

## SECONDARY STRESSES IN FRAMED STRUCTURES

BY E. W. PITTMAN.\*

---

Secondary stresses in framed structures are due, primarily, to faulty details. Attention will be directed to some of the more common faults and inconsistencies that are of frequent occurrence in structural details, and an effort made to illustrate their effects upon the strength of the structures.

In the general design of an articulated structure, such as a bridge or roof truss, it is assumed that the axes of the various members meeting at a joint are concurrent; that is, intersecting at a common point, and that they are free to rotate about this point as elastic deformation takes place.

In the case of a pin connected truss, the assumed conditions are very nearly realized, but in the case of a riveted truss, the last condition is not fulfilled. The riveted joint fixes the direction of the members at their ends, and when the structure deflects under a load, all members are placed in double curvature.

The computation of the resulting bending moments in the members is a rather tedious process, as it involves the determination of the angular displacement of each joint. For bridge trusses of ordinary proportions, the deflection is small, and the resulting bending stresses in the members may be safely neglected, but for the shallow trusses with deep gusset plates, they should be considered.

This condition of secondary stress is sometimes further accentuated by faulty joints, such as shown in Fig. 1. The axes of the members are noncurrent, and a bending moment is, therefore, induced at the joint. All members are bent in op-

\* Chief Engineer, Pittsburgh Steel Construction Co.

Presented before the Structural Section, January 5, 1909. Address of retiring Chairman.

posite directions at their ends, and by approximately the same amounts. This places them in double curvature and makes a point of contra flexure, or zero moment, at their centers. All members, therefore, resist the bending moment due to eccentricity in proportion to their relative rigidities. The angular displacement of the joint is the same for all members meeting at the joint, and is resisted by all, acting as beams fixed at one end, the joint, and free at the other end, the middle point, where the moment is zero. The angular displacement at the joint then is the deflection of the middle point of any member,

divided by the half length of that member, or: 
$$a = \frac{M_1 l}{3 E I}$$

where  $M_1$  is the bending moment, resisted by one member,  $l$  its half length,  $E$  its modulus of elasticity, and  $I$  its moment of inertia.

From the above, we have 
$$M_1 = \frac{3 E I a}{l}$$

Since  $E$  and  $a$  are the same for all members, it is seen that the total bending moment is divided among the several members in proportion to their respective values  $\frac{I}{l}$

In order to make this result more tangible, and to illustrate the effect of this construction, let us assume an actual case and derive the numerical values of these secondary stresses.

Fig. 1 shows a joint in the top chord of a Warren truss. The make-up and properties of the several members are marked in the figure. Taking  $A$ , as a center of moments, we have for the total bending moment, due to eccentricity,  $35\ 600 \times 7.5 = 267\ 000$  in. pounds. Apportioning this between the four members meeting at the joint according to their values of  $\frac{I}{l}$  it is found that each chord section resists a bending moment of 97 000 in. pounds, and each web member resists a bending moment of 36 500 in. pounds. The extreme fibre stress  $f$ ,

which these bending moments induce, in the members, is given below:

$$\text{For Chords } f = \frac{M y}{I} = \frac{97\,000}{26.94} = 14\,400 \text{ lb. per sq. in.}$$

$$\text{For Web Members } f_s = \frac{M y}{I} = \frac{36\,500 \times 2.75}{6.76} = 14\,850 \text{ lb. per sq. in.}$$

Thus it is seen that the secondary stresses due to eccentricity are one and one-half times as great as the primary stresses, which alone were considered in proportioning the members.

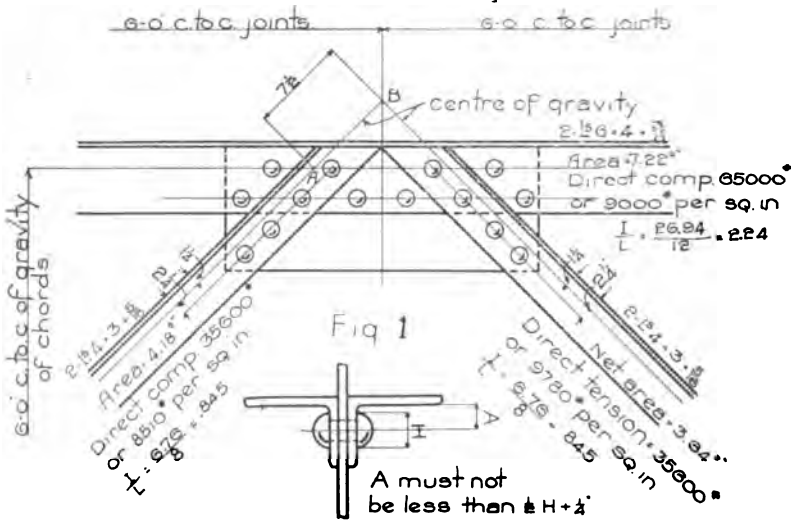
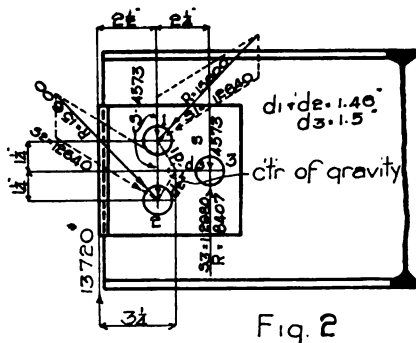


Fig 1a

Another condition which tends still further to increase the secondary stresses in the web members is the eccentricity of the rivet lines to the center of gravity axes of the members. This eccentricity is  $l$ , as marked, and the resulting bending moment is 35 600 in. pounds. The general equation for extreme fibre stress for compression member with fixed ends, is

$$f = \frac{M y}{I - \frac{P l^2}{32 E}} = \frac{35\,600 \times 2.75}{35\,600 \times 9216 - \frac{896\,000\,000}{32 E}} = 15\,320 \text{ lb. per sq. in.}$$

Particular attention is directed to this result, because this eccentricity of rivet line to center of gravity axis is a fault of very common occurrence in all types of riveted structures. Where angles are used to resist direct stress, and connected through one leg only, the gauge line for the rivets should be set in as close to the back of the angle, or as near to the center of gravity axis as possible. This matter is of fundamental importance, and yet it is habitually disregarded in detailing structural work.



It is customary to use so-called "standard gauges" for angles, pitching the rivets from the back of the angle a distance somewhat greater than the half width of the leg. The rivet clearance for machine driving is shown in Fig. 1 *a*. In the case of the web members just discussed, the dimension, *A*, would be 15/16 in. Adding to this the thickness of the outstanding leg, we obtain 1 ¼ in. as the permissible gauge of these angles. This coincides exactly with the center of gravity axis of the angles, and if the rivets were so placed, the fibre stress of 15 320 lb. per sq. in. would be entirely eliminated.

Rivets in eccentric connections are sometimes subjected to secondary stresses very much in excess of what they are designed to resist. A good illustration of this is afforded by standard connections for beams. Fig. 2 shows the standard connection for a 10-in. beam 25 lb. section. The Manufacturers' Hand Books give 9½ ft. as the minimum span length for which this connection may safely be used with a beam loaded

to its full capacity. From the table of safe loads, we find that a 10 in. beam, 25-lb. section,  $9\frac{1}{2}$  ft. long, will sustain a uniform load of 13.72 tons, giving an end reaction of 13 720 lb., as shown in Fig. 2. This end reaction may be replaced by an equal force parallel thereto, and passing through the center of gravity of the rivet cluster, and a couple with a moment:

$$M = 13\,720 \text{ lb.} \times 3.25 \text{ in.} = 56\,290 \text{ in. lb.}$$

Each rivet in the cluster is subjected to a direct stress,  $\frac{13\,720}{3} = 4573 \text{ lb.}$ , and a stress due to bending moment.

The stress in any rivet due to bending moment varies directly as its distance from the center of gravity of the cluster, and its resisting moment varies as the square of this distance. Calling,  $a$ , the stress in a rivet due to bending at a unit's distance from the center of gravity, we have the equation:

$$M = a (d_1^2 + d_2^2 + d_3^2)$$

$$\text{transposing. } a = \frac{M}{d_1^2 + d_2^2 + d_3^2} = \frac{56\,290}{6\,513} = 8\,650 \text{ lb.}$$

Now the stress in each rivet due to bending is equal to this figure, multiplied by the distance of the rivet from the center of gravity.

$$S_1 = 8\,650 \times 1.46 = 12\,640 \text{ lb.}$$

$$S_2 = 8\,650 \times 1.46 = 12\,640 \text{ lb.}$$

$$S_3 = 8\,650 \times 1.5 = 12\,980 \text{ lb.}$$

These forces are drawn in the figure, and combined with the forces  $S=4573 \text{ lb.}$  The resultant stress on rivets 1 and 2, is 15 600 lb., as shown.

The web thickness of a 10-in., 25-lb. beam is .31 in. The bearing area of a  $\frac{3}{4}$  rivet is, therefore,  $.31 \times .75 = .2325 \text{ sq. in.}$

15 600 pounds divided by .2325 = 62 100 lb per sq. in. bearing stress on web of beam. That this is excessive can hardly be denied. Let us hope that in the next issue of the Manufacturers' Hand Books, this table giving minimum span length for which standard connections may safely be used, will be revised.

Fig. 3 shows a joint in a riveted Pratt truss that is of common occurrence. Here the axes of the members are con-

current, but the rivet connection through the chord is eccentric to the intersection of the lines of stress, and a bending moment results. The proper construction of this joint is as shown in Fig. 4.

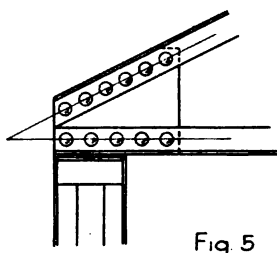


Fig. 5

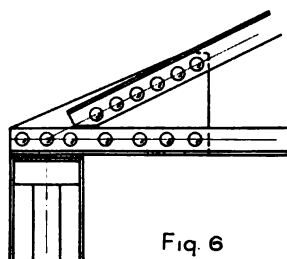


Fig. 6

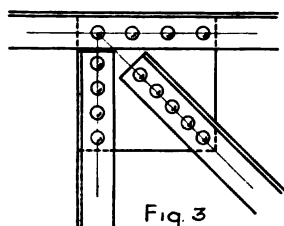


Fig. 3

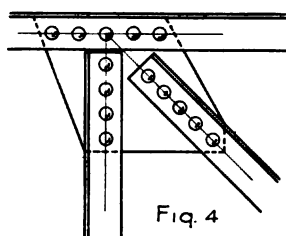


Fig. 4

Fig. 5 shows the heel of a roof truss. This detail has been made familiar by its wide use, and yet the fault is pronounced. The three forces acting at the heel, namely the compression in the rafter, the tension in the bottom chord and the column, or wall, reaction are non-concurrent. A bending moment results which induces large fibre stresses in the members. This detail is susceptible of the same analysis as the eccentric joint of the Warren truss.

Fig. 6 is, likewise, an improper detail unless the heel plate is thick enough to resist the bending moment between the point of intersection of the three forces and its attachment to the members. The plate should also be planed or chipped flush with the backs of the angles of the bottom chord when it is not possible to get sufficient rivets immediately over the column to transmit the total reaction into the plate.

Fig. 7 shows an efficient and proper detail for the heel of a roof truss.

The practice of using  $\frac{1}{4}$ -in. and  $\frac{5}{16}$ -in. gusset plates in roof trusses is very common, yet considerations of economy, as well as efficiency, would seem to dictate the use of thick plates. The plates should be of such thickness that the bearing value of a rivet in the plate is about equal to the value of the rivet in double-shear. This would reduce the number of rivets at a joint by nearly one-half, and reduce the size of the plate correspondingly. Whatever slight increase in weight the thicker plates entail is more than compensated by the reduction in rivets. The use of smaller plates and fewer rivets also measurably reduce the secondary bending stresses in the members due to fixity of their ends. This is quite an advantage, and would justify the use of thick plates aside from any other consideration.

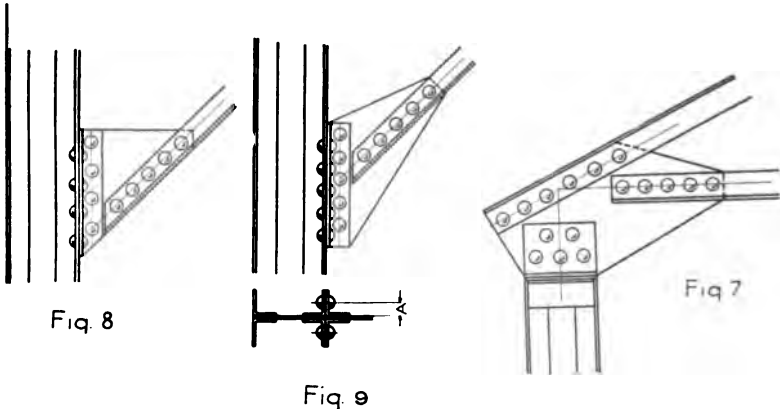


Fig. 8 shows the detail of a knee brace connection to a column, which is not uncommon in mill building construction. This detail is open to the same criticism as the other eccentric connections already discussed. It is especially to be condemned in view of the fact that the knee brace is subject to tension, as well as compression, and when the knee brace is in tension, the entire stress must be resisted by two rivet heads. Fig. 9 shows the proper detail for this connection. The gauge,  $A$ , for the rivets connecting the knee to the column flange should be as small as possible, and the thickness of the con-

nection angles should be such that their moment of resistance at the rivets is equal to the bending moment. This bending moment is equal to one-half the horizontal component of the stress in the knee brace, multiplied by  $A$ .

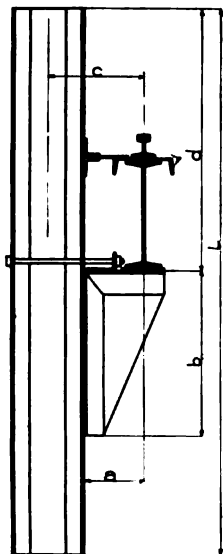


Fig. 10

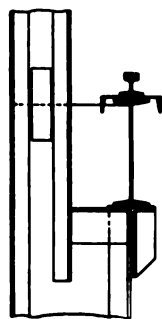


Fig. 11

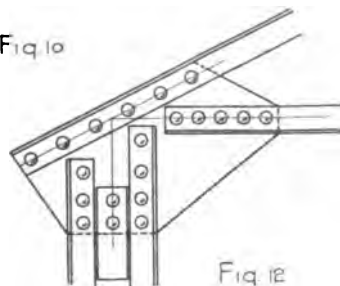


Fig. 12

Fig. 10 shows the detail of a bracket for the support of a crane runway girder. As usually detailed, this style of support has a dangerous weakness, and it has come to be regarded with distrust. When the bracket is correctly detailed, however, and all forces properly provided for, it affords an economical and efficient support for light crane runway girders.



Through bolts should be used at the top of the bracket capable of resisting a stress equal to  $\frac{Pa}{b}$

The load  $P$  being eccentric to the axis of the column, bending moments  $M$  and  $M_1$  are induced.

$$M = \frac{Pc}{l} \times d \qquad M_1 = \frac{Pc}{l} \times (L - (b - d))$$

These bending moments, in the case of light cranes, are usually much less than the bending moment at the foot of the knee brace due to wind load, and the bracket attachment requires only a small increase in the moment of resistance of the column. Herein lies the economy of the bracket support over the direct column support, as shown in Fig. 11.

The metal in the wide web plate below the crane seat takes the crane reaction and relieves the bending moments in column due to this reaction, but it does not measurably increase the moment of resistance of the column at the point of maximum bending moment; that is, at the foot of the knee brace.

This leads to a consideration of what is, perhaps, the most common fault in mill buildings with knee braced bents, and that is the inefficiency of the column at the foot of the knee brace. In a very large proportion of the mill buildings, as ordinarily constructed, the column above the crane seat is made from six to ten inches wide, regardless of theoretical requirements, and in most cases the columns are insufficient to resist the bending moments due to the wind load for which the building purports to have been designed. Most specifications for mill buildings that are regarded as standard require that the structure be designed to withstand a wind pressure of 20 to 40 lb. per vertical square foot. Nevertheless it is probable that half the knee braced mill buildings standing to-day would actually collapse under a wind pressure of 10 or 15 lb. per sq. ft. In view of this fact, an assumed wind pressure in excess of 20 lb. may well be regarded as absurd.

If the columns and knee braces of a mill building about 60 ft. wide with 20-ft. bays and 40 ft. high to the chord were prop-

erly proportioned to resist a wind pressure of 30 lb. per sq. ft., the result would be startling. The columns at the foot of the knee brace would be from 20 to 24 in. deep, and the knee brace, chords and main web members of the truss would be correspondingly massive.

Purchasers of mill buildings seem to derive some satisfaction in specifying high wind pressures, but they usually seem satisfied to accept the design submitted by the lowest bidder. It is hardly necessary to add that this design is made in utter disregard of the specifications. While a designer is, perhaps, justified in disregarding absurd requirements in specifications, there is certainly no justification in many, if not most, of the designs for high mill buildings.

This stricture applies with particular force to such construction as is shown in Fig. 12. This construction is sometimes used, in lieu of a knee brace, in order to economize head room and to avoid obstructing the crane trolley travel. This is a gross and flagrant fault. The knuckle plate should never be used as a substitute for the knee brace in a building high enough for a crane.

Knee braces, at best, are not very efficient, and they should be resorted to only when there is no better method of bracing a building to withstand the horizontal wind pressure.

When a building is of indefinite length, or subject to future extension, knee braces are necessary, as each bent must be self-sustaining, and transmit all of its portion of the wind load to the foundations direct.

In the case of a building of fixed length, however, it is generally more economical to make the bottom chord lateral system a horizontal truss to transmit the wind loads to the gable ends of the building, and thence through diagonal bracing, to the foundations. In this case the eave struts are the chords of the horizontal truss, and they should be made stiff enough to act as compression members, unsupported for the panel length.

Fig. 13 shows a bottom chord lateral system suited to this condition. In all cases, whether knee braces are used or not, the bottom chord lateral bracing should be made continuous

in order to insure good alignment for the columns. This is very important, especially where traveling cranes are used.

Figs. 14 and 14a show two systems of continuous bottom chord bracing, either of which will serve the purpose of aligning the tops of the columns, and, therefore, the crane runways.

Fig. 15 shows discontinuous bottom chord lateral bracing which is not uncommon. Nevertheless it is a glaring fault, and should be avoided even in the cheapest buildings without cranes.

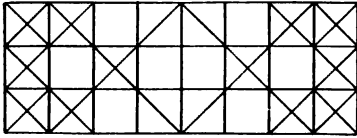


Fig 13

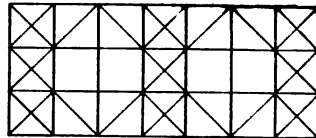


Fig 14

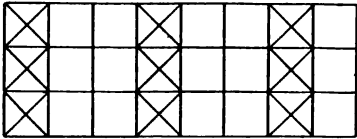


Fig 15

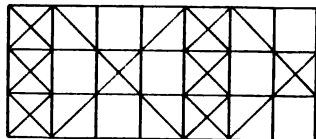


Fig 14a

A few years ago, there came under my observation a building with discontinuous bottom chord bracing where high speed, heavy cranes were in use. Acute trouble developed in the use of the cranes due to bad alignment of the runways. Operation of the cranes was suspended for a few days, and the master mechanic of the plant undertook to correct the trouble by rectifying the alignment of the rails. This was done with a transit, and new holes were drilled in the flanges of the runway girders, where necessary, and the rails clamped in place. On completion of the work, all rails were straight from end to end of building, and no further trouble was anticipated. Operations were resumed, and at the end of a few days, there was a recurrence of the same old trouble. It was discovered that the rails were as badly out of line as ever. The master mechanic was non-plused, and in his uncertainty he was overwhelmed with sug-

gestions from interested employees. Some suggested the reinforcing of the columns below the crane seat, and some suggested the tearing down of the building and its total reconstruction. Sane advice finally prevailed, however, and the trouble was permanently cured by the simple expedient of making the bottom chord bracing continuous throughout. The rails and clips were replaced in their former position, and the building was pulled into line and held there by means of nuts and turnbuckles on the bottom chord lateral rods.

In a building of indefinite length, the function of the bottom chord bracing is simply to prevent the lateral movement of adjacent bents relative to each other, and to reduce the unsupported length of the bottom chords of the trusses. This second function is important because in knee braced buildings the bottom chord is subjected to compression stresses, due to wind action, sometimes in excess of the dead load tension stresses, and it must, therefore, be designed as a strut, as well as a tie.

Of late, there has been a marked tendency toward heavy construction in mill buildings. This is manifested in the many new specifications in which low unit stresses are specified, and in which it is provided that no metal of a less thickness than  $\frac{1}{8}$  in. or  $\frac{3}{8}$  in. shall be used in the structure. Of course, this provision is designed to procure a stronger and more stable structure, but it fails woefully in its purpose.

It serves only to concentrate metal and weight in parts of the structure where it does absolutely no good. The use of high unit stresses, and the use of  $\frac{1}{4}$  in., or even  $\frac{3}{8}$  in. metal is not undesirable, if the building is scientifically designed and all details intelligently worked out. The destruction of mill buildings by corrosion is not nearly so rapid as the destructive action of racking forces due to insufficient bracing and faulty details. Unless all the various forces that may act on a building are considered and proper provision made for their resistance, the building will rapidly deteriorate, and soon rack itself to pieces, however low we take our unit stresses, and however thick we make our metal.

In conclusion, it is urged that cognizance be taken of some of the more common faults in existing mill buildings, and steps be taken to prevent their perpetuation.

### DISCUSSION.

MR. W. G. WILKINS: Mr. Pittman's remark about a building supposed to be designed for a wind pressure of 30 lbs., which he thought would collapse under a very much less wind pressure, reminds me of a remark one of my classmates made to me some years after we graduated. He said, as engineer for the Railroad Commission, in a large number of the railroad bridges in a certain State he had calculated the strains in a large number of the railroad bridges in that State, and from the results of his figures he could not understand why many of them had not fallen down long ago, but they were still standing and carrying teams over them every day.

MR. WILLIS WHITED: It might be well to remind the younger members of the Society that it is not well to place too much reliance on the flexibility of pin-connected joints, especially in old bridges. I have in mind a viaduct whose columns were pin-connected at top and bottom, which, by the settlement of a pier under an adjacent span, was pushed about 6 in. out of place, throwing the columns that much out of plumb, but, instead of the columns hinging at the bottom, as they should have done, they were rusted so firmly into the shoes that, being well anchored to the masonry, one side of the coping was lifted about  $1\frac{1}{2}$  in. off the pier. This, of course, involved raising the span about  $\frac{3}{4}$  in.

Speaking of knee braces, I call to mind a building about 75 ft. high, in which the knee braces consisted of two 4 by 3 by  $\frac{5}{16}$  in. angles, while the member of the roof truss, which connected to the bottom chord at the top end of the knee brace and had to carry practically all the stress from the knee brace to the other members of the truss, consisted of one 2 by 2 by  $\frac{1}{4}$  in. angle.

As to internal stresses, practically every piece of every

structure contains numerous internal stresses due to punching, riveting and straightening.

MR. HERMANN LAUB: Secondary stresses in framed structures are not only due to faulty design, but principally to imperfect workmanship. The abutting joints of compression members are not always accurate, especially on large sections, and must therefore produce eccentric or secondary stresses in adjoining members. Likewise the pin holes cannot be bored accurately, perpendicular to the bridge axis of chord and post sections, which again will cause eccentric or secondary stresses after being connected up to the structure. Such imperfections in the workmanship, which can hardly be avoided, are the weakest spots of our pin-connected bridges and may be of very serious consequences on large spans. This is accentuated by the fact that we do not know to what extent of accuracy the work is or can be done, which makes it difficult to take proper precautions by additional reinforcement of sections.

Secondary stresses are sometimes beneficial to the structure for the sake of stiffness and rigidity. Railroad companies now-a-days build bridges up to 200 ft. spans of riveted trusses, which involve at the joints far more secondary stresses than on pin connected bridges. On ordinarily light roof trusses we introduce knee braces so as to make the structure safe against lateral forces, but we know that such struts are the cause again of secondary stresses in roof trusses and posts to which these knee braces are attached.

These secondary stresses can never be avoided, but must be calculated and taken care of in the most efficient manner, especially if the structures are very large.

MR. G. H. DANFORTH: I ran across a case the other day that shows how some people imagine eccentric loading may be avoided. Up in New York State, a job as designed called for a column to be connected to a beam that passed some 15 inches to one side of the column center. The drawing room in detailing put riveted brackets on the sides of the column and rested the beam on the brackets, making a good stiff connection on the column.

The architect on the job condemned the connection at once, and would not have it "on account of the eccentricity of the loading." He made a detail to avoid this eccentricity which consisted of substituting a diaphragm riveted into the web of the plate and angle column for the brackets riveted into the flanges of the column. This, in his opinion, avoided the eccentric loading, but as the point of application of the load remained the same, it would require peculiar reasoning to show how the bending moment on the column was reduced in any way, and certainly the column was not re-enforced. The connection was made as requested, as the matter was not of sufficiently serious nature to call for a protest, but it all forms a rather humorous commentary on present efforts of well intentioned people to avoid conditions that are liable to lead to serious results.

MR. J. A. McEWEN: I noticed a statement in one of the hand books that where a sufficient snow and dead load were assumed the lateral wind pressure could be ignored in trusses up to 100 ft. span. I think, however, that is a very radical statement. The question of taking care of wind stress is one on which there are a great many different opinions.

MR. P. S. WHITMAN: We have heard a careful review of the present theory governing the action of secondary stresses. There can be no doubt of the truth of the mathematical deductions; yet how are we to account for the fact that every day one sees structures built in utter disregard of the theory of secondary stress, which still continue to stand up, often carrying external loads much above the limits for which sections have been designed. How are we to account for such contradictory phenomena? The only logical answer is that certain conditions working for safety must exist of which our theory takes no account. In my mind the saving condition is the friction between adjacent parts. The pieces of steel are closely held together by tension on the rivet heads, thus producing a much greater internal resistance than the actual shearing value of the rivets. As this frictional resistance is difficult to estimate mathematically and as its action is always

on the side of safety it seems to be the practice to ignore it completely. However, I think that therein lies the secret why our buildings stand up instead of falling down as they theoretically should.

Mr. Pittman's paper dwelt at considerable length on theory and value of knee braces in a mill building. On this point there seems to exist a wide diversity of opinion. Quite a common mill building detail is to omit the knee braces. The columns in such buildings are frequently called to take heavy bending moments from crane runway girders attached to brackets; which stress must be taken care of by the tensile strength of the column flanges in addition to the wind pressure. In such a design it is apparent at once that the only actual condition which prevents the building from falling over is the stress in the anchor bolts at the base of the column. With a good system of continuous bottom chord bracing the local wind and crane loads at any given column are uniformly distributed to every column base in the structure. No steel in a building can be used to better advantage than that in a good design of continuous bottom chord bracing. The saving grace of such a system does not seem to be generally appreciated. The building without knee braces is simply another example of the fact that our theory considers the stress taking direct path only; while as a matter of fact it may be taken care of entirely, through an indirect path.

MR. H. S. PRICHARD: In calculating stresses in trusses and bracing, it is the general practice to assume articulated, frictionless joints, and the intersection at a single point of the axes of all the members meeting at each joint; and it is the further practice to term the stresses so determined "primary."

Not so many years since the determination of the primary stresses was all that was considered necessary, even in cases where it was evident that the assumptions were quite different from the real conditions. In the eighties scarcely anyone but so-called "cranks" paid much attention to arranging riveted connections so that truss members would intersect in common points at the nodes, or to placing pin holes in the centers of



gravity of end posts and top chord sections, composed of channels and cover plates; even after comparative tests at the Watertown Arsenal showed that columns composed of two channels latticed both sides stood more total load than columns composed of similar channels, but with cover plates in place of one set of lattice, when the pins were placed in the center line of the channels instead of placing them in the center of gravity of the cover plates and channels combined.

There has been a marked and commendable tendency of late years to make the actual construction conform more nearly to the theoretical assumptions, where it is practicable to do so without loss of stiffness, and, where it is not practicable to follow the assumptions, to consider the effect of departures therefrom. The paper of the evening is a valuable contribution and should help to establish good practice in these regards.

In the general sense in which the author has used the term secondary stresses, it includes all the stresses which make up the difference between the primary stresses and the actual stresses which the assumed static load would produce; whether due to deformation, to deliberate eccentricity, or to imperfect workmanship or construction.

The method of determining the secondary stresses due to eccentricity in beam connections, which the author has so clearly explained, is similar to the method used and published in an article on standard connections of beams\* by the speaker, while he was Engineer of the New Jersey Steel and Iron Company.

The author has expressed the hope that his method of proportioning beam connections will be adopted in place of those now in use. The speaker entertained a similar hope when he published his article on the subject in 1895, and subsequently he wrote to *Engineering News*† criticising the tables of strength of beam connections, given in manufacturers' hand books, and the methods by which they were computed. To this Mr. Christie, for A. & P. Roberts Co.,‡ and Mr. Thackray,

\* *Engineering News*, 1895, Vol. I, p. 318.

† *Engineering News*, 1896, Vol. II, p. 43.

‡ *Engineering News*, 1896, Vol. II, p. 203.

for Cambria Iron Company, replied by making tests, which they published, claiming that they refuted the speaker's criticism. This stimulated the speaker to make a few tests for the New Jersey Steel and Iron Company, which were published in a letter,\* which is here reproduced in part, as follows:

"It is well to state the requirements for safe connections. At 16 000 lb. extreme fiber stress, a steel beam has a factor of safety of about two, as regards the beginning of failure, and about four, as regards complete failure; supposing it to fail by bending. The factor of safety required for the connections

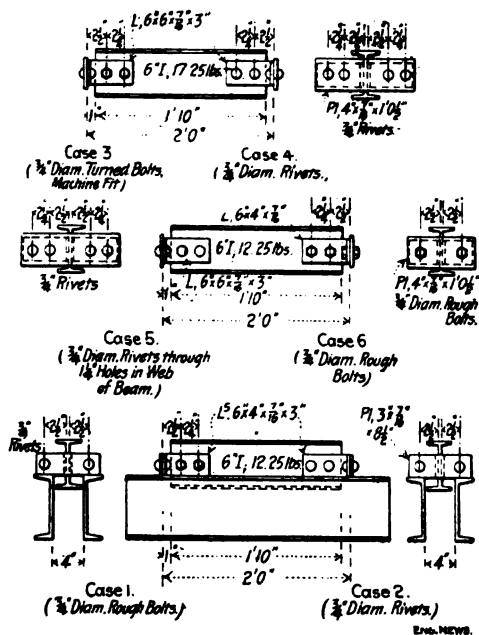


Fig. 16

are illustrated in the accompanying cuts, Fig. 16. The outstanding legs of the connecting angles were riveted to the flat connecting each pair in all cases except No. 6, in which they were bolted. The connection angles were 6 by 4 by 7/16 in. for Cases 1, 2 and 6, and 6 by 6 by 7/16 in. for Cases 3, 4 and 5. The beams were 6 in. 12 1/4 lb. per ft., with webs 0.23 in. thick

\* Engineering News, 1898, Vol. II, p. 203.

for Cases 1, 2, 5 and 6, and were 6 in.  $17\frac{1}{4}$  lb. per ft., with webs 0.46 in. thick for Cases 3 and 4.

In Case 1 the connection angles were bolted to the beam with rough bolts, forced to a bearing before the test, and the nuts were screwed up to a gentle touch only. In Cases 2 and 4 the connection angles were riveted to the beams. In Case 3, the connection angles were bolted to the beam with turned bolts, which fitted the holes, and the nuts were screwed up to a gentle touch only. In Case 5 the connection angles were riveted to the beam, but the holes in the beam were made  $\frac{1}{2}$  in. larger than the diameter of the rivets to prevent the rivets from getting any bearing on the beam, the object being to test the frictional resistance from the clamping power of the rivets. In Case 6 the connection angles were bolted to the beam with rough bolts forced to a bearing before the test, and the nuts were screwed up tight so as to give all the frictional resistance practicable. The rivets and bolts were all  $\frac{3}{4}$ -in. diameter, the rivets were machine driven, and the holes were punched  $13/16$  in. diameter, except those for the turned bolts and those in the web of the beam in Case 5.



Fig. 17.

It was intended to test the connections only and not the beams, and to insure the beam from failing distributing flats were placed between the top flange and the pressure edge of the testing machine. In Cases 1 and 2 the flats were not added till the beam began to fail.

The results of the tests are given in the table below, to which is added a comparison between the calculated safe loads

for the connections and those indicated by each test separately considered.

### RESULTS OF TESTS FOR STRENGTH OF STANDARD BEAM CONNECTIONS.

Case.	Load at the beginning of failure, lb.	Ultimate Load. lb.	Safe Load—		
			Indicated by Test. lb.	Calculated by—	
				Usual Method. lb.	Writer's Method. lb.
1.1 .....	2 000	24 900	1 000	6 900	1 470
2.2 .....	13 000	.....	6 500	6 900	1 765
3.3 .....	9 500	17 790	4 450	13 800	2 940
4.4 .....	12 500	40 310	6 250	13 800	3 530
5.5 .....	2 000	.....	1 000	.....	.....
6.6 .....	4 000	24 700	2 000	6 900	1 470

<sup>1</sup>Beam split from hole "a" to edge at 22 800 lb., after ultimate had been reached.

<sup>2</sup>Did not fail under load of 24 900 lb.

<sup>3</sup>At 17 790 lb. bolt "a" sheared off.

<sup>4</sup>At 35 000 lb. cracking noise; cause not discovered. At 40 310 lb. rivet "a" sheared off.

<sup>5</sup>Frictional resistance test, carried to slipping point only.

<sup>6</sup>At 22 000 lb. one clip began to crack; at 24 700 lb. bolt "a" sheared off.

In calculating the safe loads by the usual method the bearing value for both rivets and bolts is taken at 20 000 lb. per sq. in. In calculating by the writer's method, the bearing for bolts was taken at 18 000 lb. per sq. in. and for rivets 21 600 lb. per sq. in., to agree with the article on beam connections in Engineering News of May 16, 1895. In obtaining the safe load indicated by each test a factor of two was used with regard to the beginning of failure, and of four with regard to complete failure.

The point at which the connections began perceptibly to rotate with reference to the web of the beam was taken as the beginning of failure. In Case 5 it was possible to obtain this point easily and accurately because after the point was reached the pressure on the machine remained stationary for a few moments. In the other cases, however, it is probable that the results are a little high, as it is difficult to perceive by simple

observation the very slight movement which accompanies the beginning of the crushing of the bearing surfaces.

In arranging for the tests the beams were first supported at the ends by resting the flats connecting the outstanding legs of the connection angles on a pair of channels. After the connection at one end of a beam had failed, a support was placed under the beam at that end.

The load was applied midway between supports in each case, and one-half the pressure indicated by the machine was taken as the load on a connection. In each case the connection angles rotated about an axis perpendicular to the web of the beam, as shown in the photograph, Fig. 17, bringing pressures in opposite directions on the two bolts or rivets connecting the clips to the beam. That the pressure on the bolt or rivet nearest to the edge of the beam was much greater than on the other one was shown by the amount the holes enlarged and by the shearing of the bolts and rivets. The connection angles at each end of each beam also rotated in a plane perpendicular to the web of the beam, and in opposite directions, as shown in the photograph, so that their tops approached each other, producing a toggle joint action, and firmly clamped the beam. The effect of this was to weaken the connection of the outstanding legs to the flat and strengthen the connection to the web of the beam. The clamping pressure developed was in some cases so great that the webs of the beams were crushed a full  $1/32$  in., and a bearing thus secured at the upper edges of the clips. The strength added to the connection to the web of the beam by this toggle-joint action appears to have been the chief element in causing the high ultimate strength which most of the connections developed. It is probable that it also added considerable strength to some of the connections before failure began. This is indicated by the fact that in Case 2, in which the chance for toggle-joint action was greater and the bearing surface of the rivets less than in Case 4, the point at which failure began was higher than in Case 4. The writer's theory also neglected the partial fixedness of the ends and the frictional resistance from the clamping effect of the rivets and bolts. If the ends were partially fixed it would have some in-

direct as well as direct effect on the strength of the connections by modifying the toggle-joint action, but the writer's experiments have not covered this point. The frictional resistance from the clamping power of the rivets and bolts amounted to 2000 lb. in Case 5. In Case 4, by comparison with Case 3, in which the nuts simply touched, it was probably about 3000 lb., and in Case 6, by comparison with Case 1, about 2000 lb.

That the loads at which failure commenced in Cases 1 and 6, were small as compared with the other cases, is probably due to the fact that the bolts did not fill the holes and consequently had very little bearing at the start (theoretically only a line of bearing.) As the bolts would get their full bearing before failure had progressed very far, the safe load indicated by the tests can hardly be regarded as a fair criterion. That Case 4, in which rivets were used to connect the clips to the beam, developed so much greater ultimate strength than Case 3, in which turned bolts were used, was due partly to the fact that the rivets were initially tight, while the bolts were not, partly to the fact that the rivets were steel, while the bolts were iron, and partly to the fact that the great strength of the rivets made them last longer and thus enabled a greater toggle action to be developed.

The most seemingly astonishing fact was that in Case 1, the ultimate strength was much greater than in Case 3, notwithstanding the fact that in Case 1 rough bolts were used, against turned bolts in Case 3, fewer rivets connected the outstanding legs of the clips to the flats, and the bearing surface of the bolts was 50 per cent less. This fact is probably explained by the greater toggle-joint action permitted by the construction in Case 1.

Mr. Christie's, Mr. Thackray's and the writer's experiments, considered together, show conclusively that the strength of a connection cannot always be obtained by simply multiplying the nominal strength of one rivet (or bolt) by the number of rivets in the connection, and that the strength of a connection depends not only on the bearing and shearing strength of the rivets (or bolts), but also on a number of other

elements. Whether or not it is good practice to rely at all on these elements of strength is an open question."

The speaker is decidedly of the opinion that it is best not to rely on them, especially in view of the possibility of loose rivets. The number of rivets in a beam connection is small at best, sometimes as low as two, and one loose rivet may cause a large proportional loss in efficiency. It is to be hoped that the author will have more success than the speaker had in persuading manufacturers to reduce the amounts indicated in their handbooks as the safe loads for beam connections.

A method of analysis similar to that which the author uses so well in dealing with eccentric beam connections, indicates that even when all the axes of the various members joined by a common connection plate, intersect in a common point, there will be some modification of the stresses if a line of rivets by which a member is connected is placed eccentrically; but the greatest bending moment at any point, instead of being the product of the eccentricity of the line of rivets and the combined longitudinal component of all of the rivets, is the product of the eccentricity and the longitudinal component of, usually, one rivet. A general proof of this proposition is complicated, but it can be easily demonstrated, and the general principles can be readily illustrated by the specific case shown in Fig. 18. Three members, each composed of two 3 in. by  $\frac{1}{8}$  in. bars, are connected to a common plate, their center lines intersecting in a common point  $C$ , and each making angles of 120 degrees with each of the others. Two of these members are connected to the plate by rivets located on their center lines, which are likewise their axes. The third, a vertical member  $CF$ , is connected by a line of three rivets with one inch eccentricity to the right of the axis and spaced  $4\frac{1}{2}$  in. on centers. Conceive three forces of 18 000 lb. each, in the direction of the members, to be applied to them at points in their axes at their free ends. These forces, according to the conditions of the problem, will form a balanced system. The member  $CF$ , separately considered, is held in equilibrium by the downward vertical force of 18 000 lb. at  $F$ , and forces applied at the rivet points  $A$ ,  $B$  and  $D$ . According to the

theory outlined by the author in dealing with beam connections, each of the forces at *A*, *B* and *D* will have an upward vertical component of  $18\,000\text{ lb.} \div 3 = 6000\text{ lb.}$ ; the forces at *A* and *D* will have horizontal components  $(18\,000\text{ lb.} \times 1\text{ in.}) \div 9\text{ in.} = 2000\text{ lb.}$  (acting to the left at *A* and to the right at *D*), while at *B*, which is the center of gravity of the group of rivets, there will be no horizontal component.

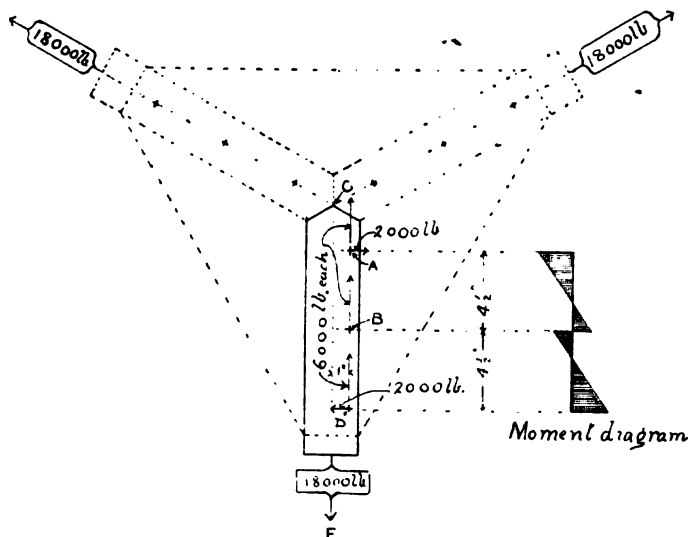


Fig. 18.

As the force at *F* is applied at the axis in the direction of the axis, and as there is no other force applied between *F* and *D*, there will be no bending moment from *F* to *D* but simply a direct tension of 18 000 lb. or  $18\,000 \div (\text{area} = 1) = 18\,000\text{ lb. per sq. in.}$  of gross section. From *D* to *B*, the direct tension will be  $18\,000\text{ lb.} - 6000\text{ lb.} = 12\,000\text{ lb.}$ , and there will be a bending moment of 6000 in. lb. at *D*:

$$[(6000\text{ lb.} \times 1\text{ in.}) - (2000\text{ lb.} \times 0\text{ in.})]$$

Which gradually decreases to zero and then increases in the opposite direction to 3000 in. lb, at *B*:

$$[(6000\text{ lb.} \times 1\text{ in.}) - (2000\text{ lb.} \times 4\frac{1}{2}\text{ in.})]$$

At *B*, by reason of the one inch eccentric application of an



additional upward force of 6000 lb., the moment takes a sudden shift to 3000 in. lb. in the former direction, and then gradually changes toward *A* to 6000 in. lb., in the reversed direction, but shifts to zero at *A*, as indicated in the diagram in Fig. 18.

The intent of this analysis is not to contend for the absolute accuracy of the method used in making it, but to refute the theory that the member with the eccentrically placed rivets will at any point be subject to a bending moment of the entire direct force times the eccentricity. The assumption that the vertical components of the forces from the three rivets will be equal is not strictly accurate, the eccentric position of the holes will affect the position of the axis, the concentration of pressure at the points of application of the rivets will intensify stresses, and practical considerations, such as loose rivets, may affect the actual stress distribution. In general it is good practice to place the rivets as nearly symmetrical with regard to the axis as possible.

In addition to the secondary stresses pointed out by the author, the speaker calls attention to the bending stresses which floor beams produce in the vertical posts to which they connect; the bending of floor beams and the stresses in stringers produced by the endeavor of the floor system to share in the chord stresses; the stresses in the end connections of floor beams and stringers produced by the effort of the connections to fix the ends; the bending in stiff chords from the deflection of the trusses; and the bending in chords and columns from imperfect butt joints.

In the design for the new Pittsburgh & Lake Erie Railroad bridge at Beaver, Pa., expansion joints in the stringers occur at every second or third floor beam, to avoid secondary stresses from the endeavor of the floor system to share in the chord stresses.

In view of the formidable array of frequently unconsidered stresses of largely unknown amount which the margin of safety is expected to take care of, it is well that there are some circumstances which mitigate the penalty when the said margin is over worked and the elastic limit of the material is, in consequence, exceeded. In many cases it is only a small por-

tion of the material which is overstrained and this portion simply yields to the force it cannot resist, and thereby either relieves the structure of the over-stress or diverts it to other and stronger lines of resistance. The overstrained metal usually does not break but simply becomes plastic as regards the excess stress and recovers from this temporary fatigue with increased ability to resist the kind of stresses which originally overstrained it. There are other cases in which secondary stresses cannot be relieved by overstraining and it is therefore highly desirable that engineers should have a thorough understanding of the principles involved to aid them in forming correct judgments.

MR. E. W. PITTMAN: The method of deriving the fibre stress in the angles, Fig. 1, due to eccentricity of rivet line to center of gravity axis has been questioned, and attention directed to the fixity of the ends as a condition tending to reduce this fibre stress. The equation used takes this into consideration and is perfectly general and rigidly correct.

The general equation for fibre stress due to bending moment in fixed end members is

$$f = \frac{M y}{I \pm \frac{P l^2}{32 E}}$$

In the case of free end members

$$f = \frac{M y}{I \pm \frac{P l^2}{10 E}}$$

In both of these equations the plus sign in the denominator is to be used in the case of tension members and the minus sign in the case of compression members. The second member in the denominator takes into account the small increment of bending moment due to the deflection of the member, equal to direct stress multiplied by deflection. Applying these formulae to the web members of the truss shown in Fig. 1, we obtain the following values:

Compression, free ends,	f = 17 500 lb. per sq. in.
“ fixed “	f = 15 300 “ “ “ “
Tension, free “	f = 12 300 “ “ “ “
“ fixed “	f = 13 700 “ “ “ “