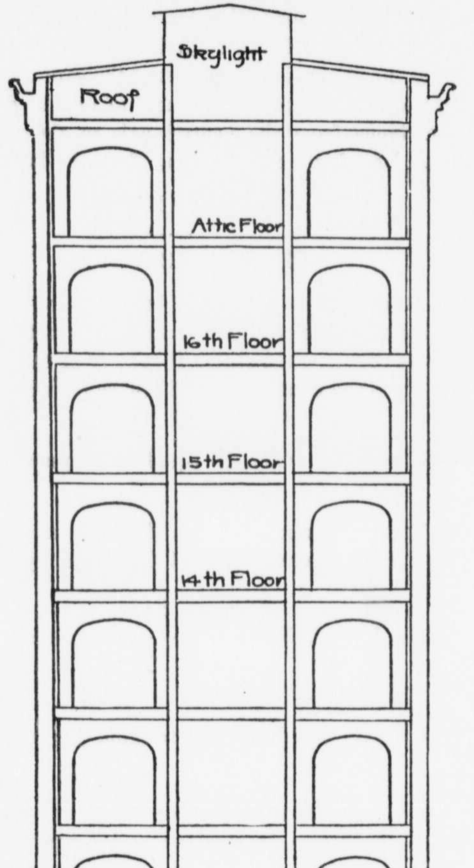


FIG. 15

line of action of the load W to the centre of the column. The bending moment M must equal the resisting moment M_1, and M_1 must equal the product of the safe unit fibre stress of the material and the moment of inertia, divided by the distance from the neutral axis to the extreme fibre. All of which expressed by formula is

$$M = M_1 = \frac{S\,I}{C}$$

Where S equals the safe unit fibre stress of the material; I equals the moment of inertia of the section, and C equals the distance, the centre of gravity of the section is located from the neutral axis. Then, as previously explained $R^2 = \frac{I}{A}$, in which R^2 equals the square of the radius of gyration, and A equals the total area of the section. I must, therefore, equal R^2A, and by substituting in the formula $M = M_1 = \frac{S\,I}{C}$ the value of $M = \frac{S\,R^2\,A}{C}$, or $A = \frac{M\,C}{S\,R^2}$, by which formula the area that is necessary to add to the section of the column in order to resist bending stresses shall be found provided the radius of gyration is not materially changed. If the value of R^2 or the square of the least radius of gyration is materially changed, and hence the distance C from the neutral axis of the section to the extreme fibres, a new section is assumed, and the radius of gyration and the distance C obtained, the solution proceeding as before. In application of the above, the following shall be considered as an example. The column section shown in the accompanying figure has a least radius of gyration on the axis X X of 4.85. Determine the area of metal necessary to add to the section in order to resist the bending moment created by the eccentric load W, as shown, provided the allowable unit fibre stress of the material is 15,000 lb.

$M = Wl$; substituting $M = 10{,}000 + 18$ or $180{,}000$ inch-pounds, and $A = \frac{M\,C}{S\,R^2}$, or by substituting $A =$

$$\frac{180{,}000 + 6}{15{,}000 + 23.52}$$

, or 3.8 square inch, which is the area necessary to add to the column section in order to resist the bending moment created by the eccentric load. Since the sectional area of the original column section is 18.94 square inches, the angles and plates will have to be increased to $\frac{7}{16}$ inches in thickness, this makes the area 22.17 inches, giving a gain of 3.23 inches, which will be ample.

The rivet spacing in columns built up of rolled steel plates and shapes shall be carefully considered, and the following shall be observed in conjunction with the subsequent data given on rivets. Use the same size rivet throughout, if possible, as it saves trouble and expense in the shop.

The rivets holding the elements of the column section together shall be spaced, for several feet above the base plate, not further apart than $3\frac{1}{2}$ inches for a $\frac{3}{4}$ inch diameter rivet, and 4 inches for a $\frac{7}{8}$ inch diameter rivet. Likewise the rivets near any splice or beam connection shall be spaced the same. The rivets throughout the length of the column shall be placed at a distance apart, not greater than 6 inches from centre to centre, provided such a distance does not exceed 16 times the thickness of the thinnest outside plate. The distance from the side of a plate or rolled section to the centre line of a rivet shall not be less than one-half the diameter of the rivet plus the sum of one-half the thickness of the plate plus $\frac{1}{8}$ inch; while the distance from the end of a plate to the centre line of a rivet should not be less than the thickness of the plate plus the sum of the diameter of the rivet and $\frac{1}{2}$ inch.

The size rivet to be employed in the construction of columns shall be $\frac{3}{4}$ inch, $\frac{7}{8}$ inch, and 1 inch in diameter. Rivets $\frac{7}{8}$ inch in diameter shall be used if there is more than one cover plate exceeding in thickness $\frac{5}{8}$ inch; if less than this $\frac{3}{4}$ inch diameter rivets will be amply large. Should the thickness of the angles and plates connected be equal to 3 inches, 1 inch rivets shall be used, though rivets of this size should be avoided if possible, as all shope are not fitted up to drive them with facility.

Column connections require the exercise of much judgment and skill in their design and construction to insure good work. A few points shall always, however, be observed; the columns should be so spliced as to be a unit from basement to roof, and all connections shall be rivetted at the shop or in the field with hot wrought-iron or steel rivets; in this manner only can perfect rigidity, which insures against movement by wind pressure, be attained. Riveting shall be much preferred to bolting, as the rivets in driving upset and fill the hole completely, while the bolts are apt to have more or less play, which destroys the rigidity of the connection. It shall be considered advisable to make the

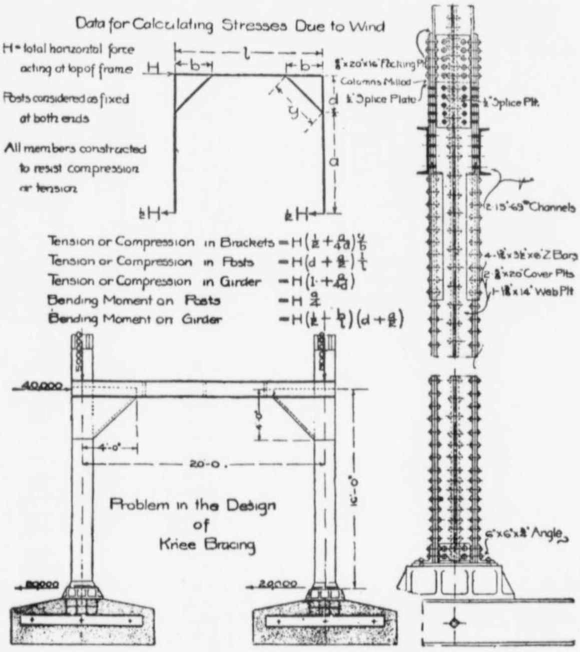


FIG. 16

column splice near the floor beam connections, as it facilitates the erection, the floor beams forming a staging from which the upper tier of columns can be placed, fitted and rivetted. Where it is desired by the architect to run all pipes, electric wiring, etc., along the column and between it and the fireproofing; the connections and splices should be considered to facilitate such designs, and never, on any account, shall the cutting of holes in the bed-plates of columns to allow the passage of such pipes or wires, be tolerated.

When beam, or girder connections, are made to a column sufficient rivets shall be placed therein, that their allowable shearing or bearing value shall equal the end reaction of the girder or beam; that is, assuming that the load upon the girder, uniformly distributed, is sixth tons, or 120,000, the end reaction will be 60,000 pounds. If the allowable value of a 7/8 in. rivet is 4,500 pounds, the number required shall equal 60,000 ÷ 4500 or 14. When the rivets are field driven—that is, driven during erection, 10 per cent to 15 per cent additional rivets shall be used to make due allowance for inferior workmanship. Details of the usual column splices and beam connections shall be as shown in the

The bracing of column connections and floor systems to resist lateral wind pressure shall be accomplished in the several ways enumerated herewith.

First.—The connections of the floor beams and girders to the columns may be so rigid as to provide all the stiffness required; especially may such a system prove available when the floor principals are deep plate girders securely riveted to the columns. This character of construction is amply safe for buildings whose height does not exceed two and a half times their least base. Where deep girders are employed as above mentioned, a height of even three times the length of the least base shall not be considered excessive, provided the exterior walls are well built and of sufficient thickness, or if the building is constructed with substantial interior brick partitions.

Second.—Sometimes the connections and supporting brackets take the form of triangular gusset plates re-enforced on the edges with angles; the whole tending to stiffen the connection materially. This is one of the most usual forms of providing lateral stiffness to the skeleton framework of a tall building. Such a form of brace and connection is shown in detail on the plate, page 66.

Third.—An outcome of the gusset plate system of bracing is the portal system, in which each bay is really a steel arch, the wide columns forming the piers for the support of the arch. This system is used somewhat, especially in the lower stories of extremely high buildings. It is, however, an expensive construction, and the interior of the building, as to the arrangements of partitions, etc., must conform to the location of the portal braces. Where the portal is left open, however, they admit of fine decorative treatment and become advantageous rather than detrimental. Such a system of bracing is shown in detail on page 68.

Fourth.—The system of knee bracing is often used, but is far from being economical, as it produces heavy binding moments in both the columns and horizontal struts. Details of such a system is shown on the plate, page 63.

Fifth.—Sway bracing, which connects the caps of the one column with the foot of the other is undoubtedly one of the most economical in material and makes each pair of columns connected a cantilever bridge truss, its support being at the foundations and its uniformly distributed lead the wind pressure. The great objection to this system is that it interferes materially with the location of door openings, and cannot be used for the interior columns on account of the diagonal braces crossing the window openings. This system of bracing, with details of connection, is shown on page ——.

In calculating the stresses on bracing in buildings, due

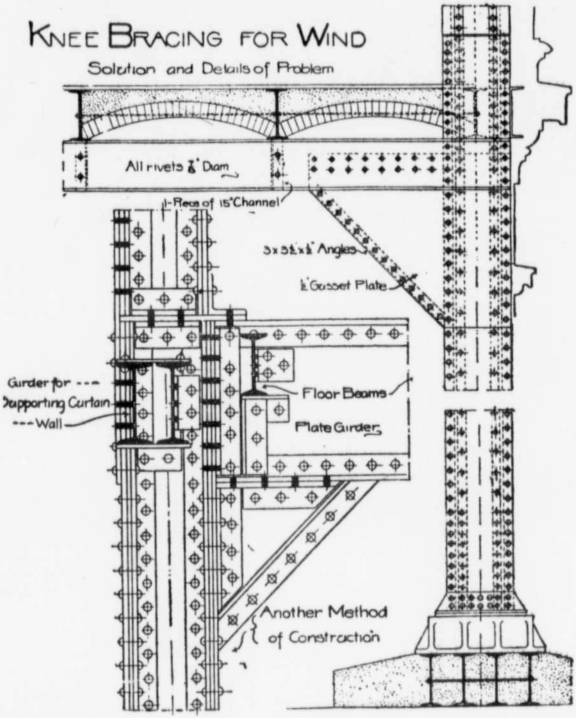


FIG. 14

to lateral wind pressure, the force of the wind shall be taken between 30 and 40 pounds per square foot of vertical surface, as the judgment of the designer shall dictate. The stresses in the wind bracing will be maximum when the direction of the wind is normal to the exterior wall, or parallel to the plane of bracing. In calculating the stresses on any system of bracing instituted for the sole purpose of resisting the lateral pressure of the wind, all rigidity imparted to the building from partitions, stiff floor connections, &c., must be considered an unknown factor, and be disregarded; except that the floor system shall be considered sufficient to transmit the horizontal shear due to the wind. The horizontal shear at any floor level, which, regarding the bracing and the attendant system of columns and floor beams as a cantilever truss supported at the foundation becomes the panel points, shall be taken as the sum of all the forces above the point considered. The force of the wind on any one panel is equal to the horizontal length of the wall surface depending upon the system for support, multiplied by the distance between the floors, half way above and half way below.

It shall not be deemed necessary that every bay in a tall building be braced, only such bays located at these points that the designer shall consider advantageous. Thus, in a narrow building braced crosswise with a single system, it is a good practice to locate the system midway; while if the building has two systems it is well to place them equidistant from the ends. Such symmetry shall be adopted so that the load upon the several systems shall be equalized, and thus any tendency towards twisting eliminated.

The wind bracing shall be so designed that all the stresses are transferred to the foundations, and rigid and adequate connections shall be provided for the accumulated forces at that place.

A factor of safety of at least three shall be adopted in the bracing of tall buildings to resist lateral wind pressure.

The method of determining the stresses on wind bracing differs with the form of bracing employed. The solutions are not exact, owing to the several inde-

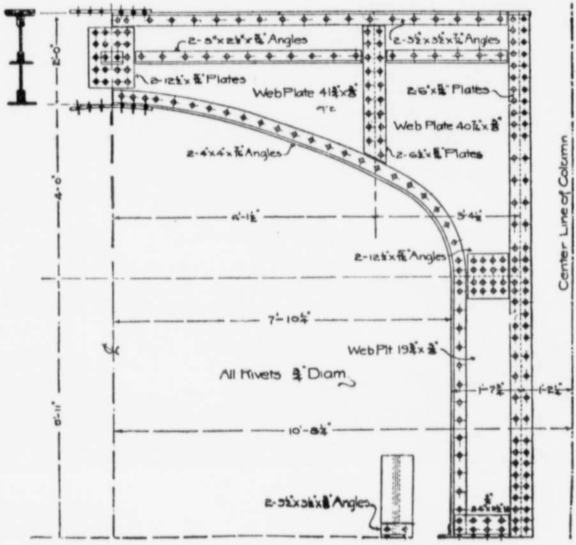


FIG. 18

terminate factors entering therein, and the consequential assumption that must be made. The best practice, however, in figuring the stresses in the several forms of bracing previously treated of is as given on the accompanying plates, and first class buildings should be designed according to the accompanying details.

The heavy girders in building construction, which carry the usual floor beams, are built up of steel plates and angles, and are known under the general term of "plate girders." They are classified according to their cross sections as "single-web girders" or "box girders." The single web-girder, as its name implies, is composed of a single vertical web-plate with angles rivetted back to back along its upper and lower edges, the angles often being re-enforced with plates known as flange plates. The box girder is merely a double single-web girder, that is, it has two or more web plates, the flange or cover plates being sufficiently wide to rivet to the angles attached to all of the web plates.

Each of these girders possesses its specific advantages and disadvantages.

The single-web plate girder is readily constructed, and easily accessible for painting when in place, but lacks lateral stiffness. On the other hand, a box girder is difficult of access for painting, and offers some trouble in construction, but possesses considerable rigidity in a horizontal direction. In comparing the strength of the two girders for weight, it has been found that the box girder is about 10 per cent stronger than the single web girder. This comparison, however, is only true where the girders are unsupported laterally by floor beams or other structural members. When used in a floor system there is little actual difference in their relative strengths.

The elements of a plate girder are the web plate, the upper and lower flanges, which correspond to the chords in a bridge, and the stiffeners or re-enforcing member for the web plate. The web plate is designed to resist the vertical shear upon the girder, while the top and bottom chords take care of the bending moment. Since the top fibres in any simple beam are in compression and the lower fibres of the bottom section are in tension, it naturally follows that the upper flange must be so designed as to resist compressive strength, while the lower flange is subjected to a tensile stress. Theoretically it would be correct to proportion these members, especially for the stress they are called upon to resist; but in actual practice, owing to considerations regarding shop construction and adaptation of material, it is the practice to make both flanges identical, proportioning the flange under tension for the stress to which it is subjected, and making the compressive flange of the same shape and construction.

The first element that is usually considered in designing a plate girder is the web plate. The width of this plate limits the depth of the girder, and it has been found from experience that it is inadvisable to make a web plate girder of a greater span than sixteen times its depth. Some engineers more conservatively adopt twelve times the depth of the girder for the maximum span, while others deem that excessive deflection will not exist, under a load proportioned within the safe unit fibre stress, if a girder has a span twenty times the depth.

The practice of making the minimum depth of the girder $\frac{1}{16}$ of the span is almost universal, and can be adopted with safety. Following such a rule, a girder 40 ft. long would be approximately 2 ft. 6 in., or 30 in. in depth. Having regulated the depth for a girder, which has likewise been found to be the most economical, it is next required to ascertain the minimum thickness of web plate that may be used. When plate girders were first adopted it was the custom to use exceedingly thin web plates, some being as thin as $\frac{1}{4}$ in. or $\frac{3}{16}$ in. these old girders, after many years of service, were cut apart, and it was found that many of the rivet holes had elongated considerably. Gaining by this experience, conservative engineering practice has determined that no web plate shall be less than $\frac{5}{16}$ in. in thickness, and

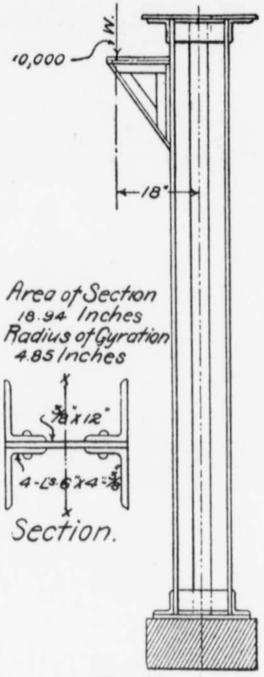


FIG. 19

for the ordinary light girder used in building construction this thickness is commended. The web plate, however, must be proportioned to withstand the maximum vertical shear to which the girder is subjected. The maximum shear in a simple beam is located at the points of support, and is equal to the reaction at the abutment, the gross vertical shear of a beam uniformly loaded being equal to one-half of the load, or $\frac{W}{2}$.

This vertical re-action $\frac{W}{2}$ must be resisted by the shearing value of the web plate directly over the abutment. The entire cross-section of the web-plate is not available at this point, because a certain percentage is cut away in rivet holes, as it is necessary to rivet angle-iron stiffeners through the web plates at this point. The net section of the web plate is therefore equal to the thickness, multiplied by the depth, minus the number of rivet holes in a straight line multiplied by their diameter. When the net section has been obtained in this manner, the shearing value of the web plate may be determined by multiplying the section area by the allowable unit shearing value of the material, which is usually taken at 11,000 pounds per square inch of section.

From these premises we have therefore the formula:—

$(D - nd)tS = \frac{W}{2}$, in which D equals the depth of the web plate, t equals the thickness of the web plate (both in inches), n equals the number of holes through a vertical section of the web plate at the point of support, d equals the diameter of the rivet hole (taken as $\frac{1}{16}$ in. larger than the diameter of the rivet), and S equals the allowable unit shearing value for the material, which, as previously mentioned, is taken at 11,000 pounds for structural steel, while W equals the entire uniform load upon the girder. For the value $\frac{W}{2}$ the gross reaction must be substituted when the girder is unsymmetrically loaded.

By transposition the thickness of the girder may readily be found from the following formula:—

$$t = \frac{W}{2\,S(D - nd)},$$

or, if R equals the gross reaction, the formula may be written as:—

$$t = \frac{R_1}{S(D - nd)}.$$

For example, the maximum shear on a web plant equals 120,000 pounds, the web plate is 48 in. in depth, and through the web plate at point of support is a row of ten rivets $\frac{3}{4}$ in. in diameter, the rivet holes being $\frac{1}{16}$ in. larger, making them $\frac{13}{16}$ in. in diameter; upon substituting these values the formula:—

$$t = \frac{R_1}{S(D - nd)} \text{ equals } \frac{120{,}000}{11{,}000\,(48 - 10 \times \frac{13}{16})} = 27 \text{ in.}$$

In this particular case it is determined by the formula that the theoretical thickness of the web plate should be a little over $\frac{1}{4}$ in., but from the practical considerations before mentioned, which precludes the use of a web plate for any girder less than $\frac{5}{16}$ in. in thickness, this minimum thickness should be used.

It is not sufficient that the web plate of a girder shall be built to resist the shear alone, since there is a tendency for the plate to buckle. This tendency to buckling must be overcome by the use of angles rivetted to the sides of the web plate; these angles known as stiffeners, are spaced at intervals not exceeding the depth of the girder. In no case, however, should these stiffeners be at a greater distance apart than 5 ft. Conservative engineering practice provides that such stiffeners shall be used on all girders at the points of support, and directly under any concentrated load. Other stiffeners must be used when the shearing strain per square inch exceeds the strain obtained by the following formula:—

$$S = \frac{12{,}000}{1 + \dfrac{h^2}{3000\, t^2}}$$

in which S equals the allowable shearing strain per square inch, h the depth of the web between the flange angles of the girder, while t equals the thickness of the web plate, the values being in inches.

For example: A girder 30 in. between the flange angles, and provided with a $\frac{5}{16}$ in. web plate, sustains a vertical shear of 100,000 pounds. By substituting in the formula just given—

$$S = \frac{12{,}000}{1 + \dfrac{30 \times 30}{3000 \times .312 \times .312}} = 2940 \text{ pounds.}$$

The shearing stress upon the section of the web is found to be about 10,000 pounds, which is in excess of the allowable stress determined by the formula, consequently stiffeners will be required for the girder. These stiffeners should in every case extend to the under side

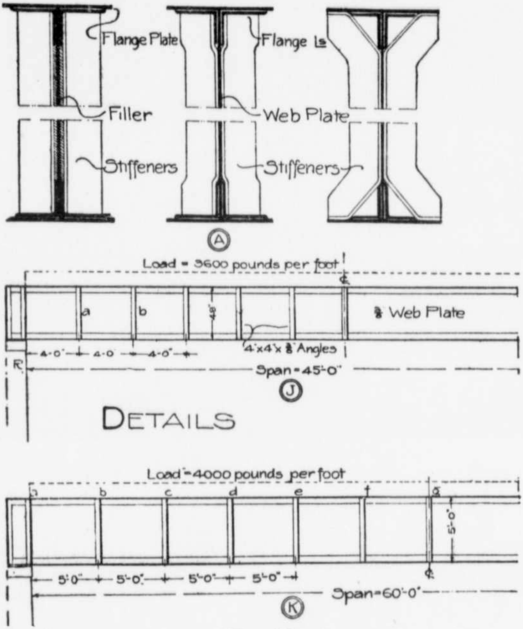


FIG. 20

of the flange angles; that is, they should cross the vertical leg of the flange angles, as shown at A on Figs. 20 and 21. From these figures it will be observed the angle stiffeners may be made straight, and a packing piece introduced beneath them so that the space between the vertical legs of the flange angles will be filled flush, and the stiffeners will take a firm bearing; or they may be rivetted directly next to the web plate, and swaged outward at the ends so as to clear the flange angles. In

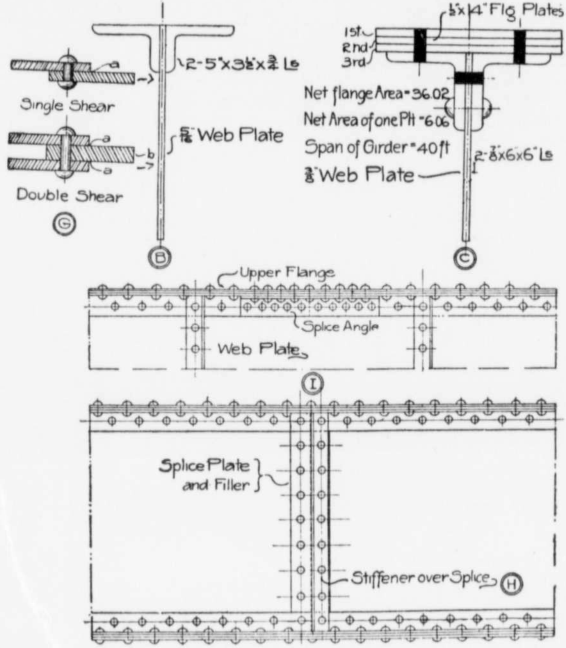


FIG. 21

some cases the ends of the stiffeners are bent outward at an angle of about 45°, so as to clear the flange angles entirely, as shown in the figure. In proportioning the end stiffeners over the abutments, they should be considered as columns, and should be proportioned to take the entire load coming upon the support. Sufficient rivets should be placed through the end stiffeners to transfer the load to the web of the girder. The strain upon the end stiffeners should not, in any case, exceed 15,000 pounds per square inch of cross section, and it should be preferably less than this.

Where the stiffeners are used under a concentrated load, they act to prevent the buckling of the web, and must be so designed that they will transmit the entire concentrated load to the web through the rivets, and through their end section, which takes a bearing upon the upper and lower flanges. A sufficient number of rivets should be provided wherever possible, in order to transmit the entire concentrated load.

The top and bottom elements of the girder which, as before mentioned, correspond to the upper and lower chords of a bridge truss, and which are known as the flanges, comprise the flange angles and the flange plates, and many engineers in figuring the necessary flange section include in the sectional area of the flange 1-6 of the web plate. Where 1-6 of the web plate is thus considered, it is bad practice to splice the plate near the centre of the girder when it is uniformly loaded, for at this point the greatest bending moment takes place. The entire sectional area of the flanges is not available in resisting the bending stress, for the reason that all rivet holes found in a single vertical section must be deducted from the gross area in order to get the net area of metal in the flange. In order to determine the required sectional area in the flange it is necessary to determine the greatest bending moment upon the girder; this bending moment is usually stated in foot-pounds. By dividing the bending moment in foot-pounds by the product of the depth of the girder in feet and the allowable unit fibre stress, the required flange area will be determined. This rule can be stated in the following formula, where

$$S\,a = \frac{M}{S \times D}$$

in which S a equals the net sectional area of the flange required, M the bending moment in foot-pounds, S the allowable unit fibre stress of the metal, usually taken at 15,000 (though by less conservative engineers considered safe at 18,000, or even 20,000 for building work), and D equals the depth of the girder in feet. For example: A plate girder having a depth of 30 in. and a span of 40 ft. is required to support a uniformly distributed load of 2000 pounds per lineal foot.

The load upon the girder equals 80,000 pounds; the bending amount equals:

$$\frac{W\,L}{8} = \frac{80{,}000 - 40}{8} = 400{,}000 \text{ foot-pounds.}$$

Upon substitution in the formula—

$$S\,a = \frac{400{,}000}{15{,}000 \times 2.5} = 10.66,$$

or the required net area in the flange in inches. The gross area in the flange must be considerably more than this, because there are at least, even if no flange plates are used, one rivet out of each flange angle, and one rivet hole out of 1-6 of the web plate; assuming that the flange section of the girder proposed is as shown at B in Fig. 21, its gross section and area is as follows:

1-6 of web plate equals $.3125 \times 5 \ldots\ldots = 1.56$ sq. in.
2 flange angles, $5 \times 3\frac{1}{2} \times \frac{3}{4}$ in. @ $5.98 = 11.96$ sq. in.

Total gross area.................. = 13.52 sq. in.

The deduction for rivet holes in this flange, provided the rivets used are $\frac{3}{4}$ in. in diameter, will equal—

2 holes at $.8125 \times .75 = 1.22$ sq. in.
1 hole at $.8125 \times .312 = .25$ sq. in.

1.47 sq. in.

which, when deducted from the gross area of the flange plate, equals 12.05 sq. in. in the net area of the flange plate. Since this is in excess of the required area, the section assumed is ample for the girder in question.

If the width of the top flange of any girder is not at least equal to 1-20 of the span, the section of the top flange should be increased, provided the girder is not held against lateral deflection either by floor beams or secondary girders. The formula commonly used by conservative engineers to determine the necessary increase for the top flange is as follows:—

$$A^1 = A \quad I + \frac{l^2}{5000}$$

in which A_1 equals the net area required in the top flange when the girders is unsupported laterally; A equals the net area required in the top flange, the girder being supported laterally, while l equals the span of the girder in inches, divided by the width of the flange in inches.

If in designing the girder in the preceding example the flange area as assumed had not been found sufficiently great, then flange plates would have been used. It is usually preferable to make such flange plates from $\frac{3}{8}$ in. to $\frac{5}{8}$ in. thickness, and if more area is required than can be gained by the use of one flange plate, a number are usually provided; in very heavy girder construction as many as four or five flange plates are often used.

As it requires considerable time to calculate the necessary flange area of a girder, it is often convenient to consult a table which will give upon inspection, the necessary information. A convenient means for obtaining the net area necessary in the flange of a plate girder is given in the table shown.

In using this table it is necessary to note the load, span, and depth. When these values have been decided upon, multiply the coefficient given in the table by the uniformly distributed load. The load is figured in tons of 2000 pounds. The result obtained will be the net area in square inches required in each flange, the safe unit fibre stress being 15,000 pounds. For example:— A plate girder has a span of 30 ft., and its depth from centre to centre of the flange angles is 24 in., while the uniform load the girder is required to sustain equals 1500 pounds per lineal foot of girder.

The entire load on the girder equals 1500×30, or 4500 pounds. The product of the load in tons by the coefficient from the table for a girder of the dimensions stated equals $22\frac{1}{2} \times .25$ or 5.62 sq. in., the required flange area.

Plate, or built-up girders, are usually used to support a uniformly distributed load, and when they sustain this character of load they are subject to the greatest bending moment at the centre of the span. This bending moment diminishes to zero at the supports. It is consequently unnecessary to have the full flange section extend the whole length of the girder, and for economical considerations where one or more flange plates are used, they can be stopped some distance from the supports without affecting the strength of the girder in the least. The outside flange plate is shortest, while the flange plate next to the angles is the longest. This latter, in order to stiffen the girder laterally, is usually extended the full length of the girder, though as far as the bending moment is concerned, it is not needed after a certain distance from the centre. The length of the flange plates is readily determined by the formula—

$$L^1 = 2 + L \times \sqrt{\frac{A^{11}}{A}}.$$

In this formula L^1 equals length of the flange plates under consideration, and which, for a uniformly distributed load, are symmetrically arranged each side of the centre of the girder; this length is determined in feet; L equals the span of the girder from the centre of the abutments, likewise in feet; A^{11} equals the total net area of the flange from the outside and including the plate in question, while A equals the entire net area of the flange. Applying this formula to the plate girder designed as designated in C, on Figure 21, the following is true for each of the three flange plates.

$$\text{Length of first plate} = 2 + 40\sqrt{\frac{6.06}{36.62}} \text{ or } 18.4 \text{ ft.}$$

$$\text{Length of second plate} = 2 + 40\sqrt{\frac{12.12}{36.62}} \text{ or } 24.8 \text{ ft.}$$

$$\text{Length of third plate} = 2 + 40\sqrt{\frac{18.18}{36.62}} \text{ or } 30 \text{ ft.}$$

When a girder supports a uniformly distributed load, or even a number of loads, or both, the length of the flange plates may be found readily by several interesting graphical methods. While a formula has been given which is readily calculated, and is, possibly, more convenient to use than the graphical method for the same system of loading, it is not possible by it to obtain the most economical length of the flange plate when a girder is loaded with both uniformly distributed, and concentrated, loads; in fact the graphical method for such a case is the only one that can be used. Many engineers prefer to use the dia-grammatic or graphical methods in all cases and for the solution of all problems, for the reason that while not nearly so accurate, the possibility of a considerable mistake is very much less. The bending moment upon a girder, when the load is uniformly distributed, is represented graphically by the parabola, and the basis of the method for determining the length of the several flange plates is founded upon the construction of this line. In Figs. 22 and 23, the line a b is laid off to any scale, equal to the span of the girder; at the centre is erected the line c d, upon which the net flange area is laid off to scale; the horizontal line e f is now drawn, and the rectangle a, b, f, e is thus completed. Divide the line e a into any number of equal spaces, as shown at 1, 2, 3, 4, etc., and likewise divide the line a d into the same number of equal spaces, as designated at g, h, i, etc., draw from these points g, h, i, j, k, etc., the vertical ordinates as shown, and from the points 1, 2, 3, 4, etc., upon the line a e extend lines converging to the point c; at the intersection of the oblique lines with their corresponding vertical lines, mark points as at m, n, o, etc., and through these points draw in the curve of the parabola. Now assume that

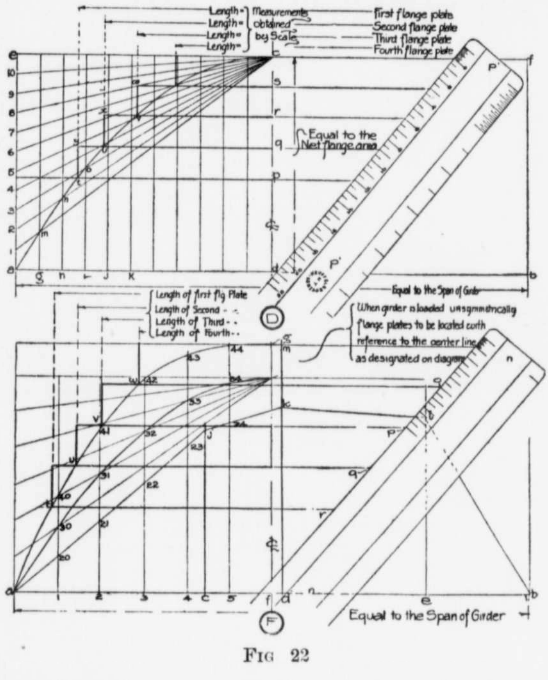


FIG 22

the girder for which the diagram is being drawn has a net flange area of 21 sq. in., and making up this area there are four flange plates, each having an area of 4 sq. in., and two flange angles having a net area of 9 sq. in. It is now necessary to divide the line e d into such portions as will graphically represent the net flange area of the plates and of the flange angles; for instance, each plate will be in length equal to 4-21 of the length e d, and the two flange angles will equal 9-21 of this line.

The most convenient way to lead out this line to the true proportions is to place the scale across the figure at any angle until, in this case, 21 divisions are contained between the two horizontal lines e f and a b; this scale is shown at p^1p^1, and the method is sufficiently clear from the figure. Through the points p, q, r, s, draw the horizontal lines as shown, and where these lines intersect the parabola, as at t, u v, draw upwards vertical lines as designated at v, w, u, y, and ty; the length of the several flange plates may then be readily determined by sealing the distances of y,x, and w respectively from the central line e d. This will give one-half the length of the flange plate, theoretically; one foot should be added to this length in order to allow for rivetting. In this case the theoretical lengths of the first, second, third, and fourth flange plates are found to be as designated in the figure. The entire theoretical length of the flange plate in each case being twice this distance, measured for the respective plates from the centre line e d, plus the 2 ft. added to each of the plates for rivetting.

The diagram for determining the length of the flange plates, where the girder supports several concentrated loads, is similarly constructed as shown in E, Fig. 23, the method pursued being practically the same. In this figure lay out as before the line a b to scale, and locate upon it the position of the several concentrated loads, as shown at c, d, e and f. From these points extend upwards the vertical ordinates as designated, making them equal in length to the net flange area required at the several locations upon the girder in order to sustain the load. In this way will be obtained the points g, h, i, j; having obtained these points, connect a and g, g, and h, h, and i, i and j, and j, and b; through h draw any horizontal

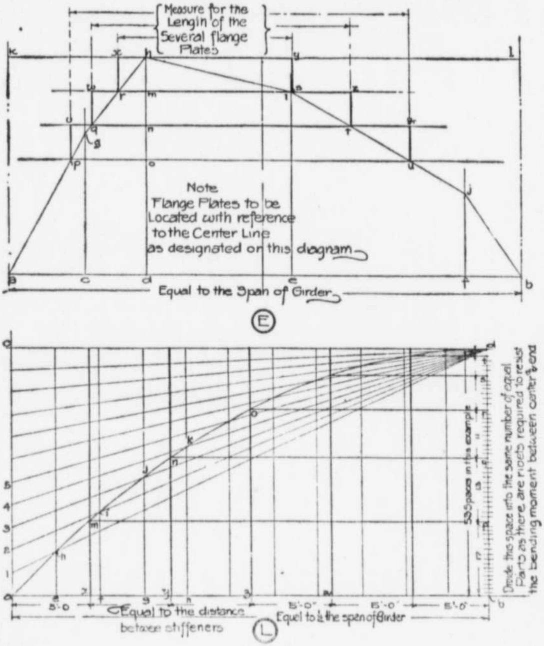


FIG. 23

line, and pursue the same method as described in connection with Fig. 22. Lay off between the horizontal lines a b and k l, the points m, n, and o; the distances hm, mn, no, and od represent the net areas of the three flange plates and the two flange angles respectively, laid off upon the line a h, which represents the greatest net area required in the girder. Through the points m, n, and o just obtained, draw the horizontal lines as shown, and where they intersect the outside lines ag, gh, hi, etc., of the irregular polygon, as at the points p, q, r, s, t, and u, draw upwards vertical lines extending to the next horizontal line above; these lines are represented in the figure pv, qw, rx, sy, tz, and u &, and the distance of the first flange plate may be found by scaling the line xy with the same scale to which the span was marked off on the line ab; the length of the second plate is scaled from wz, while the third is found by measuring the line v, etc.

As explained in connection with the previous diagram, it is necessary to add 2 ft. to the end of each flange plate in order to compensate for the rivetting required.

The method for obtaining the length of the flange plates in a girder, when it is required to support both a uniformly distributed load and several concentrated loads along its length, is a combination of these two methods just given, and the diagram for obtaining the flange plates under such conditions is designated in Fig. 23. Let it be assumed that the girder a b (that is, one having a span to scale equal the line a b) is loaded with a uniformly distributed load, and three concentrated loads located at the points c d and e. On the centre line f g lay off a distance h i equal by scale, as in the first method, to the net flange area required for the uniformly distributed load, and by the system described in connection with Fig. 22, and which is shown constructed with light lines, to the left of the centre line, draw the parabola, which represents graphically the bending moment upon the girder due to the distributed load; upon the ordinates from the points c, d, and e lay off the points j, k, and l, and connect, as described in Fig. 23, the points a j, j k, k l, and l b, and the resulting polygon will be the representation of the bending moments due to the concentrated loads. The distances c j, d k, and e l, as in Fig. 23, lay off equal to the net flange area required to resist the bending moments accruing from the concentrated load *only* at these points; now from 1, 2, 3, 4, etc., extend the ordinates as shown, and upon the first ordinate lay off 1-20 from the point 30, obtaining the point 40; likewise lay off the distance 2-21 from the point 31, obtaining the point 41; proceed likewise with 3-45, and thus until the points 40, 41, 42, 43, etc., have been obtained, and through these points draw an irregular curve as shown. The distance from the highest point m to the horizontal line a b will represent to scale the net section required for the greatest bending moment due to both the uniformly distributed and the concentrated loads. Through this point m draw the horizontal line as designated, and with the scale as shown at n n, and by the method previously described in connection with Fig. 22 lay off the spaces o, p, q, r, representing to scale the net area of the flange angles and flange plates, and the horizontal lines drawn through o, p, q, r, will divide the line m d into portions which truly represent the net section of the flange plates and the flange angles. Where these lines, as previously described, intersect the outside polygon as at t, u, v, w, draw in the vertical lines as shown. By scale the theoretical length of the flange plates are found as designated on the figure, although the fourth plate, at least in this case, would extend throughout the length of the plate girder. It will be noted in every case that the flange angles are required the full length of the girder. If it is desired to include 1-6 of the web plate in the flange area this allowance must of course be made in marking off the divisions between the upper and lower horizontal lines. The same consideration applies to all of the methods described.

After the general dimensions of the girder have been determined, and the sizes for the flange angels and plates, together with the stiffeners decided upon, it is necessary to obtain the rivet spacing throughout the girder. The sizes of rivets employed in structural work in connection with building construction range from $3/4$ in. to 1 in. in diameter, most of the work being put together with $3/4$ in. diameter rivets. The rivets are driven hot, and upon contracting in cooling they clamp the several plates of the structure firmly together. It is a much disputed question whether this clamping effect, which produces certain frictional resistances between the plates, shall be considered in calculating the rivetting, or whether the shearing resistance of the rivets and bearing value of the plate alone should be taken into account.

This clamping effect, or frictional resistance of the plates, is, however, such an indeterminate factor, that it can hardly be considered in the design of steel structure. The only certainty of strength in a rivetted joint or connection lies in the shearing value of the rivets, or the bearing value of the plate. The rivetted joint may fail in two ways—by the rivet shearing, or by the plate around the rivet hole crushing or crippling. When the plate is thin, and the rivet of considerable diameter, there is liability of the rivet holes stretching or elongating through the crippling of the plate, but when the plate is of considerable thickness the rivet will shear before the plate will be affected. In considering the strength of any joint or rivetted connection, it is therefore necessary to consider both the shearing resistance of the rivet and the bearing value of the plate adjacent to the rivet hole. When two plates are lapped and held together by a rivet, the rivet is said to be in single shear, because only one section of the rivet is required to resist

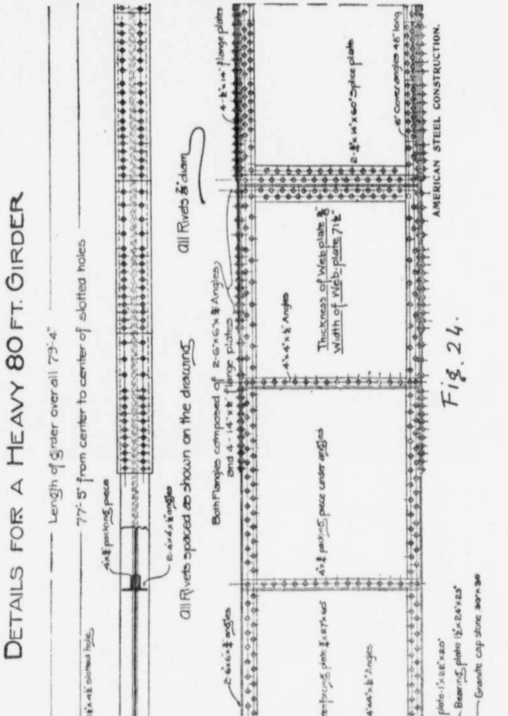


Fig. 24.

the tendency of the plates to cut the rivet at their junction. When, however, there are three plates held together by a rivet, two having a stress tending in one direction, resisted by the other or central plate, then there is a tendency to shear the rivet through the section along two lines, and the rivet is said to be in double shear. At Fig. 24, are shown rivets in single and double shear respectively.

Observing these figures, the plates having the rivet in single shear bear upon the rivet at a a, and are said to be in ordinary bearing, while the plate b having the rivet in double shear is said to be in web bearing at a a. A rivet in double shear, as will be observed from a study of figures, is twice as strong as one in single shear, and it is usual to allow that a plate in web bearing is one and a half times as strong as a plate in ordinary bearing.

The commercial strength of structural steel is tensile, and it is usual to base the shearing and compressive stress upon the allowable tensile; for instance, conservative practice usually considers the allowable bearing value of the plate in ordinary bearing as equal to about $\frac{13}{16}$ the allowable tensile stress, while the allowable shear upon the rivets is commonly taken at about one-half the allowable bearing value, so that the following values are usually taken in practical work:—

Character of Work.	Shearing	Ordinary Bearing.
Iron rivets, railroad bridges	6000	12,000
Iron rivets, highway bridges and buildings	7500	15,000
Steel rivets, railroad bridges	7500	15,000
Steel rivets, highway bridges and buildings	9000	18,000

All of these values being the unit allowable stress, or the stress per square inch of shearing or bearing surface.

The following table gives the value for rivets of different diameters and for plates of different thicknesses—

RIVET VALUES FOR SHEARING AND BEARING.

Diameter of rivet.	Area of rivet.	Single shear 6000.	Bearing value at 12,000 for different thicknesses of plate.							
			¼	5-16		7-16	½	9-16	⅝	11-16
⅜	·110	660	1120							
½	·196	1180	1500	1880	2250					
⅝	·307	1840	1860	2320	2790	3250	3720			
¾	·442	2650	2250	2810	3370	3940	4500	5060		
⅞	·601	3610	2630	3280	3940	4590	5250	5910	6560	7220
1	·785	4710	3000	3750	4500	5250	6000	6750	7500	8250

Diameter of rivet.	Area of rivet.	Single shear 7500.	Bearing value at 15,000 for different thicknesses of plate.							
			¼	5-16		7-16	½	9-16	⅝	11-16
⅜	·110	830	1410							
½	·196	1470	1880	2340	2810					
⅝	307	2300	2340	2930	3520	4100				
¾	·442	3310	2810	3520	4220	4920	5630	6330		
⅞	·601	4510	3280	4100	4920	5740	6560	7380	8200	9020
1	·785	5890	3750	4690	5620	6560	7500	8440	9380	10,310

Diamater of rivet.	Area of rivet.	Single shear 9000.	Bearing value at 18,000 for different thicknesses of plate.							
			¼	5-16	⅜	7-16	½	9-16	⅝	11-16
⅜	·110	990	1680							
½	·196	1770	2250	2820	3370					
⅝	·307	2760	2590	3480	4180	4870	5580			
¾	·442	3970	3300	4210	5050	5910	6750	7590		
⅞	·601	5410	3940	4920	5910	6880	7870	8860	9840	10,830
1	·785	7060	4500	5620	6750	7870	9000	10,120	11,230	12,370

In considering the riveting of a plate girder it is advisable to introduce in the end angles—that is, the stiffeners over the abutments—sufficient rivets to resist by their shearing, and by the bearing value of the plate, the entire reaction or shear on the girder at this point. For example, a girder 48 in. in depth is loaded so that the reaction at the abutment is 200,000 pounds, the web plate being $\frac{5}{16}$ in. in thickness, and the rivets $\frac{3}{4}$ in. in diameter, the angles stiffeners being $\frac{3}{8}$ in. in thickness. Both the ordinary bearing of the angles and the web bearing of the web plate is greater than the shearing strength of a $\frac{3}{4}$ in. rivet; but the web bearing is not as great as the double shear which necessarily exists upon the rivets; in consequence the weakest point of the construction is in the web bearing of the middle plate, which figured on a basis of a unit bearing stress of 15,000 pounds, one and a half times the value given in the table for a $\frac{5}{16}$ in. plate in the second section of the table. The least value for each rivet is then found to be equal to 5280. Dividing the reaction 200,000 pounds by 5280, gives approximately 38 rivets, or 19 rivets in each pair of stiffeners. The pitch of the rivets will then be (provided the distance between the extreme row of rivets in the upper and lower flange angles is 44 inches) 44 in. $\div$ 19, or 2.31 inches.

The spacing of the rivets in the stiffener throughout the girder is more a matter of judgment than of calculation, though some engineers deem it advisable to place enough rivets in the stiffeners to take care of the ordinary shear at the particular point at which the stiffeners occur. This is not necessary, however, and the usual rule is to space the rivets at the maximum pitch for a compressive member, namely, sixteen times the thickness of the thinnest outside plate, providing such a pitch does not exceed 6 inches. It is not always possible to secure wide web plates of sufficient length so that they may be introduced in the girder in one piece; it is therefore necessary to splice these web plates, and it is usually arranged to have these splices come at the point of least shear, which, in a girder uniformly loaded, is at the centre of the span. In Fig. 21, is shown a splice for a web plate, and it will be noticed from this drawing that the splice plate does double duty in holding the web plate together, and in acting as a filler for the stiffener, the rivets in the stiffener being so arranged as to assist in the splice of the web plate. Like the web plate the flange angles must sometimes be spliced, and when this is necessary a smaller sized angle or bent plate is usually introduced, as shown at a in Fig. 20. It is considered good practice to arrange the splice so that no two will come opposite—that is, in the upper flange the splice of the angle on one side is at one end of the girder, while the splice of the angle on the other side is at the other end of the girder. Few rivets are required in the splice of the angles in the upper flanges either when the cover plates are spliced or the flange angles, because this portion of the girder is in compression, and the only use of the splice is to keep the member in alignment, so that it may take the compression directly.

When, however, it is required to splice the lower chord of a plate girder, either the cover plates, the flange angles, or the web plate, if 1-6 of its depth has been included in the design of the flange it is necessary to introduce sufficient rivets so as to realize in their resistances the entire tensile strength of the flange in question. In riveting the flange plates to the flange angles, it is the practice to introduce enough rivets in the end of the plate to realize the tensile strength of the net section of the plate. Take, for example, a plate 12 in. wide by $\frac{3}{8}$ in. thick, punched through on a straight line with two $\frac{7}{8}$ in. holes for rivets; the net section of this plate is equal to $12 \times .375$, or 4.5, and if an allowable tensile strength of 15,000 pounds has been considered, the strength of the plate will equal $4.5 \times 15{,}000$, or 67,500. Supposing that the value of one rivet through this plate is equal to 4800 pounds, there will be required in the end of the plate $67{,}500 \div 4800$, or approximately 14 rivets, will be placed in the end of the plate, pitched about $2\frac{1}{2}$ in. or 3 in. from centre to centre. The balance of the rivets in this plate will be spaced at the maximum pitch, except where such rivetting coincides with the end of another plate in which the rivets are spaced as in this plate, at the end.

The calculation for the number of rivets in the vertical leg of the flange angles and through the web plate is somewhat more complicated than for the flange plates, stiffeners, or splice plates. There are three methods pursued in practice, and also a method by which the number may be determined graphically. The latter is especially to be commended on account of its simplicity.

The simplest rule for determining the number of rivets through the vertical leg of the flange angle is to space the rivets between any two stiffeners so that there will be a sufficient number to resist the vertical shear that would occur on the point on the girder where the stiffener nearest to the abutment is located. For example, a girder is loaded as designated at J, Fig. 20, and it is desired to ascertain the number of rivets required between the points a and b; the reaction or, R, equals one-half of the load, or $\frac{1}{2}\ (45 \times 3600) = 81{,}000$ pounds; the shear at the point a in consequence equals 81,000, minus the load between R and the point a, which is equal to 14,400 pounds, or 66,600 pounds. The distance between a and b, equal to the depth of the girder, is 48 inches, and 66,600 pounds divided by 48 equals 1387 pounds resistance required per lineal inch of the flange angles.

From the sketch the web plate is $\frac{3}{8}$ in. thick, and the flange angles, which are $\frac{1}{2}$ in. thick, are rivetted with $\frac{3}{4}$ in. diameter rivets. The values of a $\frac{3}{4}$ in. diameter rivet in such an instance, when a unit stress of 18,000 pounds is assumed as the safe working value of the material, is as follows—

Web bearing 7575
Double shear 7940

so that the least resistance of one rivet is in the web bearing of the flange angles. Having obtained this value, which is 7575 pounds, and knowing the number of pounds resistance required in each running inch of the flange angles, it is only necessary to divide 7575 by 1387 and obtain the pitch of the rivets, which, in this case, equals 5.46 inches.

Since this dimension falls within the maximum for a compression member, which is sixteen times the thickness of the thinnest outside plate, provided such does not result in a number greater than 6 inches, the pitch just found may be safely employed. This rule may be stated more explicitly by the following formula:—

$$P = \frac{V\,D}{S},$$

in which P equals the pitch of the rivets between the two stiffeners; V, the least resisting value for one rivet; D, the distance between the two stiffeners when it equals the depth of the web plate, and S, the vertical shear at the stiffener nearest the abutment. While this rule is only approximately correct, it gives at least a minimum pitch, and is well within the safety limits.

Another method which is frequently employed to determine the pitch of the rivets in the vertical leg of the flange angle is to calculate the increment of stress for the several points throughout the length of the girder, and to proportion the pitch of the rivets accordingly. The increment of stress, or the amount of stress by which the stress in the flange angle is augmented at each point in its length, and which is the stress that is transmitted to the flange from the web, is obtained by dividing the vertical shear at any point on the girder by the depth of the girder in inches. The increment of flange stress may be determined by the following formula, in which S i equals the increment of flange stress desired; S, the maximum shear on the girder at the point in question in pounds, and D, the depth of the girder in inches:—

$$S\,i = \frac{S}{D}.$$

When the increment of stress has been obtained, it is only necessary to divide by the maximum value of the rivet in order to obtain the pitch of the rivets adjacent to the point in question.

Assume that it is desired to obtain the pitch of rivets between the several points of the girder marked a, b, c, d, e, etc., in the Fig. 20. The load on the girder is 4000 pounds per lineal foot, and the vertical shear at the several points is as follows:—

$$\begin{aligned} a &= 120{,}000 \\ b &= 100{,}000 \\ c &= 80{,}000 \end{aligned}$$

d = 60,000
e = 40,000
f = 20,000
g = 0

Then the increment of stress at the several points may be designated as is tabulated below:—

a = 120,000 ÷ 60 in. = 2000
b = 100,000 ÷ 60 in. = 1666 2-3
c = 80,000 ÷ 60 in. = 1333 1-3
d = 60,000 ÷ 60 in. = 1000
e = 40,000 ÷ 60 in. = 666 2-3
f = 20,000 ÷ 60 in. = 333 1-3
g = 0

From this data just obtained, the number of rivets which may safely be used between the several points on the girder may be obtained by dividing the several increments by the least value of the rivet, which may be assumed at 6750. With this value for one rivet the theoretical pitch of the rivets will be as follows:—

Between a and b = 6750 ÷ 2000 = 3.37 in.
Between b and c = 6750 ÷ 1666 2-3 = 4.05 in.
Between c and d = 6750 ÷ 1333 1-3 = 5.06 in.
Between d and e = 6750 ÷ 1000 = 6.75 in.
Between e and f = 6750 ÷ 666 2-3 = 10.12 in.
Between f and g = 6750 ÷ 333 1-3 = 20.24 in.

For practical reasons it would be advisable to make the pitch from a to b equal to as near $3\frac{3}{8}$ in. as possible, and the pitch from b to c equal to 4 in., and from c to d equal to 5 in., while throughout the remaining panels the pitch must not be greater than the maximum pitch, or 6 in.

Besides the horizontal stress existing between the flange angles and the web plates, there is a vertical stress when the load rests upon the top or bottom flange, and this vertical stress is sometimes provided for in the rivet spacing. This tendency to shear the rivets in a vertical direction is uniform throughout the girder, and is equal for each lineal inch in the length of the girder to the load per foot divided by 12.

The vertical load per inch in the previous example equals therefore 4000 pounds, the load per lineal foot, divided by 12, or 333 1-3 pounds.

The increment of horizontal stress in the same example between a and b was found to equal 2000 pounds. There exists in consequence the vertical component of 333 1-3 pounds and the horizontal component of 2000 pounds of an oblique force, which is the resultant, and which determines the pitch of the rivets. This resultant is found in the same manner as the hypothenuse of a triangle—that is, according to the formula—

$$R = \sqrt{x^2 + y^2},$$

in which **R** equals the resultant required, **x** the horizontal increment of stress, and y the vertical stress. Upon substituting in this formula—

$$R = \sqrt{2000^2 + 333\ 1\text{-}3^2},$$

or 2027 pounds, and this divided by the value of one rivet as previously assumed to equal 6750 gives a theoretical pitch between a and b equal to 3.34 in. instead of 3.37 in., as obtained when the vertical stress on the rivet is neglected. The difference in the two pitches is usually so slight that it is the common practice to neglect this vertical stress entirely. The method of calculating the pitch of the rivets in the vertical leg of the flange angles by the bending moment is probably the simplest, and is the one commonly employed. The bending moment at any point on the top or bottom flange of a plate girder may be readily resolved into the compressive or tensile stress in that flange by dividing by the depth of the girder in feet. If, then, the stress in the flange due to the bending moment at that point is divided by the allowable value for a rivet, the result will be the number of rivets required throughout the flange angles from the point in question to the nearest abutment, and the theoretical pitch of the rivets may readily be found by dividing the required number of rivets into the distance in inches from the point in question to the first abutment stiffener.

The above method or rule, may be conveniently stated by the following formula:—

$$P = \frac{d_1 V D}{M_1}$$

In this algebraic statement P equals the theoretical pitch of the rivets from any point on the length of the girder, D_1 equals the distance in inches at which this point is located from the stiffener at the abutment, V the allowable working value for one rivet, D the depth of the girder in feet, and M_1 the bending moment on the girder at the point in question in foot pounds. For example, there occurs in the girder designated at K, Fig. 20, a bending moment at the centre of the span, or at g equal to 1,800,000 foot pounds, and since the girder is 5 ft. in depth, and the value of one rivet in the flange equal to 6750 pounds, while the distance from g to a is 360 in.:—

$$P = \frac{360 \times 6750 \times 5}{1{,}800{,}000}$$

or 6.75 in., which, as it exceeds the allowable pitch for a compression member, will have to be reduced to 6 in., the safe limit.

While the bending moment is greatest at the centre of the span when a girder is uniformly loaded, the accumulation of stress for each lineal unit is greatest towards the supports, so that though only 53 rivets are required in the above example between the point f and the abutment, they should be, to fulfil theoretical requirements placed closer together towards the abutment and spread further and further apart as they advance towards the centre of the girder.

In order to determine what the pitch should be between each set of stiffeners, or between the points a b, b c, c d, d e, e f, and f g, it is necessary to calculate the bending moment at each of the points b, c, d, e, f, and g. Upon making the necessary calculations, these values are found to be as follows:—

Bending moment or M, at b = 550,000
Bending moment or M, at c = 1,000,000
Bending moment or M, at d = 1,350,000
Bending moment or M, at e = 1,600,000
Bending moment or M, at f = 1,750,000
Bending moment or M, at g = 1,800,000

By substituting these values in the formula—

$$P = \frac{d_1 \, V \, D}{M_1},$$

the following data for the pitches between the several stiffeners is obtained:—

$$\text{Pitch between a and b} = \frac{60 \times 6750 \times 5}{550{,}000} = 3.68 \text{ in.}$$

$$\text{Pitch between b and c} = \frac{120 \times 6750 \times 5}{1{,}000{,}000} = 4.05 \text{ in.}$$

$$\text{Pitch between c and d} = \frac{180 \times 6750 \times 5}{1{,}350{,}000} = 4.50 \text{ in.}$$

$$\text{Pitch between d and e} = \frac{240 \times 6750 \times 5}{1{,}600{,}000} = 5.06 \text{ in.}$$

$$\text{Pitch between e and f} = \frac{300 \times 6750 \times 5}{1{,}750{,}000} = 5.78 \text{ in.}$$

$$\text{Pitch between f and g} = \frac{360 \times 6750 \times 5}{1{,}800{,}000} = 6.75 \text{ in.}$$

These several pitches can be modified to suit the practical requirements of the design of the plate girder.

The graphical method for determining the pitch of the rivets in the vertical leg of the flange angles, while not as exact as the other method, gives results that are sufficiently accurate for all practical purposes. By means of a single diagram, which is general in its scope, the pitch of the rivets in any plate girder uniformly loaded may readily be determined with but little calculation, and with an expediency that is often convenient. The principal of the graphical method for determining the pitch of the rivets is practically the same as that described in connection with the graphical determination of the length of the flange plate. Proceed with the diagram shown in Fig. 21. This diagram has been drawn to determine the rivets for the girder of the dimensions shown in Fig. 20, and loaded, as described previously, with a uniformly distributed load of 4000 pounds per lineal foot. In considering the diagram, lay off the distance a b, equal to one-half the span, and to any scale; assume any height, such as a c, and draw the rectangle a, c, d, b; divide the distance a c into any number of equal parts, and the distance a b into the same number of parts; draw the lines 1 d, 2 d, 3 d, etc., and the ordinates from e, f, g, etc.

Where the several lines intersect, as at h i and j. etc., draw in the parabola as designated. Before proceeding with the diagram further, calculate the gross bending moment on the girder, which is equal in this case to

$$\frac{W\,L}{8}, \text{ or } \frac{4000 \times (60 \times 60)}{8}, \text{ 1,800,000 foot-pounds.}$$

This amount divided by the depth of the girder, which is 5 ft. will give 360,000 pounds, the horizontal stress in either flange. Having obtained this value it is next necessary to determine the entire number of rivets required from the centre of the girder towards the abutments through one pair of flange angles. Since the value of one rivet equals 6750, the total number of rivets required will equal approximately 53; having obtained the number 53, divide the distance in the diagram b d into 53 equal parts, which may readily be done by laying the scale across the diagram until 53 divisions are included between two horizontal lines a b and c d; proceed now to divide the length of the line a b into the number of panels between the abutments and the centre of the girder. In the case of the girder shown in Fig. 20, there are six panels, or five stiffeners between the abutments, and the centre of the girder; lay off then the distances x y, y z, z w, etc., equal to one-sixth of the distance a b; from these points just determined draw the ordinates shown in heavy lines, and through the points where they intersect the parabola, as at m, n, o, etc., extend horizontal lines until they intersect the line d b at the points p, q, r, s, etc.; count the number of divisions between b and p, p and q, q and r, and the number of rivets required in the first, second, and third panels respectively will be determined. The rivets in the remaining panels may be determined from r s, s t, t d. When the number of rivets has been determined for each panel, the pitch can readily be ascertained by dividing the distance in inches between the stiffeners by the number of rivets needed. The results obtained from the diagram agree closely with the results obtained in the former calculations demonstrated, which had for their ultimate purpose the determination of the necessary rivets in the flange angles of the plate girder shown in Fig. 20.

In designing any plate girder it is well to observe the fact that only commercial rolled sections should be used. Special rolled sections are difficult to obtain, and when required usually cause delay in the delivery of the work. The plate girder should always be of such size that it can readily be shipped to its destination.

When it occupies more than one freight car—that is, when its length extends through two cars—it should be erected upon the car upon suitably pivoted bearings, so that it may turn with the direction of the cars in taking the curves. The girder should be properly braced for shipment, and should be thoroughly painted before leaving the shop with one coat of red lead and linseed oil, care being taken to observe that all scale from rolling and all rust have been carefully cleaned off with a steel brush.

Upon the data given the girder shown on Fig. 24, has been designed and the drawing shows fully the standard details of construction employed in the best work by conservative engineers.

TABLE FOR DETERMINING THE FLANGE AREAS OF PLATE GIRDER.

Span in Feet	Depth of Girder from C to C of Gravity of Flanges.										
	22	24	26	28	30	32	34	36	38	40	42
10	.091	.083	.077	.071	.067	.063	.059	.055	.053	.050	.047
11	.100	.092	.085	.079	.073	.069	.065	.061	.058	.055	.053
12	.109	.100	.095	.086	.080	.075	.071	.067	0.63	.060	.057
13	.118	.109	.100	.093	.087	.081	.077	.072	.068	.065	.062
14	.127	.117	.108	.100	.093	.087	.083	.078	.073	.070	.067
15	.137	.125	.115	.107	.100	.094	.088	.083	.079	.075	.071
16	.145	.133	.123	.114	.107	.100	.094	.089	.084	.080	.076
17	.155	.142	.131	.121	.113	.106	.100	.095	.089	.085	.081
18	.163	.150	.139	.129	.120	.113	.106	.100	.095	.090	.086
19	.173	.159	.146	.136	.127	.119	.112	.105	.100	.095	.091
20	.182	.167	.154	.143	.133	.125	.117	.111	.105	.100	.095
21	.191	.175	.161	.150	.140	.131	.123	.117	.110	.105	.100
22	.200	.185	.169	.157	.147	.137	.129	.122	.115	.110	.105
23	.209	.192	.177	.164	.153	.144	.135	.128	.121	.115	.109
24	.218	.200	.185	.171	.170	.150	.141	.133	.126	.120	.114
25	.227	.209	.192	.179	.167	.156	.147	.139	.131	.125	.119
26	.237	.217	.200	.186	.173	.163	.153	.145	.137	.130	.124
27	.245	.225	.208	.193	.180	.169	.159	.150	.142	.135	.129
28	.255	.233	.215	.200	.187	.175	.165	.155	.147	.140	.133
29	.263	.242	.223	.207	.193	.181	.171	.161	.153	.145	.138
30	.273	.250	.231	.214	.200	.187	.177	.167	.157	.150	.143
31	.282	.259	.239	.221	.207	.194	.183	.172	.163	.155	.147
32	.291	.267	.246	.229	.213	.200	.188	.178	.168	.160	.152
33	.300	.275	.254	.236	.220	.206	.194	.183	.173	.165	.157
34	.309	.283	.261	.243	.227	.213	.200	.189	.179	.170	.162
35	.318	.292	.269	.250	.233	.219	.206	.195	.184	.175	.167
36	.327	.300	.277	.257	.240	.225	.212	.200	.189	.180	.171
37	.337	.309	.285	.264	.247	.231	.217	.205	.195	.185	.176
38	.345	.317	.292	.271	.253	.237	.223	.211	.199	.190	.181
39	.355	.325	.300	.279	.260	.244	.229	.217	.205	.195	.185
40	.364	.333	.307	.286	.267	.250	.235	.222	.210	.200	.191

APPENDIX
STRUCTURAL STEEL

STRUCTURAL STEEL.

INDEX.

STRUCTURAL STEEL

It is proposed to give a short description of the principal rolled sections in steel commonly used in ordinary riveted construction, and some practical remarks on their use.

Angles. The angle-steel, or, to use the older nomenclature, the angle-iron, is perhaps the most commonly in use among all those sections of material which go to make up riveted work. It may be equal-legged, as in Fig. 1, or unequal-legged, as in Fig. 2.

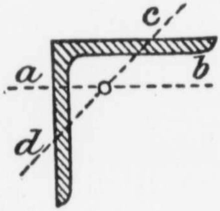


Fig. 1.

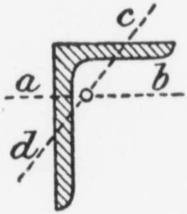


Fig. 2.

The equal-legged angle in its ordinary form has a rectangular outline with a square corner on the outside, the interior faces being sometimes slightly tapered with a connecting round in the inner corner, and the edges rounded off to a quadrant of small radius.

These tapers and the radii of the roundings are not quite the same in all section books, varying with the shape of the rolls of the respective makers, although proposals for the adoption of a uniform standard in this as in other sections have not been wanting.

The unequal-legged angle presents the same general characteristics, while its name speaks for itself. Both the equal- and unequal-legged section is of uniform thickness, without taper.

Variations from these forms are found in the acute-angled angle and the obtuse-angled angle, used where oblique connections of riveted work have to be made. The use of acute-angled angles is attended sometimes with the difficulty of getting the rivets into the acute angle, which must be borne in mind in these cases, it being sometimes necessary, if the angle of connection is very acute, to use a bent plate of sufficient dimensions in lieu of an angle.

Fig. 3.

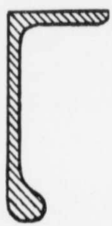


Fig. 4.

Both equal- and unequal-legged angles are also rolled with a round back, as in Fig. 3. They are most commonly used to effect the splice in the main angles of plate or lattice girders, an example of which is given in Fig. 30, the round back of the connecting angle fitting into the interior rounding of the main angles to be spliced. This method of splicing the main angles of a riveted plate girder is the one most commonly adopted, and leads to the consideration of the net sectional area of the angles to be spliced, the corresponding thickness of the angle cover, which must necessarily be greater than that of the main angles, and as a consequence the spaces left for the rivets, their heads, and the amount of metal left outside them.

When, however, angles are used for ties or struts, either singly or in pairs, as in roof trusses, it becomes a simple matter to splice each leg of the angle with a flat of suitable width and thickness.

Another variation in the equal- or unequal-legged angle is that in which the legs are rolled of equal thickness without taper, and the edges and corners, both internal and external, are square and sharp. This, however, is usually considered a special section, and not frequently adopted in ordinary riveted work.

A section of angle-iron used frequently in the frames of ship or caisson work, as beams subject to transverse stress, is the so-called bulb-angle, shown in Fig. 4. This angle is usually unequal-legged, the object of the bulb being to increase the moment of inertia of the beam in the plane of its greatest depth as a beam, while it also serves the purpose of thickening and rounding the edge of the angle where exposed to passing traffic, etc.

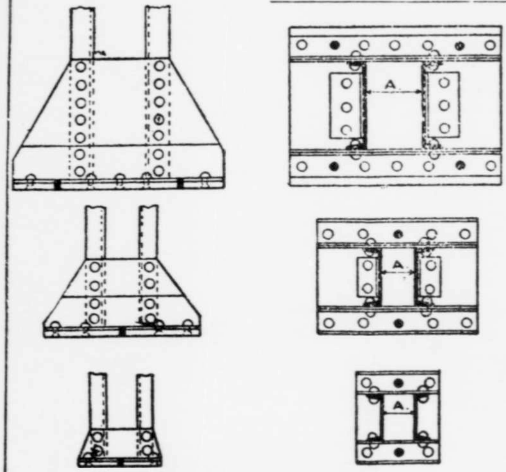
The bulb may be rolled on one side of the longest limb as shown, or on the opposite side. The sectional area of the bulb varies slightly with different makers, but is standardized in the standard section.

According to the usual practice, the vertical limb or web with the bulb is made parallel, the taper being given to the other limb of the angle. In the standard section of bulb-angle, both limbs are of uniform thickness, there being no taper. If the section be increased beyond a certain minimum thickness the dimensions of the sides are increased proportionately, while the bulb retains the same projection from the face of the web.

The uses of angle-steels are multifarious. In addition to their primary function of connecting members of a structure in planes at right angles to one another, such as the web and flanges of a plate girder, or in oblique connections, they are also found used to a great extent either as beams, struts, or ties. As beams we find them in purlins to roofs, as secondary beams in a variety of structures, in the framing to sides of corrugated iron sheds, and the like. As struts they are employed in the members of lattice girders, in the compression members of roof principals, and as secondary bracing where some lateral stiffness is required in a number of cases. As ties they appear more or less successfully in the tension members of light trusses, but their effective use in tension is somewhat qualified by the necessity of securing them in many cases by one leg only. This remark also applies under similar conditions to their use as struts. In this latter capacity they will be further referred to.

The selection of the dimensions and scantlings of angles will be determined by a variety of considerations depending on the use to which they are put. As connecting members simply we shall find their dimensions ruled to a large extent by the size of rivets employed, the bearing stress allowed, and so on; as beams, by their moment of inertia; as struts, by their least

STANDARD BASES FOR CHANNEL COLUMNS



Dist. A	Column	Safe Load	Size of Base	Size of Gussets
9"	15"	227	28"x1½"x2'-8"	21"x½"x2'-8"
		137	28 x1 x1'-8	12 x⅜ x1'-8
8"	13	214	24 x1½ x3'-0	21 x½ x3'-0
		130	24 x1 x1'-10	12 x⅜ x1'-10
7½"	12	182	21 x1½ x2'-11	21 x½ x2'-11
		83	21 x1 x1'-4	
6"	10	136	20 x1 x2'-3	12 x⅜ x2'-3
		69	20 x¾ x1'-2	
5½"	9	104	18 x1 x1'-11	12 x⅜ x1'-11
5¾		57	18 x¾ x1'-1	
5½	8	91	16 x1 x1'-11	12 x⅜ x1'-11
5¾		45	16 x¾ x1'-	
5½	7	83	15 x1 x1'-10	12 x⅜ x1'-10
"		39	15 x¾ x0'-11	
	6	66	14 x1 x1'-7	12 x⅜ x1'-7
5¾"	"	32	12 x¾ x0'-11"	

Weight of Plates	Size of Angles		Weight of Angles	Weight of Rivet Hds.	Total Weight
571	6"x4"x½"x2'-8"	6"x6"x½"x1'-0"	126	11	708
210	6 x4 x⅜ x1'-8	6 x4 x⅜ x1'-0	66	7	283
581	6 x4 x½ x3'-0	6 x6 x½ x0'-10	130	11	722
206	6 x4 x⅜ x1'-10	6 x4 x⅜ x0'-10	66	7	279
527	6 x4 x½ x2'-11	6 x6 x½ x0'-9½	126	11	658
95	6 x4 x⅜ x1'-4	6 x3½x⅜ x0'-9½	51	3	149
222	6 x4 x⅜ x2'-3	6 x6 x7/16 x0'-7½	77	7	306
60	6 x4 x⅜ x1'-2	6 x3½x⅜ x0'-7½	43	3	106
176	6 x3½x⅜ x1'-11	6 x3½x⅜ x0'-7	59	7	242
50	6 x3½x⅜ x1'-1	....	25	3	78
163	6 x3½x⅜ x1'-11	....	45	6	214
41	6 x3½x⅜ x1'-11	....	23	3	67
150	6 x3½x⅜ x1'-10	....	43	6	199
35	6 x3½x⅜ x0'-11	....	21	3	59
124	6 x3½x⅜ x1'-7	....	37	6	167
28	6 x3½x⅜ x0'-11	....	21	3	52

15" column for 227 tons
13" column for 214 tons
12" column for 182 tons

15" column for 137 tons
13 column for 130 tons
10 column for 136 tons
9 column for 104 tons
8 column for 91 tons
7 column for 83 tons
6 column for 66 tons

12" column for 83 tons
10 column for 69 tons
9 column for 57½ tons
8 column for 45 tons
7 column for 39 tons
6 column for 32 tons

Bearings on Foundations 500 lbs. per sq. in.

Submitted by L. A. Starrett

Safe Loads in Tons of 2000 lbs. for St'd Angles used as Columns with Square Ends

By F. G. Walker

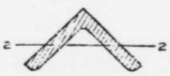
P=Ultimate strength in pounds per square inch.
L=Length of column in feet.
Y=Least radius of gyration in inches, at Axis, 2—2.

$$P = \frac{50000}{1 + \frac{(12 \times L)^2}{36000 \times r^2}}$$

$\frac{P}{4x200}$ =Safe load in tons per square inch.

Safe load on column in tons $= \frac{P}{4x2000} \times A$

A=Area of section in square inches.
5000=Ultimate compressive strength of material in pounds per square inch.



Size of Angle	Weight per Foot	Area of Section	Least Rad. of Gyrati'n	LENGTH IN FEET													
Inches	Lbs.	Sq. Inches	Inches	2	3	4	5	6	8	10	12	14	16	18	20	22	24
6 × 6 × 7/8	33.1	9.74	1.16	60.27	59.67	58.52	56.89	55.12	51.36	47.22	42.99	38.62	34.88	31.13	28.23	25.22	22.70
6 × 6 × 7/16	17.2	5.06	1.19	31.31	31.00	30.40	29.55	28.95	27.17	25.18	23.10	21.06	19.10	17.29	15.62	14.12	12.78
4 × 4 × 3/4	18.5	5.44	.77	33.33	32.07	30.90	29.30	27.34	23.58	20.04	17.02	14.43	12.34	10.48	9.02	7.81	5.44
4 × 4 × 3/8	9.8	2.86	.79	17.52	17.03	16.24	15.96	14.80	13.06	11.34	9.65	8.40	7.18	6.20	5.38	4.69	4.11
3½ × 3½ × 5/8	13.6	3.99	.67	24.21	23.09	21.87	20.43	18.38	15.27	12.56	10.36	8.57	7.11	5.97	5.00	4.34	3.76
3½ × 3½ × 3/8	8.5	2.49	.68	15.10	14.54	13.77	12.84	12.15	10.38	8.74	7.33	6.19	5.21	4.43	3.79	3.27	2.85
3 × 3 × 5/8	11.5	3.36	.58	20.19	19.09	17.77	16.27	14.40	11.60	9.26	7.88	5.99	4.92	4.09	3.44	2.69	2.51
3 × 3 × 1/4	4.9	1.44	.59	8.65	8.18	7.62	7.03	6.54	5.44	4.36	3.55	2.90	2.40	2.00	1.70	1.45	1.27
2½ × 2½ × 1/2	7.7	2.25	.48	13.27	12.22	11.07	9.83	8.65	6.64	5.15	4.02	3.19	2.58	2.02	1.76	1.56	
2½ × 2½ × 1/4	4.1	1.19	.49	7.00	6.46	5.30	5.27	4.77	3.73	2.43	2.31	1.85	1.51	1.25	0.96	0.88	
2 × 2 × 7/16	5.3	1.56	.38	8.79	7.80	6.73	5.69	4.88	3.53	2.59	1.95	1.57	1.21	0.95			
2 × 2 × 3/16	3.5	.72	.39	4.10	3.63	3.17	2.70	2.30	1.66	1.52	0.94	0.72	0.57	0.47			
1¾ × 1¾ × 7/16	4.6	1.34	.34	7.41	6.44	5.47	4.57	3.38	2.32	1.65	1.28	0.94					
1¾ × 1¾ × 3/16	2.2	.63	.35	3.48	3.03	2.57	2.14	1.78	1.27	0.89	0.68	0.51					
1½ × 1½ × 3/8	3.4	.99	.29	5.20	4.33	3.03	2.83	2.17	1.64	1.01	0.74						
1½ × 1½ × 1/8	1.3	.36	.30	1.92	1.61	1.31	1.07	0.93	0.64	0.46	0.34						
1¼ × 1¼ × 1/4	2.0	.57	.24	2.78	2.19	1.68	1.30	1.00	5.64	0.44	0.32						
1¼ × 1¼ × 1/8	1.1	.30	.24	1.46	1.15	.88	.68	0.57	0.36	0.20	0.17						
1 × 1 × 3/16	1.2	.34	.19	1.48	1.07	.77	.56	0.43	0.26	0.18							
1 × 1 × 1/8	.8	.24	.19	1.04	.75	.54	.40	0.33	0.20	0.15							

by their

radius of gyration. But in most cases considerations of rivet spacing in connections, etc., will be ruling factors in the design; and the young draughtsman will in this, as in so many other cases, be wise in drawing all doubtful details full size, or to a large scale, before he finally determines his section.

Tees. The tee-steel, or tee-iron, ranks perhaps next to the angle in general utility. Its general form is shown in Fig. 5. The proportions of top table to stem or web are very variable, and the error of misdescription of the dimensions is one very frequently

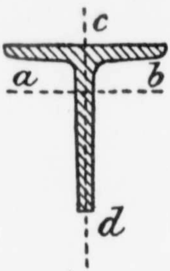


Fig. 5.

found on drawings, rectified, it may be, by dimensioning the members, but the young draughtsman will do well to remember that a 6″x3″ tee is by no means the same thing as a 3″x6″ tee. He will probably ascertain this to his cost if he specifies the one in mistake for the other, in the absence of a figured section. The width of top table is the dimension first quoted.

The top table and stem are usually both slightly tapered, and connected by roundings of small radius, the corners of the extremities of the limbs being sometimes rounded and sometimes square. A variation sometimes found is when the top table is of uniform thickness and the stem tapered, or vice versa.

The standard section of tee has the web tapered, the edges of the top table being rounded off beneath. while the edge of the web is square.

Tees are commonly used as beams, as in the case of purlins, secondary bearers in fire-proof floors, and the like. As struts they are a favorite section for the compression members of roof trusses of moderate span, lattice girders, etc., also as stiffeners to the webs of plate girders. As ties their use is more limited, there being some difficulty in making such an end connection as will effectively bring into play the whole cross-section of the metal. Tee struts will be further referred to in the chapter on columns.

The proportions of tees to be adopted in any particular detail will, apart from the value of their moments of inertia when used as beams, or of their least radius of gyration when used as struts, be frequently ruled by the dimensions and spacing of their riveted or bolted connections. Thus, to take a familiar example, the tee stiffener to the web of a plate girder will require a width of top table or flange sufficient to take the rivets required in a joint of the web plating, which again will be ruled by the shearing stresses in the web, and the number and diameter of the rivets required. Or supposing, in the case, let us say, of a footbridge with timber floor secured to tee bearers by bolts or coach-screws, the top table of the tee must be of width enough to receive screws or bolts of the diameter required, with a sufficient amount of metal outside the hole, and sufficient space between the stem and hole for the nut of the bolt or head of the lag-screw. Such elementary considerations may bear the aspect of truisms, but careful attention to points of detail such as these will always be found to characterize sound ironwork design.

Bulb Tees. A tee section with a bulb rolled on the lower extremity of the stem constitutes the useful section known as bulb tee or deck beam. This section is used to a considerable extent in shipbuilding, and oc-

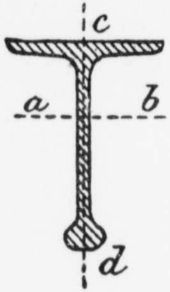


Fig. 6.

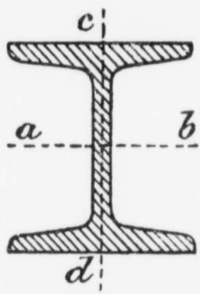


Fig. 7.

casionally in purlins and similar beams. The moment of inertia is increased by the bulb, which also forms a finish to the lower edge of the stem, which is usually rolled with parallel sides, the top table or flange having a taper similar to the flange of a rolled beam. The area of the bulb and relative thickness of stem and flange vary somewhat in different rolling mills, but are standardized in the standard section.

I Beams. This well-known, most useful, and deservedly popular section is shown in Fig. 7.

The web is most commonly rolled with parallel sides, the flanges being tapered, and connected to the web with roundings in the internal corners. The proportion of web thickness to flange thickness, the amount of taper on the latter, the radii of the roundings, have been, as in other sections, variable with different makers. These proportions exert some influence on the precise values of the moments of inertia and resistance, and the manufacturers of this section frequently give in their trade catalogues the mechanical elements and exact proportions of the sections rolled by them. This course is commendable in preference to the compilation of tables of strengths in which the data of the calculations are absent.

In the standard section the thickness of web and flanges, the taper of the latter, and the radii of the connecting curves, are all standardized.

The depths of this section as usually found in the market range from 3 inches to 24 inches, and the width of flange from $2\frac{1}{4}$ inch to 8 inches.

This width of flange has recently been exceeded in continental rolling mills, and the section thus produced offers considerable advantages in column design owing to the increase of the least radius of gyration, and in the arrangement of details in connections, where the additional space afforded is often very convenient. Notwithstanding the width of flange the section can be very cleanly rolled, right out to the edge of the flange, and straight and true in its length.

The proportions of depth and width require careful consideration when selection is being made of a section suitable for the purpose in view. The economy of this section as regards riveting, and the facility with which, aided by the table of strengths obligingly furnished by the manufacturer, the selection of a section for strength can be made, undoubtedly contribute to the favour in which the rolled beam is held. It is questionable, however, as a matter of taste, how far the indiscriminate use of the section, especially in the largest sizes, contributes to the artistic appearance, if it may be so called, of well-designed ironwork, and it must be confessed that economy of both cost in manufacture and painstaking in design are frequently attained at the expense of appearances. It is to be feared, however, that any regard for appearances in structural steelwork, if it implies any increase in cost, real or imaginary, will in these competitive days be regarded by many as an economic heresy.

No universally recognized standard of proportion of the flanges and web of rolled beams had, up to a recent period, been arrived at by manufacturers. Published lists of sections show considerable variation in the proportion of web thickness to flange width, of web thickness to height of joist, of mean thickness of flange as compared with width, or with height of joist. The thickness of web is found to range between seven and twelve hundredths of the flange width in beams of 3½ inches width of flange and upwards, and may be taken to average about eight hundredths. In beams under 3½ inches in flange width the web will average about one-tenth of flange width. The mean thickness of flange is equally variable, and will be found to range between five and nine hundredths of the height of beams in beams above 6 inches high. In shallower beams the mean flange thickness will range from eight to twelve hundredths.

The maximum moment of inertia of the cross-section will increase in value per unit of area as the web becomes thinner, but the student need not be reminded that the moment of inertia is not the only standard of the ultimate actual strength of the beam. Apart from the practical requirements of the rolling mill the web must be thick enough not only to withstand the usual web stresses, but also to resist the effects of corrosion, and to assist the top flange to resist the buckling tendency under compression which is found in practice to limit the strength of the beam when not supported laterally, the compression flange under these conditions usually failing by lateral flexure before the full tensile resistance of the metal in the lower flange has been attained.

Within the usual limits of variation of web thickness as rolled by different manufacturers, the maximum value of the moment of inertia compared with the total sectional area or weight per foot run will be attained when the flange thickness is from nine to ten hundredths of the height of the girder, but the economic efficiency is practically equally as great between the limits of six and twelve hundredths, and the lower value of flange thickness is the one more usually found in beams over 6 inches in height.

It is customary to specify the width of flange and total depth coupled with the weight per lineal foot of the rolled beam required, and this is doubtless the most desirable course to pursue. It leaves, however, the exact relative thicknesses of web and flanges an open question, though the total sectional area is of course governed by the weight per foot. If, on the other hand the designer specifies the thickness of web or flange, he must in all probability be prepared to accept the section rolled by some one particular maker, and in such a case he will do well to follow the dimensions given in the trade section books. These remarks do not of course apply to the use of the standard section, where the thicknesses of web and flanges for the given depth, width, and weight are standardized. As regards the values of the moment of inertia based upon the proportions of web and flange stated, it may be remarked that for any weight per lineal foot of beam of any one particular section, the moment of inertia for that weight may for approximate calculations be taken as simply proportional to the weight per foot.

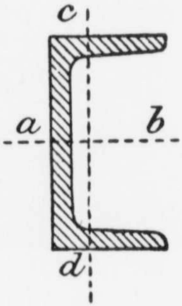


Fig. 8.

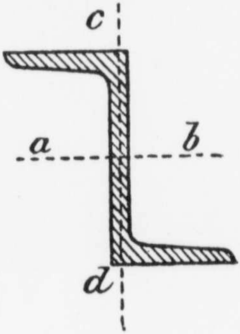


Fig. 9.

The value of the least radius of gyration will not be found to vary materially for any practical variation of the section within the limits usually rolled.

Channels. This section is represented in Fig. 8. The web is rolled with parallel sides, the flanges are tapered and connected to the web with rounded internal angles, and this is the type of the standard section. Increase of weight beyond the minimum section is obtained mainly by an increase in web thickness.

This section is frequently used as a beam in small bearers, as a strut in the compression members of lattice girders and roof trusses, and in riveted columns, while it is occasionally useful in certain connections as taking the place of two angles.

If a small section of channel is required having a rivet through the web, as, for example, in the case of two channels crossing one another, back to back, and riveted together, care must be taken in the selection to secure one wide enough to permit of the formation of the point of the rivet. For this reason in such cases a small angle will frequently be found preferable to a small channel.

Z Bars. This useful section is shown in Fig. 9. It is largely used in the frames of ship and caisson work, having a considerable moment of inertia for its weight, as compared with angles or tees, with ample width of flange for riveted connections.

The web is rolled with parallel sides, the flanges having a taper and being connected to the web by curves at the internal angles. In the standard section the flanges have no taper, but are of uniform thickness.

Increase of weight beyond the minimum section is obtained by thickening the web, the width of flange being slightly increased.

The section is frequently rolled with a uniform thickness of web and flange, the latter being tapered as above described, and the quoted thickness being the mean between that of the root and of the point of the flange. The flanges of the standard section have a thickness in excess of that of the web.

Occasionally the flanges are rolled of unequal width; this is a convenience where additional width is required for heavy riveting, and in those cases where the lesser width of flange is sufficient for the riveted attachments, then the increased width of the other flange yields a larger moment of inertia.

Further reference will be made to the use of Z Bars in the practical design of columns or struts.

In the preceding pages the sections which have been described and of which the principal mechanical elements have been given, viz. angles, equal and unequal-legged, tees, bulb tees or deck beams, rolled beams, channels, and Z Bars are those which may be called the elementary or standard sections, which in combination with plates and bars are most ordinarily employed in riveted constructional steelwork. It is not possible to consider in detail the very numerous forms of rolled sections, other than those above mentioned, employed for special purposes. These include, for example, the varied sections of railway bars (bullheaded, bridge, and flat-footed), fish plates, guard rails, sections of trough flooring (usually formed in hydraulic presses), quadrant sections for pile-work, half-round, segmental, or cope steels, sash-bars, and fancy and other sections.

With respect to the use of plates and bars, it is sufficient to point out that the dimensions to which these can now be rolled are amply sufficient to meet all legitimate demands of the designer of constructional steelwork. Various makers have their own standard maximum dimensions to which plates, sheets, or flats can be rolled, and it is customary to assign a limit of superficial area for each thickness of plate, which is not exceeded without entering into special arrangements. Thus for a $\frac{3}{8}$-inch plate, the limit of area is given by one authority as 135 square feet, the maximum length of plate being 42 feet, and the maximum width 7 feet 6 inches, it being understood that maximum length and maximum width are not rolled together, but that, given the length, the width is such as not to exceed the limit of area, or vice versa. Again, for a plate $\frac{7}{8}$ inch thick, a limit of 250 square feet is given, the maximum of length and width being 56 feet and 10 feet respectively.

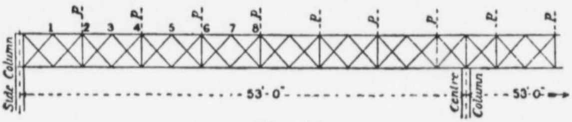


Fig. 10.

With respect to flats, usually so called when the width does not exceed 12 inches to 15 inches, the available length obtainable without joint will usually be found to meet all practical requirements, as other considerations, such as the maximum length permissible for transport or shipment, very frequently rule the case.

Lattice Girderwork for Roofing. As an example of this application of girderwork, details will now be given of a lattice girder of 53-foot span supporting a series of roof principals.

Fig. 10 gives a skeleton outline of the triangulated girder the principals being carried immediately over

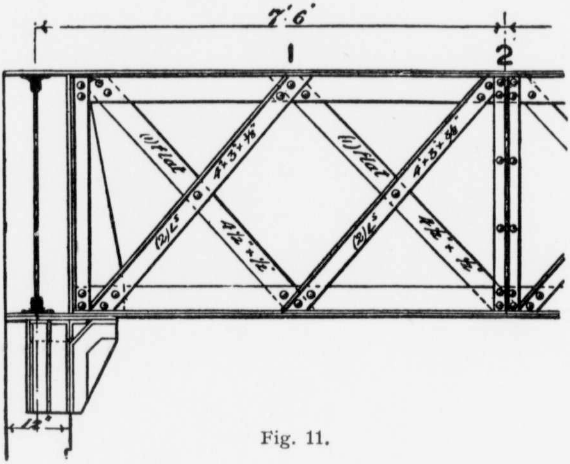


Fig. 11.

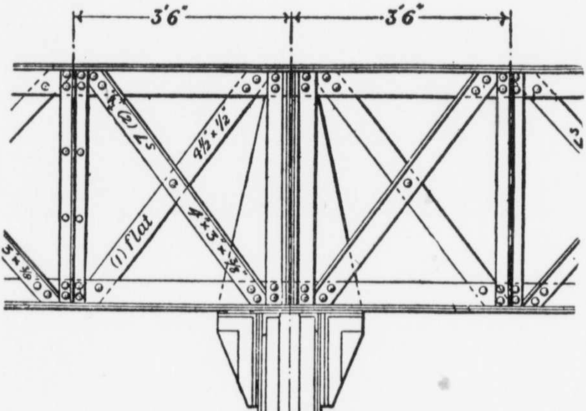


Fig. 12.

the vertical struts, as shown, p, p, p. The girders are carried on side columns and a center column as shown, but are not continuous, and a roof principal does not

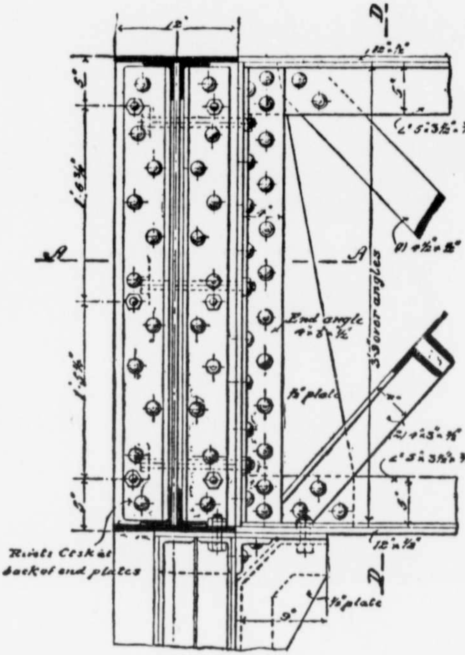


Fig. 13.

occur over the center column. The general arrangements at side and center columns are shown in Figs. 11 and 12, while the details of connections are shown to a larger scale, for the side column in Figs. 13, 14, and 15, the latter being a section of that portion of the column to which the girder is attached, and for the center column in Figs. 16, 17, and 19. The normal

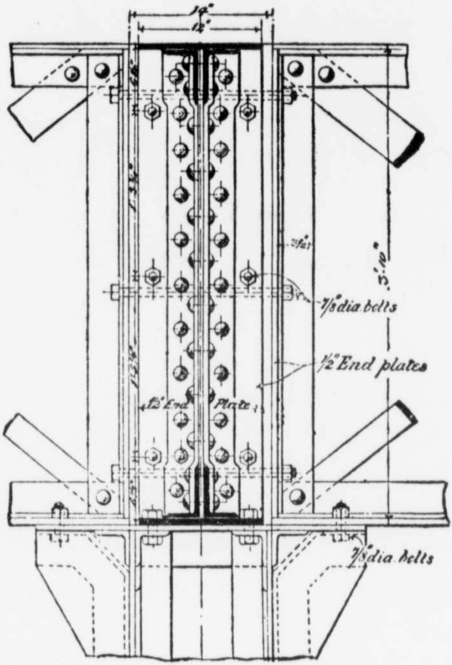


Fig. 14.

section of the girder is shown in Fig. 18. The details of the apices of the triangulations, or the intersections of the web bracings with the flanges, are shown for apices 2, 4, 6, and 8 in Figs. 20, 21, 22, and 23, and for apices 1, 3, 5, and 7 in Figs. 24, 25, 26, and 27, showing the riveted connections. These are examples on a small scale of oblique connections, and may be studied in connection with those for roof-work.

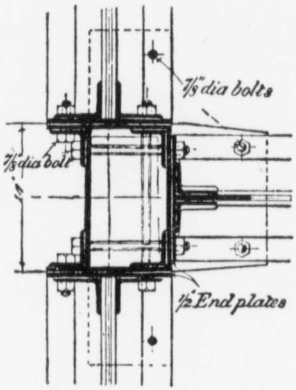


Fig 15.

In Figs. 28, 29 and 30 are given the details of the angle and flange plate joints in the girder, which occur between apices 6 and 7 in the top boom, and break joint in the bottom boom, thus dividing the girder into two lengths for convenience in transport, and keeping the lengths of angles and plates within ordinary limits.

Fig. 31, a simple flat bar, is met with as a strut or compression member in the webs of multiple lattice

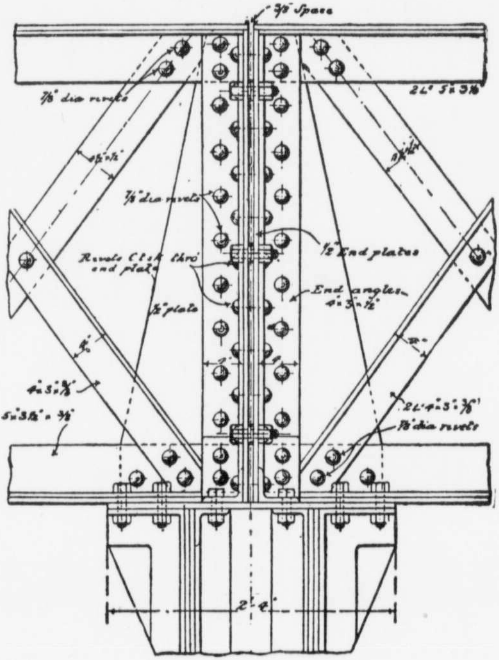


Fig. 16.

girders. It is obviously weak in the direction of its least radius of gyration, and is prevented from failure in that direction by the frequent intersection of the

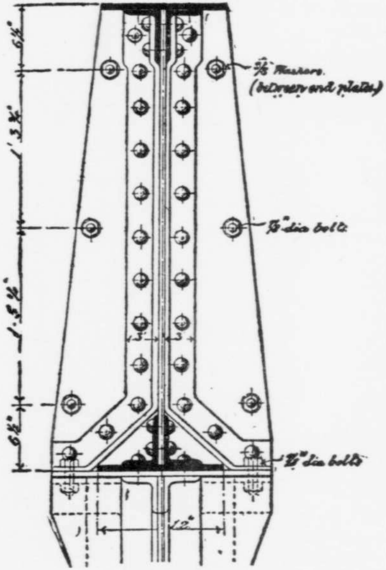

Washers.
(between end plates)
dia bolts
dia bolts
6½"
1' 3½"
1' 5½"
6½"
3
3
12"

Fig. 17.

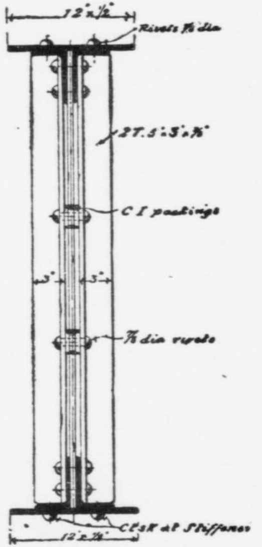

12" x ½"
Rivets dia
C I packings
dia rivets
3"
3"
C'sk at Stiffener

Fig. 18.

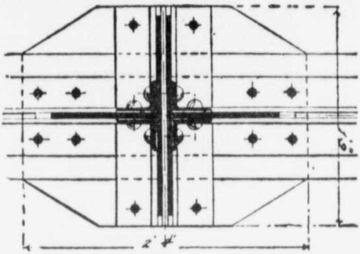


Fig. 19.

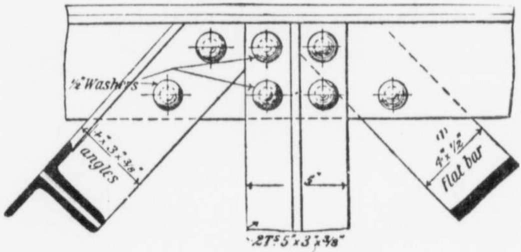


Fig. 20.

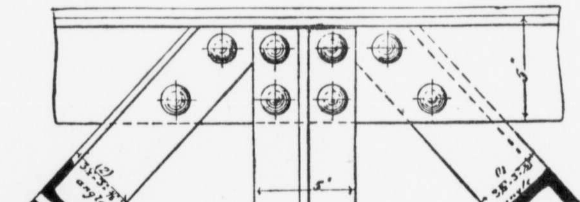


Fig. 21.

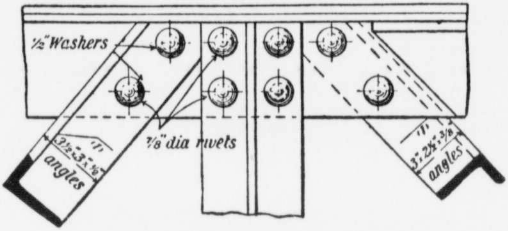


Fig. 22.

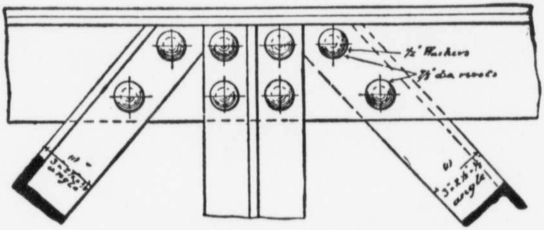


Fig. 23.

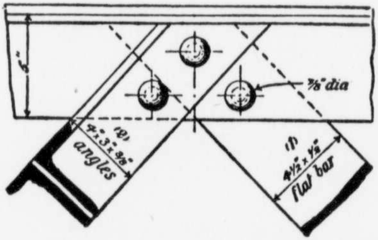


Fig. 24.

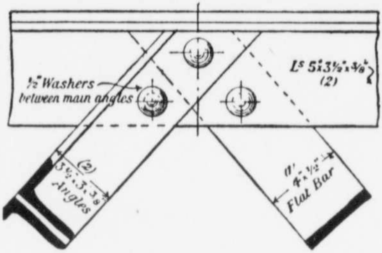


Fig. 25.

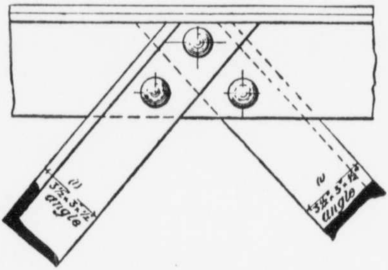


Fig. 26.

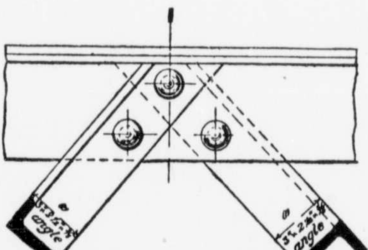
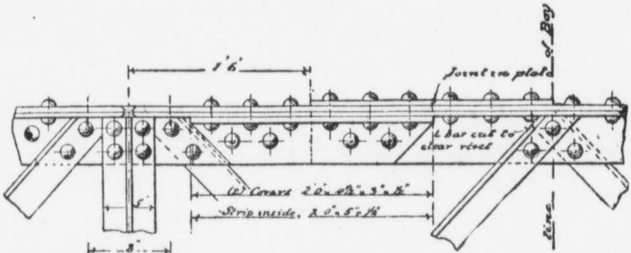


Fig. 27.

tension diagonals and the use of stiffening vertical members. Outside the particular application men-

Fig. 28.



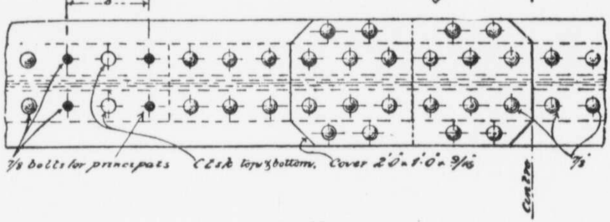


Fig. 29

tioned, it is not frequently used as a compression member, being obviously less adapted for that purpose than other and stiffer sections.

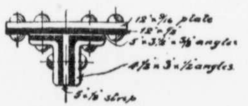


Fig. 30.

Fig. 32 is an example of the use of flat bars in pairs, frequently used as compression members in the diagonals of roof principals of small span, say up to 30 or 40 feet or thereabouts. The bars are usually pin-connected at the ends, and swelled apart at the centre by cast-iron distance pieces as shown. It is a convenient form of strut for light loads, but is apt to fail by weakness at the ends, the bars buckling near the pin connections.

Fig. 33 is a simple angle, either equal or unequal legged section. This section is in common use in the compression diagonals of small lattice girders and as a single angle in the rafters of small and light roof principals and their diagonals.

Fig. 31.

Fig. 32.

Fig. 33.

Fig. 34.

The double angle shown in Fig. 34 is a variation of the same type used for the same purposes where the loads are heavier. The angles are occasionally riveted close together, back to back, and thus form practically a riveted tee. The mode of attachment of the single or double angle as a compression member is nearly always of one leg only. Under these conditions the distribution of stress is conceivably very unequal over the whole cross-section, and it is to be regretted that while numerous experiments have been carried out upon the ultimate strength of angles in compression, so little has been done to elucidate under actual practical working conditions the ultimate strength of angles connected in the usual way, and with the direction of the compressing forces out of center with the center of figure of the section.

A further elaboration of the use of angles is given in Fig. 35, which is a somewhat special section, occasionally used in the compression diagonals of roof trusses of large span. The four angles are brought near together at the ends and swelled out at the center by cast-iron distance pieces.

Fig. 35.

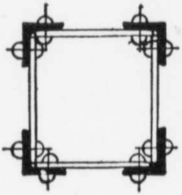


Fig. 36.

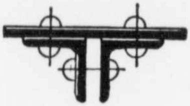


Fig. 37.

A more simple example of the use of four angles is given in Fig. 36, also used as a compression member of large roof trusses, and consisting of four angles connected by internal stiffening plates at intervals as shown, or by light lattice bracing in the same planes. Such a strut is frequently pin ended, but may also be riveted and "fixed" ended. This strut is an example on a small scale of the type shown in Fig. 55, which is intended for large columns with heavy loading.

Fig. 37 will be at once recognized as the compression or top flange of ordinary plate or lattice girder construction of moderate spans. The increase of sectional area required is usually obtained by increasing the thickness or number of the plates.

Fig. 38, showing a simple tee section, is in common use as the rafter or compression member of roof trusses up to about 40-feet span, and in their compression diagonals. Its place is sometimes taken by the double angles shown in Fig. 34. It is also frequently used as the upper flange of small lattice girders or trussed purlins in roof work.

The double tee (Fig. 39) is used in the compression diagonals of large roof trusses, especially in those types of "crescent"-shaped principals, where the

Fig. 38.

Fig. 39.

Fig. 40.

stresses in the diagonals are not great. In such cases the double tees are brought near together at the ends and swelled out in the center with cast-iron distance pieces, and are not so liable to the local weaknesses that may occur with the use of double flats, as in Fig. 32.

Fig. 40 is the simple channel, frequently used alone, but perhaps more commonly in combination, either in pairs, as in Figs. 41, 43, or as in Fig. 52.

Fig. 41 gives a pair of channels connected by lattice bars on both sides, and forming an open section used in the compression rafters of roofs of considerable span, say from 70 to 100 feet, as ordinary columns, or in compression members in large triangular girders.

Fig. 42 is the same combination of channels, but with a solid plate connection on the one side and open latticing on the other, used for similar purposes to those above mentioned, but where heavier stresses have to be provided for.

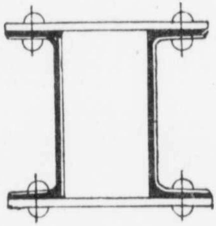


Fig. 41.

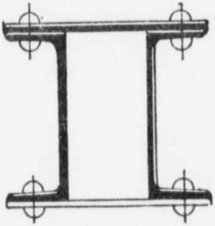


Fig. 42.

Fig. 43 shows a section of column frequently used for heavy loads in buildings, warehouses, dock sheds, and the like, consisting of two channels and two solid plate sides forming a closed cell.

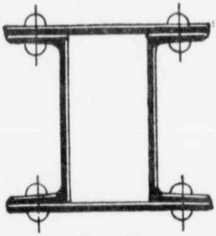


Fig. 43.

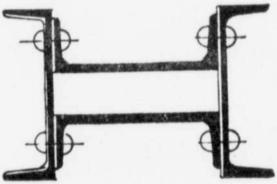


Fig. 44.

Fig. 44 is a form of section composed of four channels as shown.

Fig. 45 shows an effective section consisting of four channels connected at the external corners by four angles.

We now approach a group of sections in which the rolled beam is the principal feature, used to a very large extent in ordinary building construction, and which combine a considerable amount of stiffness with simplicity and ease of construction, and economy in riveting.

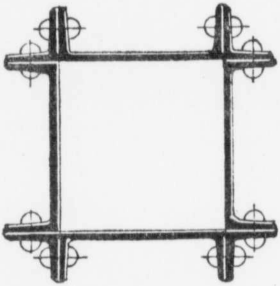


Fig. 45.

Fig. 46.

Fig. 46 is the simple rolled beam, in which no riveting is required except in end connections when used as a plain column or strut. The principal defect in this section is the inequality of the radii of gyration round the axes severally square to the web and flanges. In columns or struts exposed to lateral shock the liability to flexure in a plane square to the web must be borne in mind. This defect has apparently been recognized by some manufacturers, who have produced a section of rolled beam of exceptional width in the flange. This weakness is also to some extent corrected in the next development of this form of column shown in Fig. 47, where plates (one or more in thickness) are riveted to the flanges of the rolled beam, whereby the moment of inertia round the axis parallel to the web is increased. This section is very useful, and is largely used in columns for general building purposes.

Fig. 47.

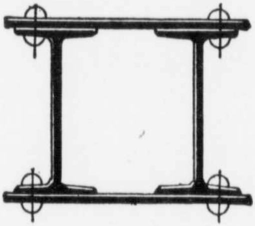


Fig. 48.

Fig. 48 shows a pair of rolled beams, connected by plates as shown, or by a system of lattice bars in the same planes. A practical example of this type on a large scale will be referred to in detail hereafter.

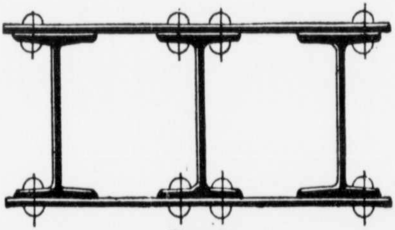


Fig. 49.

Fig. 49 shows a strong column for heavy loads, composed of three rolled beams connected by plates as shown, or by latticing.

Fig. 50 shows a combination of three rolled beams which is the prototype in miniature of the more elaborate section shown in Fig. 58.

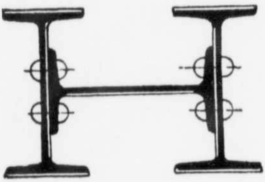


Fig. 50.

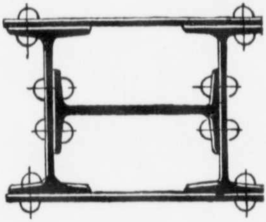


Fig. 51.

Fig. 51 shows the same combination, with the addition of external flange plates, which may be replaced either by latticing or flat stiffening plates at intervals.

Fig 52.

Fig. 52 gives a column composed of one rolled beam and two channels, a simpler form of the type shown in Fig. 56.

It not unfrequently happens that a built-up section of plates and angles, though more expensive, offers greater facilities to the designer in certain details of connections or in arrangement of cross-section than a simple rolled section of similar type of outline, and so we frequently find the built-up section consisting of a plate and four angles, shown in Fig. 53. This section, similarly to that shown in Fig. 47, may be further elaborated by the addition of flange plates riveted to the angles.

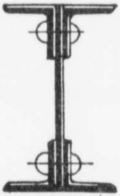


Fig. 53.

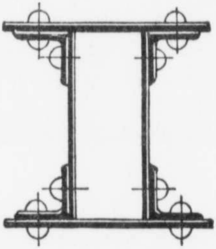


Fig. 54.

Fig. 54 gives a box section of great strength and stiffness, frequently used in columns carrying heavy loads. The same amount of metal disposed as shown in Fig. 55 will yield a greater uniformity in the value of the radii of gyration about different axes, but the riveting is more difficult to get at, and must be dealt with in a manner similar to that adopted in ships' masts, sheer legs, derricks, etc.

Fig. 56 gives a valuable section, of good appearance, and great stiffness in all planes, composed by the addition of channels riveted to the section shown in Fig. 53.

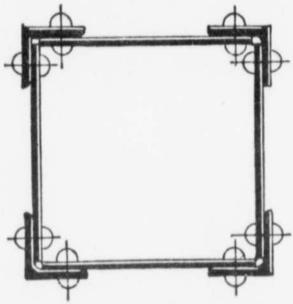


Fig. 55.

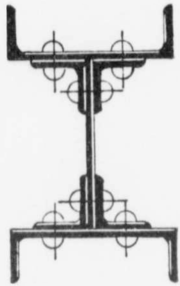


Fig. 56.

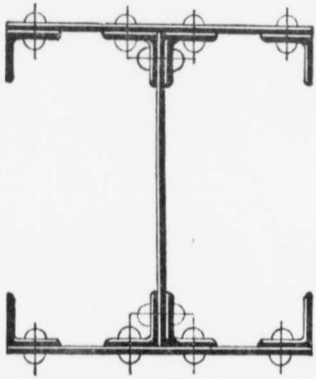


Fig. 57.

Figs. 57 and 58 show types of built-up sections of plates and angles adapted to meet special conditions in large columns carrying heavy and diverse loadings.

The use of these sections will be further alluded to in detail.

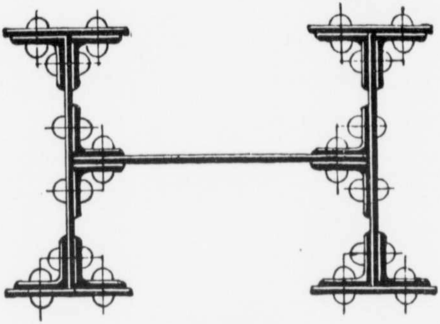


Fig. 58.

Figs. 59 to 61 give sections of columns composed largely of Z Bar sections combined with plates or latticing.

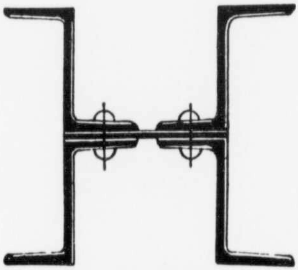


Fig. 59.

Fig. 59 is composed of four Z Bars and one central plate. Fig. 60 is of similar section, with additional plates on the outside. Latticing may take the place of these outside plates.

Fig. 61 shows a similar combination with the Z's turned the reverse way, the metal being disposed to

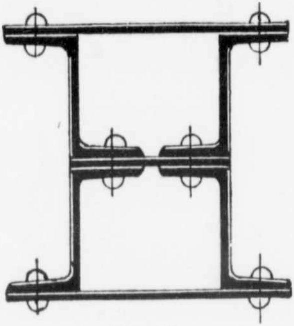


Fig. 60.

better advantage, tnough the appearance of the column is perhaps not so satisfactory.

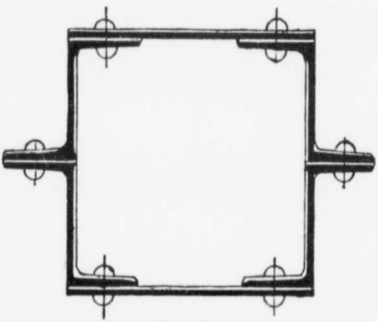


Fig. 61.

Figs. 62 and 63 are sections of a more or less special nature, somewhat less simple in their end connections and the details connecting them with other members than the types above considered.

Fig. 62 is composed of four tees, or four sets of double angles, disposed as shown and connected by bent plates, or trough sections.

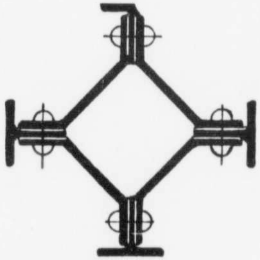


Fig. 62.

Fig. 63 is a section of a type of column made up of the so-called trough sections used for flooring and

Fig. 63.

Fig. 64.

decking. This combination produces a column of great strength and stiffness, though not so well adapted for secondary connections as others.

The use of the circular column in any other material than cast iron in ordinary building construction is somewhat limited. Its use in cast iron in the type of section shown in Fig. 64 is too universal and well known to need any further description.

The circular section in mild steel may in small columns take the form of welded tubes, and in larger sections of plates bent to a circular curve and butt-jointed with covers.

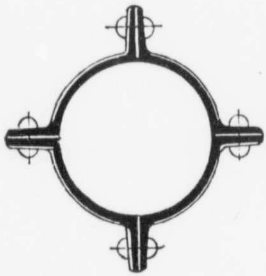


Fig. 65.

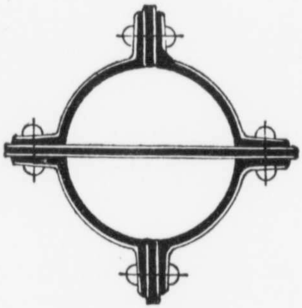


Fig. 66.

In this form we find its use on a large scale in sheer legs, ships' masts, derrick poles, and occasionally in bridge work of very large span. In such structures the circular plate is frequently stiffened internally, and in sections of sufficient size manual labor inside the tube is used for the purposes of riveting.

In Figs. 65 and 66 we have the section generally known as the Phœnix. The figure gives an arrangement in four sections only, but a larger number may be employed in accordance with the size required. This form is one of great stiffness, and in wrought iron has given very high results in the testing machine, possibly owing to the mutual support given by the form of section and the stiffening ribs, and its consequent freedom from local weakness in the unsupported part of the plate.

Secondary attachments offer some little difficulty with this form of section, and the type most frequently found is that shown in Fig. 66, where filling strips are inserted between the flanges of the segments. This gives the opportunity of insertion of sketch plates wherever required for attachments, the filling strips being stopped off as required. Occasionally the strips are carried right through the diameter of the column.

If we now endeavor to institute a comparison between the sections which have been above described, based either on the grounds of efficiency or economy in first cost, it must be premised that any section can hardly be considered per se without reference to its surroundings, and the use to which it has to be put.

The requirements in detail of the various secondary members which may have to be attached to the simple column will always have an influence in the choice of selection. The design in detail of the cap and base, the attachments for such fittings as counter-shafting brackets, the counterbracing as in the case of the piers to a viaduct, the attachments of traveler, roof, or floor girders will invariably demand careful consideration, and the success of the design as a whole will be influenced by the skill with which these details are worked out.

As regards economy in first cost, other things being equal, it may be assumed that the section having the least amount of riveting will be the cheapest per unit of weight.

The extent, however, to which the use of simple sections, with a comparatively small amount of riveting, can be carried, is governed by all the conditions of the case, and the amount of load to be carried.

A further comparison may be made of the sections above described which is not without importance, and that is the extent to which the surfaces of the respective sections can be protected from the effects of corrosion, or, in other words, the extent to which the sections can be got at by the paint brush.

All the simple sections are fairly accessible, having latticing on one or both sides and can be painted internally, while those with solid plate flanges have closed cells, which cannot easily be painted under ordinary conditions. Such closed cells are not infrequently filled with concrete, although the extent to which this acts as a preservative coating depends largely on the degree of close contact with the metal obtainable.

STRUCTURAL STEEL.

MANUFACTURERS' STANDARD SPECIFICATIONS.

Revised, February, 1903.

Process of Manufacture.

(1) Steel may be made by either the open-hearth or Bessemer process.

Testing and Inspection.

(2) All tests and inspections shall be made at the place of manufacture prior to shipment.

Test-Pieces

(3) The tensile strength, limit of elasticity and ductility shall be determined from a standard test-piece cut from the finished material.

On tests cut from other material, the test-piece may be either the same as for sheared plates, or it may be planed or turned parallel throughout its entire length, and, in all cases where possible, two opposite sides of the test-piece shall be the rolled surfaces. The elongation shall be measured on an original length of 8 in., except as modified in section (12), paragraph (*c*). Rivets, rounds, and small bars shall be tested of full size as rolled.

Two test-pieces shall be taken from each melt or blow of finished material, one for tension and one for bending; but in case either test develops flaws, or the tensile test-piece breaks outside of the middle third of its gauged length, it may be discarded and another test-piece substituted therefor.

Annealed Test-Pieces.

(4) Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material shall be similarly treated before testing.

Marking.

(5) Every finished piece of steel shall be stamped with the blow or melt number, and steel for pins shall have the blow or melt number stamped on the ends. Rivet and lacing steel, and small pieces for pin-plates and stiffeners, may be shipped in bundles securely wired together, with the blow or melt number on a metal tag attached.

Finish.

(6) Finished bars shall be free from injurious seams, flaws, or cracks, and have a workmanlike finish.

CHEMICAL PROPERTIES.

(7*a*) Steel for— Buildings, . . . Train sheds, . . . Highway bridges and similar structures, .	maximum phosphorus, .10 per cent.

7*b*) Steel for—

Railway bridges, maximum phosphorus, .08 per cent.

Physical Properties.

(8) Structural steel shall be of three grades—Rivet, Railway Bridge, and Medium.

RIVET STEEL.

(9) Ultimate strength, 48,000 to 58,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

Percentage of elongation, $\frac{1,400,000}{\text{ultimate strength.}}$

Bending test, 180 degrees flat on itself, without fracture on outside of bent portion.

STEEL FOR RAILWAY BRIDGES.

(10) Ultimate strength, 55,000 to 65,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

Percentage of elongation, $\frac{1,400,000}{\text{ultimate strength.}}$

Bending test, 180 degrees to a diameter equal to thickness of piece tested, without fracture on outside of bent portion.

MEDIUM STEEL.

(11) Ultimate strength, 60,000 to 70,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

$$\text{Percentage of elongation, } \frac{1{,}400{,}000}{\text{ultimate strength}}$$

Bending test, 180 degrees to a diameter equal to thickness of piece tested, without fracture on outside of bent portion.

Modifications in Elongation for Thin and Thick Material.

(12) For material less than 5-16 in. and more than ¾-in. in thickness, the following modifications shall be made in the requirements for elongation:

(*a*) For each increase of ⅛-in. in thickness above ¾-in., a deduction of 1 per cent shall be made from the specified elongation, except that the minimum elongation shall be 20 per cent for eye-bar material and 18 per cent for other structural material.

(*b*) For each decrease of 1-16-in. in thickness below 5-16-in., a deduction of 2½ per cent shall be made from the specified elongation.

(*c*) In rounds of ⅝-in. or less in diameter, the elongation shall be measured in a length equal to eight times the diameter of section tested.

(*d*) For pins made from any of the before-mentioned grades of steel, the required elongation shall be 5 per cent less than that specified for each grade, as determined on a test-piece, the centre of which shall be 1 in. from the surface of the bar.

Variation in Weight.

(13) The variation in cross-section or weight of more than $2\frac{1}{2}$ per cent from that specified will be sufficient cause for rejection, except in the case of sheared plates, which will be covered by the following permissible variations:

(*a*) Plates $12\frac{1}{2}$ lbs. per square foot or heavier, up to 100 in. wide, when ordered to weight, shall not average more than $2\frac{1}{2}$ per cent variation above or $2\frac{1}{2}$ per cent below the theoretical weight. When 100 in. wide and over, 5 per cent above or 5 per cent below the theoretical weight.

(*b*) Plates under $12\frac{1}{2}$ lbs. per square foot, when ordered to weight, shall not average a greater variation than the following:—

Up to 75 in. wide, $2\frac{1}{2}$ per cent above or $2\frac{1}{2}$ per cent below the theoretical weight; 75 in. wide up to 100 in. wide, 5 per cent above or 3 per cent below the theoretical weight. When 100 in. wide and over, 10 per cent above or 3 per cent below the theoretical weight.

(*c*) For all plates ordered to gauge, there will be permitted an average excess of weight over that corresponding to the dimensions on the order equal in amount to that specified in the following table:

TABLE OF ALLOWANCES FOR OVERWEIGHT FOR RECTANGULAR PLATES WHEN ORDERED TO GAUGE.

Plates will be considered up to gauge if measuring not over 1-100-in. less than the ordered

The weight of 1 cubic inch of rolled steel is assumed to be 0.2833 pound.

PLATES ¼-INCH AND OVER IN THICKNESS.

Thickness of Plate. Inch.	Width of Plate.			
	Up to 75 in. Per cent.	75 to 100 inches. Per cent.	Over 100 to 115 inches. Per cent.	Over 115 inches. Per cent.
¼	10	14	18	....
5/16	8	12	16	....
⅜	7	10	13	17
7/16	6	8	10	13
½	5	7	9	12
9/16	4½	6½	8½	11
⅝	4	6	8	10
Over ⅝	3½	5	6½	9

PLATES UNDER ¼-INCH IN THICKNESS.

Thickness of Plate. Inch.	Width of Plate.		
	Up to 50 inches. Per cent.	50 to 70 inches. Per cent.	Over 70 inches. Per cent.
⅛ up to 5/32	10	15	20
5/32 " 3/16	8½	12½	17
3/16 " ¼	7	10	15

STRUCTURAL CAST-IRON.

Except when chilled iron is specified, all castings shall be tough grey iron, free from injurious cold-shuts or blow-holes, true to pattern, and of a workmanlike finish.

Sample pieces, 1 in. square, cast from the same heat of metal in sand moulds, shall be capable of sustaining on a clear span of 4 ft. 8 in. a central load of 500 pounds when tested in the rough bar.

SPECIAL OPEN-HEARTH PLATE AND RIVET STEEL.

Testing and Inspection.

(1) All tests and inspections shall be made at the place of manufacture prior to shipment.

Test-Pieces.

(2) The tensile strength, limit of elasticity and ductility shall be determined from a standard test-piece cut from the finished material.

On tests cut from other material, the test-piece may be either the same as for sheared plates, or it may be planed or turned parallel throughout its entire length, and in all cases, where possible, two opposite sides of the test-piece shall be the rolled surfaces. The elongation shall be measured on an original length of 8 in., except as modified in section (12), paragraph (*c*). Rivet rounds and small bars shall be tested of full size as rolled.

Four test-pieces shall be taken from each melt of finished material, two for tension and two for bending; but in case either test develops flaws, or the tensile test breaks outside of the middle third of its gauged length, it may be discarded and another test-piece substituted therefor.

Annealed Test-Pieces.

(3) Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material shall be similarly treated before testing.

Marking.

(4) Every finished piece of steel shall be stamped with the melt number. Rivet steel may be shipped in bundles securely wired together, with the melt number on a metal tag attached.

Finish.

(5) All plates shall be free from injurious surface defects and have a workmanlike finish.

Chemical Properties.

(6*a*) Flange or boiler steel,	maximum	phosphorus,	.06 per cent.
	"	sulphur,	.04 " "
(6*b*) Extra soft and fire-box steel,	maximum	phosphorus,	.04 per cent.
	"	sulphur,	.04 " "

Physical Properties.

(7) Special open-hearth plate and rivet steel shall be of three grades—Extra Soft, Fire-Box, and Flange or Boiler Steel.

EXTRA SOFT STEEL.

(8) Ultimate strength, 45,000 to 55,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

Elongation, 28 per cent.

Cold and quench bends, 180 degrees flat on itself, without fracture on outside of bent portions.

FIRE-BOX STEEL.

(9) Ultimate strength, 52,000 to 62,000 pounds per square inch.

Elastic limit, not less than one-half the ultimate strength.

Elongation, 26 per cent.

Cold and quench bends, 180 degrees flat on itself, without fracture on outside of bent portion.

FLANGE OR BOILER STEEL.

(10) Ultimate strength, 55,000 to 65,000 pounds per square in.

Elastic limit, not less than one-half the ultimate strength.

Elongation, 25 per cent.

Cold and quench bends, 180 degrees flat on itself, without fracture on outside of bent portion.

BOILER RIVET STEEL.

(11) Steel for boiler rivets shall be made of the extra soft grade specified in paragraph No. (8).

Modifications in Elongation for Thin and Thick Material.

(12) For material less than 5-16-in. and more than ¾-in. in thickness, the following modifications shall be made in the requirements for elongation:

(*a*) For each increase of $\frac{1}{8}$-in. in thickness above $\frac{3}{4}$-in., a deduction of 1 per cent shall be made from the specified elongation.

(*b*) For each decrease of 1-16-in. in thickness below 5-16-in., a deduction of $2\frac{1}{2}$ per cent shall be made from the specified elongation.

(*c*) In rounds of $\frac{5}{8}$-in. or less in diameter, the elongation shall be measured in a length equal to eight times the diameter of section tested.

Variation in Weight.

(13) The variation in cross-section or weight of more than $2\frac{1}{2}$ per cent from that specified will be sufficient cause for rejection, except in the case of sheared plates, which will be covered by the following permissible variations:

(*a*) Plates $12\frac{1}{2}$ pounds per square foot or heavier, up to 100 in. wide when ordered to weight, shall not average more than $2\frac{1}{2}$ per cent variation above or $2\frac{1}{2}$ per cent below the theoretical weight. When 100 in. wide and over, 5 per cent above or 5 per cent below the theoretical weight.

(*b*) Plates under $12\frac{1}{2}$ pounds per square foot, when ordered to weight, shall not average a greater variation than the following:

Up to 75 in. wide, $2\frac{1}{2}$ per cent above or $2\frac{1}{2}$ per cent below the theoretical weight. 75 in. wide up to 100 in. wide, 5 per cent above or 3 per cent below the theoretical weight. When 100 in. wide and over, 10 per cent above or 3 per cent below the theoretical weight.

(*c*) For all plates ordered to gauge there will be permitted an average excess of weight over that corresponding to the dimensions or the order equal in amount to that specified in the following table:

TABLE OF ALLOWANCES FOR OVERWEIGHT FOR RECTANGULAR PLATES WHEN ORDERED TO GAUGE.

Plates will be considered up to gauge if measuring not over 1-100-in. less than the ordered gauge.

The weight of 1 cubic inch of rolled steel is assumed to be 0.2833 pound.

PLATES ¼-INCH AND OVER IN THICKNESS.

Thickness of Plate. Inch.	WIDTH OF PLATE.			
	Up to 75 Inches. Per cent.	75 to 100 inches. Per cent.	Over 100 to 115 inches. Per cent.	Over 115 inches. Per cent.
¼	10	14	18	....
5/16	8	12	16	....
⅜	7	10	13	17
7/16	6	8	10	13
½	5	7	9	12
9/16	4½	6½	8½	11
⅝	4	6	8	10
OVER ⅝	3½	5	6½	9

Plates under ¼-inch in Thickness.

Thickness of Plate. Inch.	Width of Plate.		
	Up to 50 inches. Per cent.	50 to 70 inches. Per cent.	Over 70 inches. Per cent.
⅛ up to $\frac{5}{32}$	10	15	20
$\frac{5}{32}$ " $\frac{3}{16}$	8½	12½	17
$\frac{3}{16}$ " ¼	7	10	15

SPECIFICATION FOR WORKMANSHIP.

Inspection.

(1) Inspection of work shall be made as it progresses, and at as early a period as the nature of the work permits.

(2) All workmanship must be first-class. All abutting surfaces of compression members, except flanges of plate girders where the joints are fully spliced, must be planed or turned to even bearings, so that they shall be in such contact throughout as may be obtained by such means. All finished surfaces must be protected by white lead and tallow.

(3) The rivet-holes for splice plates of abutting members shall be so accurately spaced that when the members are brought into position the holes shall be truly opposite before the rivets are driven.

(4) Rollers must be finished perfectly round, and roller beds planed.

Rivets.

(5) The pitch of rivets in all classes of work shall never exceed 6 in., nor sixteen times the thinnest outside plate, nor be less than three diameters of the rivet. The rivets used shall generally be $5/8$, $3/4$, and $7/8$-in. diameter. The distance between the edge of any piece and the centre of a rivet-hole must never be less than $1\frac{1}{4}$ in., except for bars less than $2\frac{1}{2}$ in. wide. When practicable it shall be at least two diameters of the rivet. Rivets must completely fill the holes, have full head concentric with the rivet, of a height not less than .6 the diameter of the rivet, and in full contact with the surface, or be countersunk when so required, and machine-driven wherever practicable.

Punching.

(6) The diameter of the punch shall not exceed by more than 1-16 in. the diameter of the rivets to be used, and all holes must be clean cuts without torn or ragged edges. Rivet-holes must be accurately spaced; the use of drift pins will be allowed only for bringing together the several parts forming a member, and they must not be driven with such force as to disturb the metal about the holes.

(7) Built members must, when finished, be true and free from twists, kinks, buckles, or open joints between the component pieces.

Eye-bars and Pin-holes.

(8) All pin-holes must be accurately bored at right angles to the axis of the members, unless otherwise shown in the drawings, and in pieces not adjustable for length no variation of more than 1-32 of an inch will be allowed in the length between centers of pin-holes; the diameter of the pin-holes shall not exceed that of the pins by more than 1-32 in., nor by more than 1-50 in. for pins under $3\frac{1}{2}$ in. diameter. Eye-bars must be straight before boring; the holes must be in the center of the heads and on the center line of the bars. Wherever eye-bars are to be packed more than $\frac{1}{8}$ of an inch to the foot of their length out of parallel with the axis of the structure, they must be bent with a gentle curve until the head stands at right angles to the pin in their intended positions before being bored. All eye-bars belonging to the same panel, when placed in a pile, must allow the pin at each end to pass through at the same time without forcing. No welds will be allowed in the body of the bar of eye-bars, laterals, or counters, except to form the loops of laterals, counters, and sway rods; eyes of laterals, stirrups, sway rods, and counters must be bored; pins and lateral bolts must be finished perfectly round and straight; and the party contracting to erect the work must provide pilot-nuts where necessary to preserve the threads while the pins are being driven. Thimbles or washers must be used whenever required to fill the vacant spaces on pins or bolts.

Annealing.

(9) In all cases where a steel piece in which the full strength is required has been partially heated, the whole piece must be subsequently annealed. All bends in steel

must be made cold; or if the degree of curvature is so great as to require heating, the whole piece must be subsequently annealed.

Painting.

(10) All surfaces inaccessible after assembling must be well painted or oiled before the parts are assembled.

(11) The decision of the engineer shall control as to the interpretation of drawings and specifications during the execution of work thereunder; but this shall not deprive the contractor of his right to redress, after the completion of the work, for an improper decision.

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