Economic Development of Bridge Construction

Civil Engineering

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ECONOMIC DEVELOPMENT OF BRIDGE CONSTRUCTION

BY

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PREFACE.

The relative merits of the different types of simple railway bridges are well understood, but very little has been written treating the subject directly. As the subject is an important one any bridge engineer should be a master of it.

The author feels the necessity of a thorough understanding of this subject for his future career, and takes the opportunity to study it when good libraries are accessible for compilation. He compiles this work to treat on the general design of railway bridges, and does not expect to deal with detail construction. It would be well to have a study of the relative merits of all the different types of bridges, but as such a study is exceedingly broad, only the relative merits of simple steel railway bridges are considered. The word "simple" is used to include bridges whose stresses are statically determinate, cantilever bridges excepted, and the word "steel" is used to mean Bessemer or open hearth steel which are sometimes called ingot iron.

The importance of the subject lies in the fact that certain types of steel railway bridges are most economical and that different types are best adapted for different conditions. Although the subject is well understood among experienced engineers, there is much discrepancy in many important points. Such discrepancy should not exist; but, fortunately it gradually disappears by selection and compromise, until now, the general design is pretty much standardized. This is particularly true in the United States and Canada, while in Europe the engineer is freer to devise or modify the general design. But American bridge engineering is leading the World as
indicated by many large bridge building enterprises, and annual output of bridges which is greater than that of any two other countries put together. So the study of this subject in this country is a most happy one.
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1.

INTRODUCTION

The art of modern bridge building is a new one. It may be considered as dating back only to the middle of the Nineteenth Century, but this art is now in a fair degree of perfection, in spite of its late beginning. This is greatly due to the demand of efficient structures to carry the heavy locomotive and train across water and depressions of the earth. This demand set the genius at work to carry the railroad over numerous rivers, deep gorges and valleys.

Requirements of economy and strength in railway bridges hastened the art of bridge building to perfection. The loads of human beings and valuable cargoes are required to be safely carried. The enormous amount of expensive material employed in numerous necessary bridges makes the good use of material imperative. The engineers are thus induced to do their best to save money in bridge construction and to insure the success of the structures.

The possibility of the present state of the art of bridge building also depends upon discoveries and inventions in other lines of engineering. Among these, the productions of cheap Bessemer and open hearth steel are the most important, as steel is the best material for bridges in most cases. This is because the material is reliable and has definite strength which may be determined. Its use is also due to the ease in manufacturing into desired shapes of various dimensions, together with the reasonable life given to the structure on which the material is employed.

Inventions of machines for fabrication, and handling are also very important for the development of bridge construction.
In the early days, such machines were crude and small, and large bridges were hard to build, and sometimes even impossible, but now the large modern bridge shops can fabricate and handle any size of structure that may be desired by the engineer.

Indeed, economy of material is attained by using material wisely. This is due to the study of Mechanics of Materials which itself has developed on account of bridge building. However, the mathematics involved in simple bridge construction is not new except for the applications. All the theories concerned are the principles of parallelogram of forces and the principle of the lever, which were known to the ancients. Thus with the aid of learning and modern inventions, modern bridge building becomes very economical and scientific. It is a science as well as an art.
CHAPTER 1. DEVELOPMENT OF TYPES.


Many different types of simple bridges have been built. Each type was invented with some features that were, at least by the inventor, supposed to be good. The few best types are continually employed, while the poorer ones have become obsolete on the principle of "the survival of the fittest." As the good features of a bridge are simple characteristics of the structure, relating to principles of Statics, Mechanics of Material and Economics, applications of such principles determine the development and extent of usage of the types. A few of the most important of these principles will be given and others will be mentioned at different places in this work as may be necessary and convenient.

The most important principles concerning the beam and plate-girder bridges, is the theory of the beam which is well presented in any treatise on Mechanics of Materials.

1. For equilibrium,

   (a) the resisting shear in the beam is equal to the vertical shear, and

   (b) the resisting moment in the beam is equal to the bending moment.

2. For economy,

   (a) material must be so distributed that the quotient of the moment of inertia of the cross-section of the beam divided by the distance of the fibre furthest from the neutral axis is maximum, and

   (b) the cross-sections of the beam taken at different points
along the longitudinal axis must be proportioned by equating the bending and resisting moments, thus reducing the cross-sectional areas to the minimum.

In railroad bridges the resisting shear is generally greater than the vertical shear for economic sections designed to resist the bending moment. The bending moment has a very important influence on the design, and also on the length for which beams and plate-girders are used.

The action of locomotive and car wheels on short beam bridges would be the action of concentrated loads if the wheels rolled directly on the beams, but the rails and ties between the wheels and beams distribute the loading considerably and the bending moment is thereby reduced. When ballast is used for the bridge floor the concentrated loads are further distributed, and as the length of the bridge increases the effect of the wheels becomes more like that of a uniform load of the same total weight.

For a uniformly distributed load, the maximum bending moment varies directly as the square of the span length. This maximum bending moment is at the middle of the span, when the bridge is fully loaded, the moment at the ends being zero. The moment curve is in the form of a parabola with the vertex in the middle of the beam and it passes through the ends, as shown in Fig. 1.

Let \( M \) denote maximum bending moment; \( L \), span length; \( w \), load per unit length; \( S \), allowable unit stress for the material of the beam;
5.

I, moment of inertia of the section at the point of maximum moment; and let c denote distance of fibre in the section furthest from the neutral axis. Then

\[ M = \frac{1}{3} w l^2, \quad S \frac{1}{c} = M, \]

and the sectional modulus

\[ \frac{1}{c} = \frac{w l^2}{2S} \]

The square of a small number is small and it increases slowly. Thus, the sectional modulus of a short beam for uniform load is small and the structure is light. The sectional modulus of the beam for concentrated loads should be greater than for uniform load of equal total weight. For the limiting case of a single concentrated load, the maximum bending moment is greatest and is twice that for equal uniform load. But even so, the structure is light as the square of the span length is small. On the other hand, a long span beam bridge is heavy even when the maximum bending moment for actual load approaches that for uniform load. This is because the square of large numbers are large and they increase very rapidly.

However, material can be wisely distributed to slightly increase the sectional modulus without increasing the area of cross-section. This is done by moving the flanges of a girder further apart or by placing more material on the flanges and less in the web, but this process of increasing the sectional modulus is limited as the web of the beam has to be thick enough to hold the flanges firmly together. With this limitation, the weight of a long span beam bridge is great because the square of the span length is great, and as the square of the span length increases in a manner unfavorable for long span beam bridges, the beam and plate-girder are only used for short spans.

For short spans the beam is rolled, and the cross-section is uniform throughout the length, but for long spans the depth of the
beam is great enough for an economic distribution of material. As the web is to be thin for economy, it can not very well be made by rolling. The beam is, therefore, built up of a plate and four angles riveted to its longer edges, and is called a plate-girder. The theory of the beam also applies to such a built up structure. In practice the plate-girder is proportioned by considering only a small part (about one-eighth) of the web plate as contributing strength to resist the bending moment. The material in each flange and the small fraction of the plate near the flange is regarded as in the center of gravity of that flange. Moments are taken with the origin of moment at the center of gravity of one or the other flange. Let $M$ denote the resisting moment; $S$ the allowable unit fiber stress; $a$, the area of one flange and a small fraction of the area of the web plate; and let $d$ denote the distance between the center of gravity of the flanges. Then the resisting moment,

$$M = Sad.$$ 

In this case the product of the depth and area of one flange is $ad\frac{Wl^2}{85}$ using notation previously given. The depth of the plate girder is generally made so as to minimize the amount of material, but still the squares of the span lengths increase very quickly for long spans, and the use of plate-girders for long spans is uneconomical.

Besides the angles of the plate-girder, cover-plates are usually added to the flanges as shown in Fig. 51. The number of cover plates may be decreased at sections taken toward the ends of the plate-girder to economize material according to the decrease of the moment. By making the resisting moment equal to the bending moment in this way material is saved. Still, the magnitude of the square of a large number prohibits the use of the plate-girder for
long spans. For longer spans the truss is employed.

The truss is a combination of straight members arranged into triangles in such a way that the members perform their function by resisting longitudinal stresses.

Care is always taken to prevent the members of a truss from bending stresses. It is well to explain the reason for such precaution. If a long member is subjected to bending and compression at the same time, the liability of buckling is very great. The greatest unit stress very soon reaches the allowable limit, and the stresses are not uniform throughout the section. Let BC in Fig. 2 be the compressive unit stress due to pure compression, and AB be the compressive unit stress due to bending. The maximum compressive unit stress in the member is \( AB + BC = AC \) which becomes great by addition of the stresses. On the other hand, the resultant tensile stress \( A'C' \) is very small. Thus, very little of the material takes great unit stresses and there is a great waste of material. The effect of combining bending with tension can be shown in a similar manner. The unit stresses may be all tensile or all compressive. It is better to use two different members to resist bending and compression or bending and tension. But for axial tension and compression, it is well to so arrange the members that they act at the same time, as the resultant stress in this case is the algebraic sum of the two which is smaller than either stress.

Economy is attained by having the stress uniform throughout
the section of the member, for then all the material can be stressed
to the allowable limit, and no part of the member is less effective
than it should be. But in order to make the stresses uniform, the
load must be axial besides being longitudinal. In his paper on
"General Flexure in a Straight Bar of Uniform Cross-Section"
(Transaction American Society of Civil Engineers, Vol. LVI) L. J.
Johnson has shown how the maximum stress due to eccentric load can
easily be more than twice the average stress; and this is due to the
load being applied a little away from the center of gravity of the
section of a bar.

It is well to note that tensile stress is the simplest and
easiest to be made uniform. It is independent of the length of the
member, unlike flexure and column stresses. It does not require
material to be distributed for economy, but rather have it concen-
trated. Members under such stresses are easiest to make and main-
tain. It is the best kind of stress for a member of the truss to
take and is preferrable to other kinds of stresses. So, it is good
feature of a truss to have most of the members in tension rather
than otherwise.

On the other hand, columns are not much in favor for con-
struction. It requires a considerable amount of material to prevent
failure by buckling. There are many empirical formulae derived
from results of failures of columns which were tested to destruc-
tion. The straight line formula is the most convenient for use and
is most popular among American engineers. In the straight line
formula,

\[ \frac{P}{a} = S - k \frac{1}{a} \]

P denotes the longitudinal axial force in pounds acting on the
column; a, cross-sectional area of column in square inches; S, unit
stressed if under pure compression, in pounds per square inches; \( l \), length of column in inches; \( r \), radius of gyration in inches; and \( k \), denotes an empirical constant. The quantity \( \frac{P}{A} \) is the average unit load that is allowed on the column concerned. It is much lower than the unit stress for pure compression for short members. This is due to the presence of the term \( \frac{k}{r} \) which represents the stress due to the buckling of the column. The average unit load allowable on the column is reduced as the length of the member increases, but it is increased as the radius of gyration increases. Thus, the strength of the column of the same cross-section is less for greater length. Efforts to strengthen the column involves much work in fabrication for increasing the radius of gyration. Due to this peculiarity of column action, the average stress is often reduced to three-fourth or two-third of the value for compression of short blocks of steel. Moreover, the quantity \( \frac{l}{r} \) is not allowed to exceed 100 for the present, according to best authority, thus making long columns very heavy. Other column formulae reduce the allowable average unit stress in different ways, but they all show the column not a very desirable form of member.

The column is also undesirable for uncertain strength due to the imperfection of material and fabrication which may cause localized failure. Little kinks, bends, non-homogeneity of material etc. increase the unit stresses at different parts of the column in a very uncertain manner. The increase to maximum stresses due to such uncertain conditions is often 40 or 50 per cent of the average fibre stress at working load and in some cases are even much higher. The general assumption in the use of columns is, that
10.

the different parts of the built-up column act together, forming a unit, but this is not so, as shown by Prof. A. N. Talbot in the Transactions of American Society of Civil Engineers, Vol. LXV for the year 1909. The stresses vary both longitudinally and transeversely in the column especially in the long and light ones with lacing. Many tests have been made in recent years showing the imperfection of the column, especially in the details and lacing. And columns usually fail before the yield point of the material is reached for average fibre stress. In view of these facts, the column is not a desirable form of member and should be shortened or avoided as much as possible. Thus the truss is better when the columns it must have are fewest and shortest.

In all types of trusses the members are arranged into triangles with common sides. The lengths of the sides of the triangles are fixed except for very small changes due to elasticity of the material and temperature. No attempt is made to fix the angles of the triangles. This method of fixing the triangle is well known to the ancients as a proposition in Geometry.

When the members of the truss are so arranged that three or less members can be cut by one straight line, the stresses are statically determinate. As there are only three equations that can be written for a body in equilibrium, only a truss of such form that three or less members can be cut by a straight line can have the stresses of the members determined by statics. However, all the stresses may be determined by the Method of Least Work when the cross-sections and the modulus of elasticity of the material are known. The stresses may also be computed with assumed simple outlines of multiple intersection trusses. Whether the loading is assumed or
the elastic properties of the members are depended upon, the stresses that are statically indeterminate can not be accurately found, thus making the arrangement of members undesirable. The change of length, or inaccuracy in length of one member effects the stresses in other members, thus making the stresses different from what are calculated. For these reasons multiple intersection web systems are not in favor with modern engineers.

The use of counters have a very great influence on the development of truss bridges. The main diagonal web members carry the shear due to dead load on the bridge. They carry the positive or negative shear as the case may be. For the second panel in the truss represented in Fig. 7, the main diagonal Bc carries the dead load and positive live load shear by tensile resistance. In Fig. 8 the corresponding main diagonal bc carries the dead load and negative live load shears by tensile resistance. For parallel chord trusses the relation of the signs of shears and main diagonals is the same as for the former case (as shown in Fig. 7), as this may be regarded as a special case with the top chord panel points much above the parabola of the bowstring truss.

When the main diagonal is designed as a tension rod to carry positive shear, it cannot carry negative shear, and vice versa; but when the same member is designed to carry shear of both positive and negative sign the member has to be made as a column with provision to carry tension. Such a design is usually undesirable, both for undesirability of the column and the use for a double purpose which involves complicated joints, but for riveted trusses when stiffness is greatly desired and the length is not great, the design for such double purpose is very desirable.
Fig. 3.

Fig. 4.

Values of Cosecant

Inclination Angle in Degrees.
13.

Usually, the main diagonals in a truss are tension members and pure tension is insured by looseness in the pin-joints. The shears opposite to those which produce tension in the main diagonals are resisted by diagonal members at inclinations opposite to those of the main diagonals. Such diagonals are called counters. The counters in Figures 7 and 8 are shown in dotted lines. Both positive and negative shears exist for every panel of a truss except the two at the ends, but a counter is necessary only when the dead load stress in the main diagonal is greater than the live load stress of opposite sign or the curve of the chords is such as to require it. Thus, counters are often unnecessary for panels near the ends of the bridge. Also, they are often used throughout the trusses to insure stiffness and safety against unexpected loads.

By principle of Statics, the stress in an inclined member of a truss is directly proportional to the vertical force acting upon it and to the cosecant of the angle of inclination. As shown in Fig. 3, let $Gd$ be the truss member in question; $V$ be the vertical force, and $\alpha$ be the angle of inclination. Then, the stress,

$$S = V \csc \alpha$$

It is well known by Trigonometry as shown in Fig. 4 the cosecant of a large angle is small and approaches unity as the angle approaches $90^\circ$. It is large and increases rapidly when the angle is near zero and approaches infinity at zero. Thus, the stress on an inclined member of a truss is small when the angle it makes with the horizontal is large, but it is large and increases very rapidly until it is infinite as this angle is small and approaches zero. The truss is then a poor one when there are members making small angles with the horizontal plane. The diagonals of trusses
are usually at inclinations about 35° from the vertical. Formerly, the engineers, especially in England, had the peculiar notion that this angle must be 45° for best economy. But the inclination is determined roughly by the theoretical economic depth and more accurately by results of practice.

The cost of the floor system in a truss bridge is a function of the panel length as the weight of stringers that support the track depends upon the panel length. For long panels the cost is great and it increases rapidly, much like the square of the panel length. Thus, the cost of flooring for panels of 30 or 40 feet is enormous, and the cost becomes excessive for panels much longer than 30 feet. On the other hand, for shorter panels, the number of panels is greater for the same span length. The number of floor beams is increased. However, for short panels, the increase of cost due to the greater number of floor beams is much less than the increase of cost for long panels having a smaller number of floor beams and longer panels. As the number of panels are increased for decreased panel length, the number of web members in the truss is increased. Then the cost of the truss increases as the cost of flooring decreases. By experience and trial designs, most economical panel lengths have been found and are adopted. There is a rule of thumb that the structure is most economical when the weight of the stringers in one panel is the same as the weight of one floor beam.

Methods of decreasing the panel length without much increase in cost of the truss is very desirable. The use of suspenders to offer immediate supports to the flooring is a very good method as the cost of a suspender is much less than the increased cost due to
greater number of panels. This method is often adopted for the Warren truss (see Fig. 54). It is better adapted for shorter spans than for longer ones.

For long spans secondary trusses are inserted between main panels in order to offer immediate support to the flooring. This is an excellent method as the members of the little secondary trusses are only affected by the load at one panel point, and their members are short. (See Art. 5.)

The panel length can be shortened also by adopting multiple systems of webbing such as the Whipple and Warren trusses of double and triple intersection, but the benefit of this method is offset by the uncertainty of the stresses in the members of the truss. Besides, the saving in material can not be as much in the two foregoing methods as the web members are long. This method is not used now.

Equality of stress in chord and web members of a truss is a very desirable feature. In a truss with parallel chords, the maximum stresses in the chord members are smallest at the ends and greatest in the middle. The maximum stresses in the web members are greatest at the ends and smallest in the middle. The method of equalizing the stresses in the webbing is to incline the chord members so that they will take a part of the shear. This will, at the same time, approximately equalize the chord members.

Let. Fig. 6 represent a truss with horizontal bottom chord with the panel points of the top chord on a curve. The stress in any member of the bottom chord, bc, is the quotient of the moment at the point C and the distance y from C to the bottom chord. Suppose the span carries a full uniform live load which is
17.

the loading like that for maximum chord stresses in a long span. Let \( w \) be uniform load per unit length; \( y \) be the height of the panel point; \( C \) which is at distance \( x \) from the left end of the span (see Fig. 5.) Let \( l \) be the length of span; \( d \) be the maximum depth of the truss; and \( S \) be the stress in the member \( t \). The maximum stress of \( t \) is then

\[
S = \frac{\frac{1}{2}wlx - \frac{1}{2}wx^2}{y} = \frac{1}{2} \frac{w}{y} (lx - x^2)
\]

If the curve is a parabola, \( \frac{1}{2} l (1-x) = k (d-y) \) when \( k \) is a constant. When \( x = 0, y = 0 \) and the constant, \( c = \frac{l^2}{4d} \). Therefore,

\[
y = \frac{4d}{l^2} (lx - x^2)
\]

Substituting this value of \( y \) in the equation for the value of the stress,

\[
S = \frac{w}{8} \frac{l^2}{d}
\]

which is a constant. Thus, when the panel points of the top chord is in the parabola of the form,

\[
y = \frac{4d}{l^2} (lx - x^2)
\]

the stress in the bottom chord is the same throughout. The stresses in any top chord member \( CD \) equals the quotient of the moment at point \( C \) and the moment arm, \( z \). The moment to be resisted by the members \( BC \) and \( CD \) is the same in magnitude. The moment arms \( y \) and \( z \) are nearly equal. Thus, the stress in member \( BC \) nearly equals that in member \( CD \). Similarly the stresses in all the top chord members are nearly equal.

In the truss of parabolic top chord, there is no stress in the diagonal members for full uniform load, as the horizontal component of the stress of any diagonal member equals the difference of the chord stresses in the panel at its foot. The diagonals
should be used for stiffness and for sustaining partial live load, as shown in Fig. 6. The panel loads on the bottom chord are carried directly to the upper chord by the vertical web members, and the stresses in the web members near the ends of the truss become less for partial live load as the incline top chord members carry part of the shear. This truss is known as the Parabolic truss.

When the panel points of the top chord is above the parabolic curve, as shown in Fig. 7, the stresses in the chord members near the ends are less than those in the Bowstring of the same span because in the equation,

\[ S = \frac{1}{2} \frac{W}{y} (lx - x^2), \]

the quantity \( y \) is greater. As the chord stresses decreases toward the ends of the truss, the main diagonals, as shown in Fig. 7 in full lines, are under positive shear. They are in tension. In like manner when the panel points of the top chord is below the parabola, as shown in Fig. 8 the stresses in the chords are greater than those in the parabolic truss. For bridges the panel points of the top chord are placed above the parabolic curve. Both the parabolic and bowstring trusses are liable to reversal of stresses in the vertical web members. The parabolic truss is not a practical form of structure, but curve top chord trusses, which are its modification, are very useful.

The ease of construction is important as controlling the development and use of curve top chord trusses. The bevel joints of the inclined chord members are hard to make. The stresses in the members have to be transmitted through the pins for pin-connected trusses and through splicing plates for riveted trusses. In
either case the joints are clumsy since a great amount of material is required at the joints. On the other hand the straight top chords may be continuous throughout the truss for short spans and the splicing of the different members is very easily made. Straight top chords are also easier to brace laterally, but as there is economy of material in the use of curved top chords, the saving in material can only be greater than the saving of work when the structure is large. For this reason, curved top chord trusses are only used for long span bridges. The ease of erection also has an influence on the adoption of types.

These and many other less important things control the development of type of railway bridges. Those types which are best for the different conditions are used more and more until their use has become quite general, while the poorer types are used less and less until they become obsolete. The adoption and rejection of the types are due to the understanding of the relative merits of the different types. For this reason, a little history and description of the types will be given.

Art. 2. Development of the Beam.

As the early bridges were wooden, especially at the beginning of the development of modern bridge design, discussion of steel bridges can not be complete without the history of the wooden bridge. It should be further noted that the total stresses in the different members of the bridge of same general outlined is the same without regard to the constructive material.

Undoubtedly the first bridge that ever existed was merely a tree-trunk fallen across a brook. Later, logs and planks were
used to span across small streams, as these are the most convenient forms of structure. Then, piers were built with piles for several spans and the structure became a pile trestle. Very few bridges have been built by the ancients. One of the most famous of them was the Pons Sublicius, built about 621 B. C. by Ancus Martius, said to be without any iron. This was a pile trestle at Rome across the Tiber. This bridge became famous for its brave defence by Horatius Cocles. Another famous wooden trestle was that built by Julius Caesar across the Rhine about 55 B. C. In the middle ages many pile trestles were built in Europe to carry roads across the streams, but there was no development in the art of bridge building until the middle of the Nineteenth Century.

Improvements of the beam and inventions of the plate-girder were due to the understanding of the Mechanics of Materials, discussed in Art. 1. These principles were arrived at by the conception that material at the top and bottom of the beam was more effective than that in the middle of the section. With this principle recognized, many cast iron beams of T, U, and bracket sections were used for railroad bridges in England about 1840. Some of these bridges were as long as 50 feet and are in one piece.

In 1842, elaborate experiments were made on the transverse strength of cast iron beams by Hodgkinson in England. The weakness of cast iron to resist tension was discovered, and the difficulty of casting long beams was experienced by foundrymen. Knowing the weakness of cast iron for tension on the lower flange, G. P. Bidder was the first to use wrought iron straps to strengthen the lower flange of cast iron beams. This was done in a bridge over the river Lea at Toddenham, England.
From 1842 to 1844 Robert Stephenson built a cast and wrought iron bridge for the Stockton and Darlington Railway. It is represented by Fig. 9. This was the bridge over the Tees with five spans for 330 feet, the three longest spans in the middle being 89 feet each. The girders were each cast of three pieces and bolted together. Each lower flange was strengthened by wrought iron straps fastened near the top. All the five girders were bolted to each other at the ends thus making the beam continuous. Although many cast iron bridges have been built in England with wrought iron reinforcement and were successful, one bridge of 108-foot span, failed with a load. Stephenson was alarmed and inserted timber braces at the ends of the spans of his bridge. This bridge then lasted until 1906 when the load became excessive and it was taken down.

Besides being weak for tensile stress, cast iron was unreliable for possible voids and uncertainty of strength due to change of composition of the material. It also suffers the effect of internal stresses due to shrinkage and it could not be made strong at joints with the iron straps. So, its use was discontinued. Soon afterwards, wrought iron and steel came into extensive use. However, cast iron is strong for compressive stress and it is more endurable against the weather, than either iron or steel.

About 1820, the first rolled beams were manufactured and used in England as wrought iron railroad rails. Such beams of different sizes and cross-section were made and used for bridges and buildings, both in Europe and America. By 1875 I-beams up to the depth of 15 inches were made for sale. With the development of the steel manufacture in making cheap Bessemer and open hearth
steel, the dimension of rolled beams were increased. Steel came into great favor of engineers about 1890. Now I-beams of 20 inches in depth and 30 feet in length are easily obtainable in the market. Many recent deck bridges and culverts of 30 feet or less in span are made of such I-beams, as will be discussed in Art. 5.

In 1832 George Smart invented the lattice girder as represented in Fig. 10, and patented it in England. The girder consisted of two parallel flanges made of angles riveted together to flat iron bars which served as the webbing of the beam. This was an effort to save material by removing a part of it from the web of the solid beam. And when bridge designing was put into rational basis the necessary amount of material was calculated and the girder became a very economical one.

About 1840, Warren first constructed the truss known by his name. This was essentially the same as the lattice girder, except some web members were stiff for resisting compression. This type was constructed with a single as well as multiple system of webbing, and the term lattice girder is also occasionally used to denote this type.

The rolled beam is a form of superstructure for smaller spans while the plate-girder is used for longer ones. The origin of the plate-girder is incidentally due to conditions that must be satisfied. In 1846 Robert Stephenson proposed to build a railroad arch bridge across the Menai Strait in Wales for the Chester and Holyhead Railway. His proposed bridge could not meet the requirement of the government for clear head room under the bridge. Boiler plates were easily obtainable in those days, and Stephenson invented a large tube made of boiler plates which would allow the train
Fig. 11  Cross-section of Tube of Britania Bridge.

Fig. 12  General Plan of Britania Bridge.
to pass through in the inside. Experiments were made to determine the form and method of construction of the bridge. Finally the rectangular cross-section was found the best. A model of one-sixth of the actual dimensions of the bridge was tested and the bridge was made according to the model. This bridge is famous, and is known as the Britannia bridge, as its middle pier is built on Britannia Rock. It was built from 1846 to 1850.

The Britannia bridge has two main spans of 460 feet clear at the middle and two shorter spans of 230 feet. Two tracks are carried by separate tubes but by the same sub-structures. There are, then, eight tubes connected together into two long ones of 1,513 feet in all. The shorter tubes were erected with scaffolds but the longer ones were floated to the site on barges and were raised to the required place by hydraulic power. The tubes were expected to be very flexible. Large chains were designed to suspend the tubes but as they were found very rigid, the chains were omitted.

Fig. 11 shows the cross-section of one of the tubes and Fig. 12 shows the general plan of the bridge. The sides of the tubes are parallel but the height varies, being 30 feet at the middle of the bridge and 22 3/4 feet at the abutments. The bottom of the tubes are on the same level while the top is in the line of a parabola. The tubes were built of wrought iron plates with T and angle irons besides strips of flat iron bars over the joints. They are strengthened at the top by eight longitudinal cells and six at the bottom. The amount of iron employed for this bridge is enormous, being 5,240 short tons for each track.

After the Britannia bridge was finished many bridges similar to it were built. Many modifications of this type were invented.
Fig. 13  Girder of Bridge over River Swale at Maunby, England.  
Length 155 Feet.
Fig. 14.

Fig. 15.

Fig. 16.

Fig. 17.

Fig. 18.

Fig. 19.

Fig. 20.

Fig. 21.

Fig. 22.

Fig. 23.
Fig. 13 illustrates one girder of the 155-foot bridge over the river Swale at Manunby, England. This is a through bridge built in 1852. The top of the girder was made in box form because of the peculiar idea that wrought iron can not be trusted for compression. Many bridges have been built this way in the early days and this one may be considered typical.

About 1850 the plate-girder was evolved by omitting a great portion of the top and bottom of the tube of the tubular bridge. More material was concentrated at the top and bottom of the vertical walls thus forming flanges. The vertical walls were made of thin plates to make the structure light. As the vertical walls were too flexible there were stiffeners made of angles riveted vertically to the plates, and when the web plates were too long to be made of one plate they were made of several plates spliced together.

Art. 3. Development of the Truss.

The solid beam is also the origin of the truss, as it is the most convenient form of structure for primitive people. Fig. 14 represents a solid beam supporting a load, P, between two points

If the middle of the beam is deeper as shown in Fig. 15 material may be saved by making the depth at the ends less for the same load and same span. More material may be saved by causing a greater part of the material to take higher unit stresses as the material near the middle of the cross-section of the beam is only under low unit stresses. This saving may be had by building the structure with three members as shown in Fig. 16. The structure is thus made for supporting the load at the top as in the case of
a roof truss. When the load comes at the bottom of the structure, it is as represented in Fig. 17. The member CD is necessary to carry it up to the top as in the case of a bridge. This latter structure is known as the King-post truss.

The King-post truss is sometimes used to carry roads across narrow streams, even in this day, but when the span is long, say about 40 feet, it is unduly high. However, combinations of several King-post trusses may be made to carry roads over wide streams without this serious objection. About the year 1560, the great Italian architect, Palladio, built many bridges by combining King-post trusses. One notable example of his work is the bridge of 108-foot span over the torrents of Cismone, near Bassano, Italy. By examining Fig. 16, which represents the outline of this bridge, the elementary King-post trusses, ABcAB, cDedc, and eFgfe are easily seen as are also the halves of trusses, ACo and EGe.

The invention of this type is the result of efforts to carry roads across rivers with strong currents and debris which have caused many wooden trestles to have been destroyed. This structure of Palladio is much like the modern truss. But it was not noted and did not have influence on the development of modern bridge engineering.

Fig. 19 represents a King-post truss with a panel added in the middle. If two more panels are added at the middle as shown in Fig. 20, with the inclined members AC and DF, a longer truss is resulted with a small depth. If still two more panels be added, the structure would be as represented in Fig. 21. Similarly, the length of the truss may be increased by adding any even number of panels, as shown in Figs. 22 and 23.
Fig. 24.

Fig. 25.

Fig. 26.

Fig. 27.

Fig. 28.

Fig. 29.

Fig. 30.

Fig. 31.

Fig. 32 Burr Truss.
During the Seventeenth and Eighteenth Centuries many wooden bridges have been built of such general outline. In 1758 Grubenman built a great timber bridge of 366-foot span on this plan. Such a type is uneconomical for the numerous long braces are subjected to compression, and many of these are at small inclinations thus causing large stresses. Neither the length of the inclined compression members nor their inclination permit the type to be good, and this type is not used in modern times. Combinations of King-post trusses may also be made as shown in Fig. 22 and 23, but such structures are also open to the same objections as the foregoing.

Again, let Fig. 24 represent a King-post truss. Adding two panels at the ends of the truss, it becomes a structure as shown in Fig. 25. Adding two more panels at the ends, it becomes one as shown in Fig. 26. In each case the loads at the panel points effect the abutments by being carried from one panel to another toward the ends of the truss; whereas in the foregoing cases the loads are carried by the verticals to the top of the structure, and then to the abutments directly. This type of structure is economical, as the web members in compression as bC, cD, De etc. in Fig. 26 are short and at large inclination angles. This is the Howe truss which has been used extensively.

If two panels are added in the middle of the King-post truss by adding short diagonals Bc and cD it will be as shown in Fig. 28. The diagonal members will be in tension. With two more panels added in the middle the structure will be as shown in Fig. 29. This is the Pratt truss which is mostly used. It is also a very economical type as only the top chord and verticle members are
in compression, except the two vertical members near the ends of the truss which are in tension.

For best economy, the number of panels is generally odd rather than even. Both of the preceding types may have odd number of panels as shown in Figs. 27, 30, and 31. They form a system which may be called the panel system. This panel system is the most important thing in the development of the truss. It is the secret of economic bridge construction.

The development of the truss in America, began with the patent of Theodore Burr, granted April 3, 1817. This was the oldest patent on bridges on record in the United States. The general form is shown in Fig. 32. The trusses were of the same type as that shown in Fig. 26. It was a wooden through bridge with top lateral bracing. The trusses were found too flexible and an arch was inserted for stiffness. But later the arch element was omitted and counters were used to stiffen the bridge. In 1804, Burr built a highway bridge of this type over the Hudson River at Waterford, N. Y., over the Hudson River in four spans of 150, 161, 170, and 180 feet clear. All the members were of timber, and there were counters throughout the spans. However, the principle of the counters was not understood at that time.

In 1836, Stephen H. Long made an important step in the progress of bridge engineering by publishing a pamphlet explaining the function of the counter in preventing distortion of the panel of the truss under the action of live load. In the early days when Long published the work, bridges were constructed of wood. The truss members in the web could hardly be made to take tension. Under the action of counter stresses as explained in Art. 1, the
Fig. 33 Original Howe Truss.

Fig. 34 Original Pratt Truss.

Fig. 35 Whipple Truss.
truss is too flexible, but with the counters and main diagonals under initial stress as explained by Long, the truss became stiff and serviceable. The functions of the counter were soon recognized, and counters were generally adopted.

A patent was granted to William Howe, July 10, 1840 for a truss bridge. Fig. 33 illustrates this patent. Wrought iron had just come in use. The verticle web members are under tension and so are made of wrought iron. Their connections are very simple, the rods going through the chords and fastened with screw nuts and threads. The use of perpendicular tie rods is also economical as the perpendicular distance between two lines is the shortest and that wrought iron was comparatively expensive in the time of invention of the truss. The bottom chord is of wood although it is in tension. The top chord and end posts are in compression and are of wood. The truss had main and counter braces for all the panels. These braces are in compression and rested against cast iron angle blocks to prevent localized crushing of the wood. The adjusting of the bridge is made by iron rods having screw nuts and by wedge pieces so placed as to be effective by the action of the screw rods. Owing to faulty construction the Howe truss was found flexible at first, and was improved in 1846 by adding an arch to the truss. But later the arch was not used.

On April 4, 1844, Thomas W. Pratt and Caleb Pratt received a patent for the well known Pratt truss. Fig. 34 illustrates this type. It was first constructed as a combination bridge of wood and iron. The chords were both wood. The verticle web members were in compression and were wood. But the diagonal web members, both main and counter braces were subjected to tension and were iron rods
with screw nuts. This truss was just like the Howe truss except for the interchange of wood and iron for the web members due to the interchange of sign of the stresses.

With the Pratt and Howe trusses the most important forms of the truss were designed since 1840, but bridge designing had not yet been put on a scientific basis. In 1846 Squire Whipple published the first book on rational bridge design entitles "A Work On Bridge Building." Whipple may be called the father of rational bridge design. This book contained a rational discussion of the determination of stresses and the proportioning of the cross-sections of the members of a truss. There were also given methods of computation of stresses due to dead and live loads and investigations for economic depth with plans and details of the Bowstring and Whipple trusses. The Whipple truss shown in Fig. 35 is essentially different from the Pratt in that the system of webbing is double instead of single.

The book was printed by Mr. Whipple's own hand including the setting of the types and making the cuts. The edition was a very small one, very crudely printed and illustrated. For some reason or other the author suppressed most of the edition only selling a few directly, keeping many copies for himself; but fortunately, a few copies became distributed and opened the eyes of the early bridge builders to the scientific principles of their arts. However, the book was exhibited in the American Institute Fair in 1847, and later editions were extensively advertised and sold. Mr. Whipple's book was entirely original. By his own word to a friend about the book, he said, "It is believed that there is no previous attempt to reduce truss bridge construction to its simplest elements."
and to determine by exact calculation the forces acting upon the various parts of such structures and to reduce thence the proper sizes and proportions of such parts upon known and reliable principles," in those early days, "It seems to have been mostly a mere matter of cut and try, and if the thing stood, all right, and no question asked as to whether the structure contained too much material here or too little there."

Many new ideas were advocated in Whipple's important little book. Among which was the omission of the vertical end posts for through bridges. Previous to 1847, the vertical end posts and the top chord members at the end panels were built even for through truss bridges. But by Whipple's method of computation no stress in these members were found and they were omitted. The inclined end post then came into importance and the great stress in it was recognized.

The Bowstring truss has the panel points of the upper chord near the parabolic curved that passes through the highest point and the ends of the truss. This is practically the same as the parabolic curve as shown in Fig. 6. The only difference is the outline is not exactly parabolic as the latter. It was patented by Whipple in 1841, with wrought and cast iron as constructive material. It was invented in view of the advantages of curve top chord which was advocated in Whipple's book (This principle is discussed in Art. 1.) Before 1850, Mr. Whipple had built over twenty such bridges over the Erie Canal.

The Whipple truss was invented in 1847 and is as shown in Fig. 35. It was invented for its capability of shortening the panel while reasonable depth of the truss is retained without making the
inclination of the diagonal web members too small. Mr. Whipple sold his patent right of this truss to Murphy about 1859. The later built it extensively for many years both for highway and railway bridges. This truss has sometimes been called the Whipple-Murphy truss.
Fig. 36 "Permanent Bridge."

Fig. 37 Colossus Bridge.

Fig. 38 Portion of Town Truss.
CHAPTER II. TYPES OF SUPERSTRUCTURE.

Art. 4. Obsolete Types.

In the days before Whipple published his first book on bridge designing, all the bridges were proportioned by testing small models. The weak parts which were broken by tests were made stronger. Bridges were made according to the best models. Bridges designed this way did not have the stresses of their members known. Every bridge constructor had his own idea of the best type, and the early bridges were very different from each other. Many distinctly different bridges have been built and many more have been proposed and patented. Only a few will be mentioned.

The arch type exerted great influence in bridge engineering at the beginning. The famous "Permanent Bridge" which spanned the Schuylkill River in Philadelphia was a composite arch and truss bridge as shown in Fig. 36. A part of its description reads as follows: "The frame is a masterly piece of workmanship combining in its principle that of king-post and braces or trusses, with those of a stone arch". It was a wooden highway bridge with the middle span 195 feet in length and two side spans of 150 feet each. It was built from 1804 to 1806 by Timothy Palmer and was replaced by a combined railroad and highway bridge in 1850.

The Colossus Bridge over the Schuylkill River at Fairmont, in Philadelphia is another famous bridge of the early days. It was also a wooden highway bridge resembling an arch bridge as shown in Fig. 37. The span was 340 feet 3 3/4 inches clear, which was considered very great. It was built by Lewis Wernwag in 1812, but was destroyed by fire in 1838.

Both the "Permanent Bridge" and the "Colossus" bridges were
regarded as wonderful structures in their days because very few people could build them. But neither was built on good design. They were clumsy and massive as compared with modern structures. The greatest bridge engineers in the early days were Burr, Palmer and Wernwag. Each of these men built many wooden bridges.

The first bridge that was neither a suspension nor an arch bridge was that patented by Ithiel Town in 1820. It had two wooden built up beams supporting the floor. The beams consisted of long pieces of timber fastened horizontally to lattice work as shown in Fig. 38. This type of structure was very easy of construction. The material required was easily available in the early days. It consisted of timbers of uniform dimensions, and the truss has been built without iron bolts, straps or rods.

The Town bridge was popular in the early days. It was used for many bridges up to spans of 220 feet, both for highway and railroad services. Due to the bridge being well protected from the weather, some of the wooden bridges built in the early days can still be found. But this type is not a scientific one. The stresses in its members cannot be calculated on account of the manner in which the web members were built. There is much waste of material in the webbing, and the bridge is laterally flexible. On account of these disadvantages, the Town type soon became obsolete. However, the Boston and Main Railway still uses this type occasionally. In 1902 a 400-foot bridge with three spans was built on this railroad at Sheldon Junction in Vermont. This bridge is also covered like the earlier ones but iron bolts are used as they are cheap.

The Bollman truss is one composed of inverted king-post trusses with a common top chord as shown in Fig. 39. AdG is a sym-
Fig. 39  Bollman Truss.

Fig. 40  Fink Truss.

Fig. 41  Fink's Military Bridge.
metrical king-post truss. AbG, AcG, AeG, and AfG are distorted king-post trusses. The load at each panel point is carried by one king-post truss independent of the others. Thus, the load at E, is carried to e by the post Ee and then to the abutments by the ties Ae and eG, the loads at other panel points being carried to the abutments in the same way. The stresses in the top chord which is in compression is the sum of all the stresses due to the loads at all the panels at the same time. The stresses in the main members can be very easily computed. The trusses were especially convenient for the early bridge builders.

There are many advantages and disadvantages in the Bollman truss. It is an advantage that no lower chord is necessary. For inverting the king-post trusses, the compression members are short and few in number which is an advantage, but the tension members are very long which is a disadvantage. The small inclinations of the inclined members make their stresses great for shear as explained in Art. 1. Besides the stresses in the counters (as shown in dotted lines in Fig. 39) cannot be calculated. This type proved to be very expensive. It was only used for short spans.

This truss was invented by Wendell Bollman about 1850, and was often employed in the twenty years following, both for through and deck bridges. It has not been built since about 1830. The 124-foot span Bollman truss bridge built in 1852, across the Potomac River at Harper's Ferry, Virginia, is the most famous bridge of this type and is a good example. The trusses had eight panels 13 feet deep. The top chord and posts were of cast iron and the inclined members were of wrought iron. It carried railroad traffic for forty-one years and was removed in 1893. The long inclined members in
this truss corresponding to bG and Af in Fig. 39 made an angle of 9° 28' with the horizontal, thus making the stress 6.656 pounds for each pound shear. Such large stresses naturally make this type undesirable.

The Fink truss, shown in Fig. 40, consists of a principal king-post truss Ael supporting secondary ones, AcE and EgI. The secondary king-post trusses, in turn support the tertiary ones, AbC, CdE, EfG, and GhI. Stresses of the members of this truss are calculated by considering the load at the middle point of each king-post truss as being carried to its end panel points. Thus great part of the load is carried toward the center of the span before being carried to the abutments. The stresses are then very great on account of the stresses being indirect. The length of the inclined members is also great. However, the columns are short and the lower chord is not necessary, these being advantages of the type, but the counter stress cannot be computed and the counters cannot be used very well. This structure is better for longer spans than the Bollman truss as the inclined members do not make such small inclination angles, but it is a very expensive type.

This type was invented by Albert Fink in 1851. It was occasionally employed for the following thirty years. The longest bridge of this type was that built across the Missouri River at St. Charles for a span of 306 feet, 6 inches. It was built in 1871 and was used until 1884.

As Mr. Fink was an army officer in the Civil War, he made a novel use of his truss. This truss is without lower chord, and dummy members were inserted as shown with dotted lines in Fig. 41. The enemies destroyed the lower chord but the bridge still stood firmly.
This illustrates a good point of the type for war. The type is also good for military bridges as the joints can be easily made by unskilled soldiers.

Both the Bollman and Fink trusses depend on the principle of suspension rather than the principle of the beam. The effect of the loads is transmitted by the vertical posts from the top to the bottom of the truss. The lower panel points are suspended by rods or eye-bars, and the horizontal components of the stresses on the rods are resisted by the top chord. This principle is good as far as theory is concerned but it fails in practice, and both of the types are obsolete.

The Whipple truss, as shown in Fig. 35, has become obsolete, but there are many good points of this truss. This type may be considered as a modification of the Pratt truss, by employing multiple system of webbing to shorten the panel length thus making the flooring cheaper. Having the span length fixed the double webbing reduces the panel length to one-half that of the Pratt truss and triple webbing reduces it to one-third.

This type also increases the possible span length by the multiple system. As the ratio of the depth of the truss to the length is not to be smaller than one-tenth, and the panel length is limited to about 30 feet for the Pratt truss, that truss cannot be much longer than 300-feet. By doubling or tripling the web system, this length can be two or three times that for the Pratt truss without increasing the cost of flooring. At the same time the inclination of the diagonals remains great enough to keep down the shearing stresses as in the Pratt truss. The stresses in the truss members cannot be well determined on account of the multiple system as ex-
plained in Art. 1.

The intermediate posts of the Whipple truss are in compression and are as short as possible since they are at right angle to the chords. The counters which are shown in dotted lines in Fig. 35 are subjected to tension as are also the main diagonals. This truss has all the good features of the Pratt truss except for the multiple webbing.

The Whipple truss has been very extensively employed for thirty-five years following its invention in 1847, especially for long spans. There is a Whipple truss bridge of 515-foot span, on the Cincinnati Southern Railway over the Ohio River. When this bridge was built in 1877 it was the longest simple truss span. The longest Whipple trusses were those for the two spans of the Cairo Bridge erected in 1889 over the Ohio River in Southern Illinois. These spans were 518 feet 6 inches.

The Whipple truss has also been employed for short spans. Fig. 42 represents the general design of a multiple Whipple truss which was called a lattice truss by those who used it. It has been much used on the New York Central Railroad, but this is an old practice. As an exception to the rule, a few riveted Whipple truss bridges have also been built in the Western States in the last few years, but statically indeterminate stresses are not tolerated in late years, and so the general rule is not to use the Whipple truss any longer. This is especially true as the place that has been held by this type for long spans is taken by the Pettit truss which is superior in every respect, including mathematical correctness, economy of material, and appearance.

The Warren truss or lattice girder may have a multiple sys-
tem of webbing, as shown in Fig. 43 and 44, for through and deck bridges respectively. As it is always the case for trusses, the top chord and inclined end-posts are subjected to compression while the bottom chord is subjected to tension. The web members, especially those near the middle of the truss are subjected to alternate stresses caused by the live load at different positions. This requires the web members to be designed for both tension and compression which causes the joints hard to be made. Moreover, the great length and number of truss members subject to compression are not desirable. Besides, the stresses in the truss members are not statically determinate.

In the early days many multiple Warren truss bridges were built. The trusses shown in both Fig. 43 and 44 were much used on the New York Central Railroad shortly after 1862. Even in the present day, the multiple Warren truss is built to a limited extent as riveted trusses both for railroad and highway bridges. As railway bridges, it is built for spans of 150 feet or less, but it has been used for railroad bridges of long span, as those over the Missouri River at St. Charles, Missouri, the span being 318 feet. One advantage of this type is that the panels are short, thus making the flooring cheap. The stresses are distributed over many points which the early engineers regarded as advantageous for safety and the life of the structure, but present practice tends to make the panels longer, to have forces concentrated in fewer points, and to have more substantial truss members. When riveted, as they are, these types are less liable to failure than the single webbing trusses, in case of accident, but the leading bridge designers consider it unscientific, clumsy, and often unsightly.
48.

The Post truss as shown in Fig. 45, is a structure designed on a compromising principle between those of the multiple Warren and the Whipple trusses. It has a single system of counters at the middle as shown dotted in the figure, besides a double system of web members. Among these web members, Aa, Bb, Cc, etc., are struts while others are ties. The end panels ab and hi are only one-half as long as the others, thus making the inclinations of bB and Bd, cC and Ce, etc., different.

This truss was invented with the idea that there is an advantage over the Whipple truss for the load at any panel point as d being carried to A by a shorter route than for the latter while the struts are shorter than in the Warren truss. These advantages are accompanied with the difficulty of manufacture of the oblique members. Unlike the Warren truss, the stresses in the web members do not alternate, which is also a good feature. This type has been more or less popular in the early days for its apparent stiffness under moving loads, the counter being extended for the whole length. The panel points are also brought close together as in the case of the Whipple truss, due to the multiple web system, but this multiple web system is the very thing that put the truss out of use in later days.

This truss was invented by Simon S. Post in 1865 and was first built with iron for the Erie Railroad at Washingtonville on the Newburg branch. Many trusses of this type were built since then until about 1880. The longest spans are over the Missouri River at Fort Leavenworth built 1872 for both railroad and highway services. The trusses were of the triple intersection of 338 ft. span with twenty-six panels, the depth being thirty-five feet.

The Whipple, multiple Warren, and Post trusses have many advantages, but the statically indeterminate stresses are intolerable
to the engineers in late years. Besides, the amount of material in the web members is greater than in trusses of simple webbing as built in the present day. For these reasons, trusses with multiple system of webbing should not be used any longer, except for lateral systems of the Warren type, when the ambiguity of stress distribution is of little importance. Since 1380, trusses with multiple systems of webbing have practically gone out of use in the United States. They are also used less in Europe than formerly, but are still built occasionally.

The tubular bridge is distinctly an obsolete type, but it has been in great favor of engineers in England about the year 1850. Many railroad bridges have been built of just the style as the Britannia bridge, as described in Art. 2. There were also modifications of this style. A good example of these is the 225-foot span over the River Aire at Brotherton, England, built in 1350. This was on the York and North Middleland Railway, with two tubes, one for each track of a double track railroad. It was different from the Britannia bridge as there was no cell on the top and bottom of the tubes.

However, the tubular bridge has been used very little in America. There has been only one tubular bridge in Canada and one in the United States. The tubular bridge in Canada was the Victoria Bridge, built across the St. Lawrence River in 1359. The one in the United States was built in the year 1346 and 1347 by James Millholland on the Baltimore and Ohio Railway at Bolton depot. The span was 55 feet.

The tubular bridge did not become popular in this country because there is no prominent good feature to the type and because of its weaknesses. There is a great disadvantage of the tubular
Fig. 46 Section of Girder Over Staindrop Road.

Fig. 47 Section of Musgrave Bridge.
bridge on account of the lack of light and ventilation inside of the tube. The bridge is more exposed to the action of fumes from the locomotive on account of the smoke being confined inside of the tube, and the inside of the cells cannot be very well painted to prevent rust. This type is rigid as compared with other types of the early days, but it is very heavy, the material in the sides, top, and bottom being more than is necessary for other types.

For spans of about 200 feet or less, girders with box sections have also been used in England as illustrated in Fig. 13 for the Maunby bridge over the river Swale. This type is out of use. It is open to the same objection as the tubular bridge except there is more light and ventilation. The cells in the longer span bridges are made large enough for a man to enter the inside for painting, but this cannot be done for short spans.

In the early days, about 1860, cast iron plate girders were built in England by bolting cast iron on wrought iron plates for the top flange. The lower flange was, however, of wrought iron. This was due to the idea, then prevalent, that cast iron is stronger for compression while wrought iron is stronger for tension. But when the weakness of cast iron was found, this type was not employed as it was unduly heavy. The use of cast iron girders with and without iron rods is discussed in Art. 2.

Plate girders with the top flange of curve cross sections and other awkward shapes were also built in England in the early days. Fig. 46 is a section of such a girder designed by Sir Thomas Bouch in 1856. This was a 53-foot span over Staindrop Road, near Darlington. The web plate had the upper edge hog-backed and the flanges were curved. Thus, the flange plates had to be curved both
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<th>Over What River</th>
<th>Location</th>
<th>Name of Engineer</th>
<th>No of Panels</th>
<th>Depth (ft)</th>
<th>Live Load per Foot (lb)</th>
<th>Weight of Trusses in Tons of 2,240#</th>
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longitudinally and transversely. The construction was most difficult and too expensive for whatever advantage that may be attained by the different construction.

Fig. 47 is a cross-section of a girder also designed by Bouch. This was a 69-foot span over the River Eden at Musgrave built in 1862. This design is better than the foregoing as the flanges are parallel and are not curved. But there is no need of making the flanges different. When first built the bridge had no lateral stiffness. The cross-bracing shown in dotted lines was added by W. T. Cudworth in 1894, thus making the bridge more like a recent plate-girder bridge. Many different kinds of plate-girders have been tried. The simpler forms were adopted more and more until now the plate-girders are almost standardized in general construction.

Table I contains a number of interesting items concerning fourteen long span old bridges erected between 1848 and 1877 in four different countries. Some of these bridges have been described. This table is compiled partially from the Proceedings of the Institute of Civil Engineers of London, Vol. 54, p. 194 and 195. There are more of these bridges in the United States than in any other country as there are more railroads and railroad bridges here. The fourteen bridges consist of five Whipple, three tubular, two Warren, two lattice, and two lenticular bridges. None of the lenticular bridges were in the United States as that type has never been popular on account of the difficulty of construction. However, the type was used in Europe. The lattice girders also were not in this country as they are only used for short spans here. The tubular bridges were all in British territory as that type was most popular in Great
Fig. 48 Relation of Weight and Span of Bridges Given on Table I
For Live Load of 2,240 Pounds Per Linear Foot.
Britain. On the other hand, the Whipple and Warren trusses were in the United States because these types were popular in this country. All the engineers were famous in their profession as these structures were considered unusual in magnitude, at the time of their erection.

Fig. 48 is a chart showing the weights of the fourteen bridges supposing the weight is proportional to the live load per linear foot and all the bridges are designed to carry a load of 2,240 pounds per foot. This gives a good comparison, as the live load is about that which was assumed. The numbers of bridges in the chart correspond to those on the table. The chart suggests that the weights are small for short spans and are very great for long spans, which agrees with the theory of the beam. There is a great variation of the weights for spans of nearly the same length due to different specifications and materials. The chart shows distinctly that tubular bridges are much heavier than the other types. The weights of the Whipple truss bridges are smallest with reasonable variations, thus indicating the type was the best for the early days. The weights of the lenticular and lattice truss bridges vary greatly, due to the difference of the designs. At any rate, all these bridges of obsolete type are much heavier than the bridges of recent types.

Art. 5. Recent Types.

The building of short railway spans is an important matter. This is due to the great number of such structures summing up to an enormous aggregate length. Such importance may be shown by considering the condition in one of the important roads. In the year
Fig. 49 Rail Top Culvert, A.T. & S.F. Ry.

Fig. 50 I-beam Bridges of 24 Ft And 26 Ft Span.
1904, there were 7,100 miles of railroad owned and operated by the Chicago, Milwaukee and St. Paul Railway Co. There were approximately 114,000 linear feet of metallic bridges and viaducts of which 76,000 feet or two-thirds were plate girder spans mostly between 20 and 90 feet. In the same year, the total weight of steel bridges erected for this railroad was 6,000 tons of which 1,600 tons or about one-fourth was in the form of plate-girders for short spans.

Having this point in view, most railroads standardize their short spans in recent years. At any rate, all the railroads have their bridge plans more or less standardized. Many railroads, such as the Northern Railway and the Atchison, Topeka and Santa Fe Railway have complete standard plans carefully designed for plate-girder and truss bridges up to 200 or 300-foot spans. The construction and erection of these bridges is also very well planned, and the designs are revised from time to time to suit changing conditions. The standard plans have been proved to be very economical. For repeated processes of fabrication and erection, the methods of practice are improved and become easy.

For very short spans, I-beams and T-rails are used to support the railroad. Simplicity of construction is most important. Where the distance from the base of the rails of the track to the bottom of the drainage opening or stream is very small, I-beams would take up too much space and T-rails are often used. Fig. 49 is an illustration showing a rail-top culvert for a 40-inch opening with concrete base and side walls such as constructed by the Atchison, Topeka and Santa Fe Railway. There are twelve rails 6 feet long with a deck of 3-inch plank to carry the ballast. This type is convenient and economical up to the span of 8 feet in the clear.
For greater spans the number of rails is greater. The rails in the superstructure are sometimes supported on pile of treated timber. In this case, the structure is not permanent as with masonry support, but as the sub-structure can be repaired without interference with traffic, this is a good construction. As shown in Fig. 49, the ties of the road rest on ballast. Sometimes there is no ballast and the ties rest directly on the supporting rails. As the spans are small, this type of superstructure has no lateral bracing and it is stiff enough both vertically and laterally.

For spans between 8 and 26 feet, I-beams are used for the superstructure. In order to gain head room, this type is used up to 34-foot spans in exceptional cases, but between the spans of 26 and 32 feet, this type is not so economical as the plate-girder. This is clearly shown in Fig. 72, page 111, which shows the relation of weights and spans of standard bridges of the Atchison, Topeka and Santa Fe Railway. Fig. 50 is an illustration showing the construction of I-beam span. The I-beams are arranged in sets and riveted to transverse channels so as to prevent change of spacing be the action of the train. They rest on end plates and are braced by channel brackets at the ends. As constructed for the Atchison, Topeka and Santa Fe Railway, four 15-inch, 50-pound I-beams are used for a 12-foot span and ten 24-inch, 90-pound I-beams are used for a 34-foot span.

The plate-girders as built in recent years consist of four flange angles riveted to the longer edges of a thin rectangular plate which forms the web. The web is braced with stiffening angles riveted across the web plate from the top to the bottom flanges. When the span is short, the cross-section of the plate girder is
constant throughout its length. In this case there is considerable waste of material as all the sections are determined by the maximum resisting moment. When the span is larger than the usual length of plate that can be rolled, the web is formed of plates spliced together. There are usually flange plates riveted to the angles (see Fig. 51) to economize material, the number of such plates increasing toward the middle of the girder according to the increase of bending moment. It is good practice to place at least one-half of the sectional area of any flange in the flange angles. The number of cover plates should be restricted to three or four, to improve the grip of the rivets and decrease injury due to excessive reaming. There should be much material of the flanges in the angles because cover-plates would make the flanges too thick if more material be placed in them, and because when the flanges have 6 or 7 inches of cover plates, the rivets binding them to the angles are too long to have good grip.

The plate-girder may either carry the floor at the level of the upper flange or near the level of the lower flange thus making the span deck or through. For deck plate-girder bridges, the track generally rests right on the girders with the bottom of the ties on top of the upper flanges for open floors. For through bridges, the track has to be carried by floor stringers which are in turn carried by floor beams. Thus, deck plate-girder bridges are more economical and they are preferable to through plate-girders.

The web plate is usually made as thin as possible as it can well serve the purpose of resisting the shear as designed in most cases. The depth of the girder, in the case of deck spans, is sometimes made less than the economic depth to reduce the level of the
Fig. 51 Showing Parts of 64-Foot Standardized Plate-Girder Spans of Atchison, Topeka and Santa Fe Railway to the Same Scale.
track and to increase clearance to suit other lines of traffic or increase the water way.

A good example of plate-girder bridge designs is found in the standard plans of the Atchison, Topeka and Santa Fe Railway of which a few semi-sections are shown in Fig. 51. For these plans, all the webs for spans from 30 to 105 feet are made of 3/8 inch plates which is the minimum allowable thickness for any part of iron or steel structure under strain. They are open floor bridges. They are divided into four classes. The lightest plate-girder spans are deck plate-girders, known as class A, which are of economical depth with two plate-girders for each single track span. They are well braced with lateral and sway bracings, but to avoid the change of grade line of the roadway, a set of plans known as class B was made. These are also deck spans, but the girders are as shallow as possible as shown in Fig. 51 B. For spans from 30 feet to 42 feet in length, four lines of plate-girders are used for a single track bridge. They have solid plate cross bracings at intervals. They are without lateral bracing but are laterally rigid. For spans longer than 42 feet, two lines of plate-girders are used for each span with laterals and cross frames.

There is a third group of bridges called class C. They are through plate-girder spans ranging from 60 feet to 105 feet 6 inches over all. They have both the plate-girders and floor beams of economic depth. They are a little heavier than the class B spans up to 80 feet, but they serve the purpose just as well as the latter, and class B is, therefore, not built longer than 80 feet.

The fourth class called Class D consists of through spans. The girders are of economic depth and the floor beams are shallow.
In the case of a 64-foot span, the depth of floor beam is 21 inches as compared with 42 1/2 inches for Class C. (See Fig. 51 C and D). The panel length for Class C girders is shorter to give the plate-girder more support by knee braces, but for spans of 70 feet or over the distance between top of floor beam and top of girder is great and the type of construction becomes bad for the usually great number of braces. The question of economy, strength and rigidity of these plate-girders has been carefully observed by their designer and the bridges built under these plans are found to be very satisfactory.

The Warren truss is intermediate in form between framework and the plate-girder. It is now built more with single system of webbing in the styles as represented by Fig. 52, 54 and 55. It may be either deck or through. To economize material suspenders are often used to render immediate support of flooring to shorten the panels, as shown in Fig. 54. The vertical lines shown in full represent the suspenders and the dotted vertical lines represent auxiliary members that are sometimes used to shorten the theoretical length of compression members on top. This form is for through bridges. For deck bridges the truss is modified as shown in Fig. 55. As tension members require no auxiliary member for bracing, there are less vertical members than in the preceding truss. The vertical lines representing the members for immediate support are columns.

The action of the members in this truss is very simple and constitutes a good feature of the type, but the stress in the inclined web members alternate except for those near the end of the truss. Moreover, the lengths of these members are great, which is bad for columns. The alternate stresses render pin connection unsuitable as it is hard to construct such joints for both tension...
and compression. This is one reason why the Warren truss is generally riveted. This type is now built entirely of steel but formerly it was built as a combination bridge, in the Southern and Western states.

The Warren truss in its simplest form as shown in Fig. 52, is not much in favor and is only used occasionally for very short spans, but the truss with sub-verticle for deck bridges, as shown in Fig. 55, is often used for long spans. The Northern Pacific employs the type represented in Fig. 54 for through spans from 110 to 120 feet. These are called lattice girder bridges in this case. The depths of the girders are less than the panel length, and the girders are braced with brackets like that shown in Fig. 51 D for through plate-girders. The brackets are riveted to the vertical web members which correspond to stiffener angles. The chords are built up with plates and angles and the web members are simply angles, thus making the structure much like a plate-girder span.

The panel length of the Warren truss may be shortened by the use of sub-trusses as shown in Fig. 56. This form of truss bears the same relation to the simple Warren truss as the Baltimore does to the Pratt truss. It may be easily mistaken for the Baltimore truss for the general outline. The only difference is in the main inclined web members which are at opposite inclinations for alternate panels. The large suspenders Cc, Gg, etc., carry the load from the bottom to the top of the truss, but the vertical members like Ee are only used to shorten the length of the top chord columns.

The stresses in this type of truss are well determined. The stresses in the main inclined web members near the middle alternate as in the simple Warren truss. This type is limited for riveted
bridges as alternate stresses forbid the use of pin connections. It has all the peculiarities of the simple Warren type regarding the main truss.

The Warren truss with subdivided panels is not a very popular type, and but a few bridges of this type have been built. It is adapted to large spans. The longest Warren truss of this type is found in the Ohio Falls bridge on the Pennsylvania Line at Louisville, completed in 1870. The longer channel span is 398 1/2 feet and the other is 368 feet. The stresses in the sub-diagonals are peculiar, all being in tension except for the sub-diagonals at the ends. The form shown in Fig. 56 is also suitable for short riveted bridges with solid floors. The truss shown in Fig. 57 has also been used but is used no more. The multiple Warren truss shown in Fig. 53 is also obsolete as are other types of trusses of multiple webbing discussed in Art. 4.

The Howe truss was originally intended for wooden construction and later it was built with wood and iron. This type of truss is best adapted to construction with the use of both wood and iron. It is not generally included by the term "Combination Bridges" but is considered as a wooden one. It is even used in this day occasionally, whenever wood is very much cheaper and more easily available than iron or steel. This was done quite extensively in the Pacific Coast thirty years ago as the wood is excellent there and steel had to be sent from the Eastern States.

Fig. 58 represents the outline of a Howe truss with counters shown dotted. This type has very few modifications, unlike the Warren and Pratt trusses. The main diagonals slope upward toward the middle of the truss, and the stresses in these diagonals are compressive for positive shear. As wood is suitable for compression
this type is suitable when such material is previously selected for economical reasons. The counters are used to resist shear in the direction opposite to that which produce stresses in the main diagonals when such shears exceed the dead load shear. As built in the present days, the vertical web members and bottom chord are of steel while the main diagonals, counters and top chord are of wood abutting against iron blocks to prevent localized crushing. The bottom chord consists of steel eye-bars. The vertical web members are rods of steel, and the truss is pin-connected. Only one diagonal act at a time, the other being released from strain by separating by a very small distance at the joint, and as the vertical web members can never be in compression no provision is made for such stress.

The Howe truss is just as stiff as other types with the same material, but in ordinary wooden Howe truss bridges the deflection is much greater than that for steel bridges. Theoretically, the deflection in a wooden truss is about twice that for a steel truss for the same depth and span designed for the same load. This has been explained by Charles H. Nichols in the Journal of the Engineering Society of Western Pennsylvania for 1903. With modern train load, the actual deflection for a wooden bridge is about three times that for a steel bridge. The deflection of a Howe bridge of either iron or steel is then between that for a steel bridge and that for a wooden one. As stiffness is quite important for modern traffic of fast trains with heavy locomotives, this makes the Howe truss not very desirable. Besides, the life of wooden structures is short. The type is adopted when circumstances do not justify something else better. However, when well designed it is a servicable structure, easy of maintenance and of moderate cost, the cost being often only
about two-thirds of that for a steel bridge.

This type has all the advantages of the panel system and of the stiffness of counters as set forth in Art. 1 and 3, but the fact that the diagonal members are in compression causes this type to be undesirable. This type is adopted for entire iron or steel bridges occasionally, but such practice is not considered good. The diagonal members are not the shortest possible web members, the posts being the shortest. The fact is that both the great number and length of columns in this type is a serious objection to the selection of the Howe truss for steel bridges, since this causes both the cost of fabrication and the waste of material required for columns to prevent buckling to be great.

Many mixed material (of wood and iron) bridges of the Howe type were built before 1850. From 1840 to 1870 it was built more than any other type. In the United States, the usual span was between 100 and 150 feet. One famous bridge over the Susquehanna at Havre de Grace, Md., was built on this plan. It had twelve spans of 250 feet each with draw span of shorter length. The longest Howe bridge was that over Saw-Mill Run near Pittsburg built by the Keystone Bridge Co., in 1861-2. The span was 270 feet. It was very heavy. It carried traffic for ten years and was replaced by two 118 1/2 foot spans in 1871.

Sometimes this kind of bridge is used for false work when the usual style of falsework can not be built so well. This has been done several times in the Pittsburg and Lake Erie Railway, particularly for erecting the Beaver bridge which has three iron spans of 150 feet.

The Pratt truss is one with parallel chords as shown in Fig.
59. As it is the case in all trusses with parallel chords, the upper chord and end posts are in compression. The lower chord is in tension. There are two vertical members between the first and second panels counting from the ends which are called hip verticals. These members are never in compression and suspend parts of the load in the first and second panels. The rest of the vertical web members which are called intermediate posts, are always in compression and never in tension. The main diagonals slope downward toward the middle of the truss. They are in tension under the action of dead load. To carry live load that would produce compression in the main diagonals, counters are inserted in the truss as shown dotted in Fig. 59. Such counters are tension members. They are only necessary when the maximum compression due to live load exceeds the tension due to dead load, but they are often inserted in every panel for safety under excess loading and for stiffness.

The tension members of the Pratt truss are usually built in the form of solid rods and eye-bars, such rods and eye-bars being the best form for tension. The truss is then a very suitable type for pin-connections.

Before the year 1900 many steel Pratt bridges were built with counters, for spans even less than 80 feet, but now the Pratt truss is not in fashion for such short trusses. For small spans in the present time, the Pratt truss is generally built without counter and is riveted. The diagonals near the middle are designed to take both tension and compression. This is due to the desirability of strength and stiffness to suit the modern heavy fast trains. The first Pratt truss without counters was built in 1870 on the Pennsylvania Railroad to suit the personal idea of the engineer. There was
then nothing done to omit the counters until recent years according to demand for stiffness, but now most railroads build the Pratt truss riveted and without counters, up to 120 or 160-foot spans.

When wood was a preferable material for bridge construction the Pratt truss was not in great favor for bridge work. The Howe truss was then a better type. As the Pratt truss was invented when wood was in extensive use it was a poor type for a time, because wood was not a good material for the many tension members of the truss and that iron and steel which were good for tension were too expensive. For a while when iron was used as much as wood there was then preference between the Pratt and Howe trusses only according to the material to be used, but with the production of cheap steel, the whole situation was changed. The Pratt truss became preferable over the Howe truss because it is a suitable type with steel.

About 1850 the Pratt truss began to be built wholly of iron. First cast iron was used for compression and wrought iron for tension. When wrought iron came into general use the type was constructed entirely of that material. It then gained much favor. Its favor was increased further as steel came into general use, until now it is built more than all other types of truss put together. It is the opinion of engineers that 90 per cent of modern truss bridges are of the Pratt type. Though the Pratt truss is nearly always built entirely of steel at present, it is occasionally built as combination bridges of wood and steel. In the West where wood was easily available the Pratt truss was built of wood as on the Green River Bridge of the Northern Pacific Railway. This was a span of 150 feet, replaced in 1903 by a more permanent steel structure.

The Atchison, Topeka and Santa Fe Railway build many through
Pratt trusses of spans from 100 to 172 feet, according to their standard plans. Their Pratt bridges were pin-connected but the connections of the flooring lateral and cross-bracings were riveted joints. For spans between 100 and 140 feet the bottom chords were riveted and stiff, this being a feature in vogue. These chords consist of angles and plates riveted together. With most of the connections rigid, these structures are sufficiently rigid for modern fast trains. The construction is simple, without curve plates for brackets. All spans have deep heavy portals and post brackets with plates having straight edges. There are always four lines of stringers each carrying equal load. This is better than the use of two supporting stringers with two lines of outside idle stringers. This construction reduces the blow from traffic for the load is not directly over the stringers. It enables the flooring to be shallower and it does not require the use of stringer laterals or cross frames.

As in the case of through plate-girder spans the Atchison, Topeka and Santa Fe Railway build through truss bridges with floor beams of economic depth which are called Class C. This railroad also builds through truss bridges with floor beams as shallow as possible which are called Class D. Bridges of the latter class are heavier, but their use is good in many cases to save head room in the stream or to reduce grades on the line.

For panels of 25 feet or less the weights of trusses are reasonable, but the long panels make the flooring heavy. Ordinary panel lengths for trusses are from 20 to 25 feet, but for long spans panels of 30 feet are sometimes adopted. The practical economic depth of a Pratt truss is about equal to the panel length. The
depth must not be greater than one-tenth of the length for good proportion. Thus, the span of the Pratt truss should not be more than about 300 feet. By inserting sub-trusses in the Pratt truss, the panel is shortened by one-half. This doubles the limiting span length.

The Baltimore truss is a type with secondary King-post trusses inserted in a Pratt truss. Fig. 60 represents a Baltimore truss diagramatically. This type was originated by the Bridge Department of the Pennsylvania Railroad in 1871. The Baltimore Bridge Company were the first to build bridges of this type. They advertised it extensively and the type became known by their name. Since then, the Baltimore truss became a prominent type.

As the insertion of the secondary King-post trusses shortens the panel of the original Pratt truss by one-half of the length, the cost of flooring per foot in the Baltimore truss of twice the length is the same as the cost of flooring for the original Pratt truss. For large spans the use of sub-panels is very economical and is almost always adopted, but the presence of the sub-panels adds many members to the truss. For a large span the saving in flooring due to the sub-panel is greater than the increase of cost due to the addition of work and material for the sub-panels, but for small spans, the use of Baltimore trusses is not justifiable on account of the increased number of members of the trusses, and their cost.

There are many modifications of the Baltimore truss with different arrangements of the secondary members. Equivalent members of the King-post truss may be used. Fig. 61 represents a Baltimore truss with tension members d'E, f'G, etc., instead of the compression members cd', ef', etc. in Fig. 60. Thus, all the sub-diagonals
are ties except those for the second panels from the ends. When for through truss with the sub-trusses near the top chord, the floor is suspended as shown in Fig. 62. In this case all the members of the sub-trusses are only subjected to tension except the two attached to the lower chord near the ends. This has the advantage that ties have over columns, but even these end ones are only subjected to tension when vertical end posts are employed as represented in Fig. 64. However, this only happened in a few cases for double deck bridges as the arrangement is not common. Fig. 63 shows the most common arrangement of a deck Baltimore truss. Sometimes, the inclined end post only extends over one panel and its slope is very great as appears in Fig. 65. Fig. 65 also shows a possible arrangement of the ends of a deck Baltimore truss with the dotted line as diagonal, but the lower chord members at the ends of the truss are not under stress, thus making the truss unstable.

There is a disagreement among authorities on bridge designing as to the preference of sub-ties or sub-struts as shown in Fig. 60 and 61. The sub-ties are lighter and easier to fabricate than the sub-struts. But with sub-struts, the load is carried more directly to the piers, besides the sub-struts have about twice the area given to the sub-ties and they reduce one-half of the deflection at the middle of the main diagonals.

Like the Pratt, the Baltimore truss is usually built with counters for larger spans and without counters for shorter ones when the truss is riveted. As the stresses in the counter-braced panels are ambiguous but are not very great, the members in the panel are often proportioned so that the load in the sub-panel point may be carried by the counters or the sub-struts. For rigidity, the verti-
cal members in the sub-trusses are sometimes made rigid and continued from the bottom to top chords. This occurs in a single track bridge of the Cleveland, Painesville and Eastern Railroad at Willoughby, Ohio, built in 1896.

Being merely a special form of the Pratt truss, the Baltimore truss has all its good qualities. The struts of the main truss being perpendicular to the chords are the shortest; the main diagonal members are in tension; the chords are parallel and are easy for construction; the web is in the single intersection system and so the stresses are easily determinate and well understood. Moreover, the sum of lengths of compression members is small compared with those of other types, and further, the secondary members may be arranged so as to be in tension, thus reducing the number of columns. The flooring is rendered cheap by intermediate supports of the sub-trusses. Thus, the Baltimore truss reduces both the number and length of compression members to a minimum, this being an important matter controlling bridge design; a more important factor than all is the fact that the sub-system enables the bridge to be much longer than the Pratt type.

The Baltimore truss was first built for spans that are regarded as short ones at present date. The longest spans then were Whipple bridges. Later, the Baltimore truss was used for long spans considerably. The longest simple spans of this type are the 542 1/2-foot span over the Ohio at Cincinnati built 1888 and the 546 1/2-foot span at Louisville, built 1893. There is also a 533-foot span over the Delaware River, built in 1896 by the Pennsylvania Railroad.

In recent years, the Baltimore truss also came into use for moderate spans with solid floor. This is because it is a convenient
means of supporting the trough floor. The New York Central and Hudson River Railway has constructed many Baltimore truss bridges with solid floors and many other railroads are following their example.

The Bowstring truss is one with the stresses in the web and chord members equalized as explained in Art. 1. The sizes of the members of this truss are made uniform, but this type of truss is not used for railroad bridges. For railway work, other types of curved top chord trusses are used. In most of these types, the panel points on the upper chord are not constructed in the line of a parabola but are placed between it and the line for parallel chords. Thus, the stresses in the web members as well as those in the chords are roughly equalized and the type is an economical one. This is because the top chord carries a part of the shear. But when the inclination of the top chord members becomes excessive, the web members become too light and vibratory, making the type undesirable. Such excessive inclination may also cause the stress in the web diagonals reverse. The reversions of stresses require the use of counters. And when counters are used in every panel there is no gain of economy by curving the top chord. The top chord members are then constructed at slight inclinations so as to save material by avoiding the use of counters near the ends of the span rather than to use more and lighter counters.

The Bowstring as invented by Whipple is not suitable for railroad service. This is mainly for two reasons, first, the portal bracing which naturally comes at the top of the end post would be too low; and secondly, the top chords with different inclinations are hard to manufacture. However, the truss can be inverted to obviate the first difficulty for railroad service. But this is not
done as there are other types more suitable for deck bridges. The truss is used for highway services occasionally. Lenticular trusses are trusses with both chords inclined. They are still more unsuitable for services than the Bowstring truss. Their advantages and disadvantages are the same as those for the Bowstring truss, with the addition of both chords being curved and hence still harder to manufacture.

Many modifications of the Bowstring truss are in use for railroad service. They are more economical than parallel chord trusses, especially for large spans, and they have come to extensive use since 1890. The curved top chord trusses can not be constructed with wood very well and their development awaited the extensive use of wrought iron.

The modifications of inclined top chord trusses are many because the bridge designers can not agree upon the best one. Bridges are built according to the idea of the designer. Some old trusses have the panel points of the top chord in a mathematical curve, others have every member of the top chord at a different inclination but do not have the panel points on a mathematical curve. Still others have two or more consecutive top chord members on one side of the center line at the same inclination. The last method is the most practical one as the construction is simpler and the top chord stronger. At the same time, economy of material is gained. In this plan many good size bridges are built. The Atchison, Topeka and Santa Fe Railroad has standard plans for truss bridges of this style from 210 to 300-foot spans. Fig. 66 and 67 serve to represent the outlines of spans of 214 feet, 6 inches and 260 feet for the Atchison, Topeka and Santa Fe Railway.
Fig. 66 A.T. & S.F. Ry. Standard 214-Foot Bridge.

Fig. 67 A.T. & S.F. Ry. Standard 360-Foot Bridge.

Fig. 68 Parker Truss.

Fig. 69 Pegram Truss.

Fig. 70 Pettit Truss.
About 1870, C. H. Parker first erected a number of curve top chord trusses of the type represented by Fig. 68, for railroad service in New England. This modification is intermediate in form between the Pratt and Bowstring trusses. It is called the Parker truss. It is a type of curved top chord truss with the panel points of the upper chord above the curve of a Bowstring truss as shown in Fig. 7. The main diagonals are in full lines and the counters are in dotted lines. The verticals of this type may some times be subjected to tension, unlike the corresponding members in the Pratt truss which are only in compression. This is one point against the use of the Parker truss. As the top chord members are at different inclinations and are in compression, the joints are liable to be weak and are hard to make, but the stresses in all the truss members are well determined. They are roughly equalized in the chords and web, and there is no objection of the length of columns. This is a good type of truss when the magnitude of the structure is sufficient to justify the use of curve top chord.

In 1887, Geo. H. Pegram first designed a truss which may be considered as the Parker truss modified by inclining the vertical web members toward the middle of the structure. This is a combination of the Parker and Posts trusses. Fig. 69 represents such a truss with the heavy lines for struts, light lines for ties, and dotted lines for counters. The inclination of the struts at the ends are greater than those at the middle, thus making the truss somewhat like the Warren truss at the ends and more like the Pratt truss at the middle. The lower chord has an odd number of equal panels. But the upper chord has the same number of shorter approximately equal panels. The upper panel points are on an arc of a
circle through the top of the end posts.

The longest trusses of the Pegram type are those for the thirteen spans, each of 200 feet, across the Arkansas River at Fort Smith, completed in 1890 in the line of the Missouri Pacific Railway. As this type has no advantage over the Parker truss and has the additional disadvantage in complication of outline, due to the increased number of inclined web members, it has not been adopted since 1897. However, Pegram trusses are used on the Union Pacific and several other western railroads, but they are only used to a limited extent. This type will probably never become popular as there are many other better types. It may well be classified as an obsolete type although it has not been invented very long.

The Parker truss has a curved top chord to economize material in the truss members by approximately equalizing the stresses. The Baltimore truss has the sub-panels which offer immediate support to shorten the panel for economizing material in the flooring. The combination of the two types is one commonly known as the Pettit truss. This type was invented by the Bridge Department of the Pennsylvania Railroad, but the word Pettit has been used to designate the type, due to an error in the report of some foreign commissioners in the Centennial Exposition and was afterward introduced into engineering literature.

The Pettit truss is one with curve top chord and sub-panels. It is very economical for long spans as economy of material justifies the extra labor for making the chord members inclined. Sometimes, every top chord member has a different inclination on the same side of the truss, but generally the members of the top chord have the same inclination one by one as shown in Fig. 70. With the members so arranged in straight lines, the joints between these mem-
bers are stronger and more reliable. Money is saved as these joints are easier to make. At the same time, the top chord members are inclined so as to approximately equalize the stresses in them and in the web members of the main truss.

The sub-panel system is very effective in saving money in the flooring for long span bridges. The inclined top chord does not save money for short spans, but it does so effectively for long ones. Thus, the Pettit truss is the most economical for long spans, with the advantages of curved top chord and sub-panels combined. Economy of material is even more important than equalizing stresses, and so the Pettit truss is used for long spans. As for long spans, the dead load is greater than the live load, reduction of dead load brings about greater reduction of stresses in the chord and main diagonals, and a consequent saving in material.

Fig. 70 represents the Pettit truss as often constructed. The heavy lines represent compression members; the light lines, tension members; and the dotted lines represent auxiliary members. The auxiliary members are rigid members connected to the long columns near their middle. (1) The sole function of the auxiliary members is to hold the middle of the columns in place thus reducing the theoretical length of the columns. But this function is not well performed. When these auxiliary members are loosely connected to the columns they are not doing any good but add weight to the bridge. When these connections are rigid, they put the middle of the columns out of line for change of length and deflection and produce secondary bending stresses. Thus, there is a tendency to build the Pettit truss without the auxiliary members.

(2) There is greater variation in size of the members in the Pettit truss than in other types, due to the main and sub-trusses.
This is also a disadvantage. With sudden changes of temperature, the thinner material changes length faster than the thicker. This causes temperature stresses which amount to 190 pounds per square inch for each degree of difference of temperature in ordinary cases, when the length of the member is fixed. But as the length of the members is not held fixed the difference of temperature causes redistribution of stresses which becomes serious as such difference is often as great as that caused by 10 degrees Fahrenheit.

(3) There is also a disadvantage of the Pettit truss on account of the weakness of the top chord. The auxiliary members generally added near the middle of the chord members cannot be greatly depended upon for bracing on account of deflection and change of length. The top chord members are then only braced laterally at the main panel points by the top lateral bracing and vertically by the vertical web members. When the top laterals are rods, they slacken when a load on the bridge puts the top chord in compression, but when the top laterals are rigid members, they are themselves so compressed that they need bracing. Unlike parallel chords, the top chord in the Pettit truss are not solid members and is in a state of unstable equilibrium until there is a lateral motion enough to put the diagonals of the top lateral in tension.

(4) As the panels are long, the truss is also susceptible to damage in the web members. Unlike short panel riveted bridges, this truss collapses more readily. (5) The joints for members at different angles of inclination are also harder to make. Moreover, they are much weaker than parallel chord joints. These are the disadvantages of the Pettit truss. However, the advantages are much greater than the disadvantages, and no other type serves the same
purpose so well.

The Pettit truss is the best type for long simple railroad bridges. It has the simplicity of stress of the Pratt truss. It has the advantage of sub-panel of the Baltimore and the approximately equality of stresses which characterizes the Bowstring truss. These qualities of this type were not very well recognized at the beginning, but as soon as they were fully appreciated, the Whipple truss which has been used for long spans, ceased to be built and the Warren type was confined to short spans.
CHAPTER III. ECONOMICS.

Art. 6. Location.

Bridges must be so built that they do not obstruct the waterway as may be required by the government. The approaches must be as nearly on grade as possible to save money for earthwork. Sometimes, however, the approaches are raised slightly above grade in order to gain clearance in the waterway. Often, the structure is modified for the same purpose, and the additional cost of the structure due to deviation from economic dimension result in a greater saving on earth-work. For elevated crossings in city streets such modification is specially effective.

(1) When there is plenty head room in the line of traffic under the bridge the road should be carried at the top of the girder, and the structure is a deck bridge. This is because a deck bridge is the lightest. The distance between the plate-girders or trusses is less than for through bridges. The pier is not required to be built up so high and there is also saving in the falsework which is also lower when material is delivered below the bridge. However, it is desirable to carry the masonry pier to the top of deck bridges for stability but this is expensive and is not done. Besides, for deck plate girder bridges, the stringers and floor beams are usually omitted, and the ties rest directly on the girders, but the Boston and Maine Railroad retains the floor beams and stringers even for deck bridges.

(2) When head room is somewhat scanty, the depth of the girder may be made less than the economic depth, thus allowing the bridge to retain the advantages of the deck bridge.

(3) When more head room is required the roadway should be at
the lower part of the structure and this is called a through bridge. In this case, a through bridge of economic depth is the best.

(4) But if still more head room is required, a through girder with flooring shallower than the economic depth may be the best. In this case, the floor beams are made as shallow as possible. As solid bridge floors are effective means for reducing the depth of flooring, they are sometimes used for this purpose, as described in Art. 5.

Plate-girders of Classes A, B, C and D of the Atchison, Topeka and Santa Fe Railway standard plans are structures designed to suit the four conditions given above. The use of these four classes of bridges has been found very economical. Thus, the requirement of head room and the relation of grade and approaches are controlling factors concerning the use of deck and through bridges, and they effect the economic depth of the girder and flooring. These are only general conditions, and for different bridges, the local conditions should also be considered.

Shallow deck structures and shallow flooring are better adapted for short spans but not long ones. For long spans, the little gain of head room by such deviation from economic dimensions is comparatively small. For truss bridges the choice is more between deck and through, but shallow floor beams are sometimes used for through truss bridges of ordinary simple spans. The Atchison, Topeka and Santa Fe Railway standard plans provide for truss bridges with shallow floor beams for spans from 100 to 160 feet. This is done for stiffness as well as economy.

In some cases, neither the deck nor through bridge satisfies the condition for best economy with truss bridges. Taking the question of clear head room and the grade of the approaches in consideration, the road would come between the top and bottom chords of
the trusses. When the trusses carry the road between the chords they are called "pony trusses"; and the bridge, a pony truss bridge. The use of pony truss is bad because the top chord can not have lateral bracing. The top chord is under liability of buckling laterally as a very long column extending from end to end of the bridge in the case of a riveted bridge. Pony bridges were used pretty much formerly but they are very seldom employed now.

Some engineers favor the use of pony bridges when deck bridges do not serve the purpose as well, especially when the floor beams are deep and well riveted to the trusses for spans from 100 to 125 feet. Pony bridges are also considered by some engineers as being suitable for places where the crossings of several streams occur on a stretch of level grade with a small elevation of grade above high water which does not afford necessary room for deck girders. "Raising the grade for its entire length might be too costly; on the other hand, short approaches to each bridge, introducing several bumps on the level stretch would be very objectionable, especially on a road where fast trains are to run," (Mr. G. Bouscareen). Such practise does not meet the approval of the best authorities. The pony truss is also considered by some as being suitable when the floor beams can be made so deep that they extend near the top chord and are only two or three feet from it so as to give rigid support. This is an allowable use of pony trusses. Thus, the use of pony bridges is limited to special cases.

The use of pony trusses is bad, especially with shallow floor beams riveted to the trusses with brackets for bracing the top chord, as usually done, because the deflection of the floor beams forces the top chord out of line. In half through plate-girder
bridges, there is a tendency to use long panels with deep floor beams. This is a bad practice as the lack of head room which prevents the use of deck girders also restricts the depth of floor. As the plate-girder is best adapted to receive smaller loads at many points, the floor beams of the bridge should be close together. Generally, they should not be over 15 feet apart and they should have brackets riveted so as to brace the top flange of the girder.

By the nature of construction, curve top chord trusses are not suitable for deck bridges. There is no saving of material by inclining the top chord as the track has to be supported at a level. The lower chord may be inclined or curved while the top chord is kept straight for a deck bridge. In this case, the support at the ends is awkward and the masonry has to be carried up high the same as for a through bridge. Again, this offsets the advantages in point of economy, and curved chord trusses are not used at all for deck bridges. Curved chord trusses are not suitable for pony truss bridges with one more reason that the top chord which is in compression needs more bracing than can be secured. Consequently, parallel chord trusses are the only ones which are good for deck as well as for through bridges. Plate-girders are also suitable for both deck and through bridges, and they are specially good for through bridges when the ties are permitted to rest directly on the girder flanges as for short spans.

As short span bridges are more economical than longer ones, the girders which support elevated railroads in large cities are short. Plate-girders are the best form of construction, but the solid web plate-girders would shut out the light from the stores near the railroad. For this reason, girders of span from 40 to 65 feet are required to be built with open web, in most locations except in
the middle of wide streets.

Art. 7. Erection.

The erection of a bridge requires a good deal of thought and consideration. The cost of erection is to be as low as possible. Sometimes, the work should be done quickly to avoid risk to the structure and obstruction of traffic on the railroad or on the stream. Sometimes, the conditions of erection even forbid the use of simple trusses. Many different methods of erection have been tried for different types of structure and in special cases the work requires ingenuity of the erecting engineer, but certain methods have become common for certain types of bridge, and they have become more or less standardized in recent years.

As the short T-rail culverts are very simple to construct, they are also very simple to be erected. The members are simply carried to the site and fastened together as may be required. There is nothing to obstruct traffic seriously in case of renewal, nor is there anything that will menace the safety of the structure.

I-beam bridges are generally sent from the shop in one piece and lifted by a derrick car or by a derrick set on the ground, to place the structure on its exact place. Short span plate-girders are erected in the same way and sometimes the structure needs to be slid or rolled to the site when the train does not carry it close enough. All this is very simply and easily done.

When the plate-girder span exceeds 25 feet, falsework may be required to carry one end of the girder over the stream. With a beam temporarily thrown across, the girders may be rolled or slid into place. The lifting may be done by derrick cars, gin-poles, or
wooden "A" frames, or scaffolds at the abutments as the case may require. As the span becomes longer, the weight becomes greater; transportation and erection becomes harder, and there is more liability to damage of the structure, but the structure should be built in one piece in the shops as far as possible and not be spliced in the field. The span is usually of three car lengths, the weight during shipment is carried by the two outer cars, the middle one being an idler. Sometimes, however, the span is of five car lengths with three idlers, one being between the two cars that carry the weight, as for the 123-foot span in Philadelphia. For example, the Northern Pacific Railway have their deck plate girders completely riveted in the shop, up to 60-foot span; and above that length and for all through spans they ship the plate-girders separately on account of difficulty in handling. In the early days plate-girders were not made as long as they are now on account of lacking transportation facilities, but now that trouble has disappeared and the cost of other things has become more important in determining the maximum span for plate-girder bridges. The Atchison, Topeka and Santa Fe Railway build plate-girder bridges up to 106 feet in span. All girders, and some deck spans are shipped whole and completely shop riveted. The matter of completely riveting deck spans is desirable in most cases.

In cases of renewal, girders are often run out on falsework and then lowered along side the old structure; and when the old structure is taken down, they are slid or rolled into place. In the majority of cases, the plate-girders are first set along side of the old structure and rolled or slid into place between the time for passage of trains. The erection of plate girders as that of other
types may be done in many ways. In general, it is easier to erect plate-girder bridges than simple truss bridges of the same length.

The erection of girder and short trusses of viaducts costs less than for other bridges. Gin-poles and derricks may be used, but it is more often done by a traveller resting on the finished portion of the structure and overhanging a sufficient distance to erect the whole span and the tower ahead of the traveller. After the tower is finished, the girders are placed between the two towers and the traveller is moved one span ahead. The same is done to each span, and very little expense is involved.

Simple trusses are almost always erected on falsework with travellers, even for short spans. This is one point in favor of plate-girders as falsework costs money. Trestle bents are erected at the site, across the stream with a track to carry the traveller back and forth along the line of the bridge. The members are hoisted by men or steam. In case the top of the piers and abutments are high above the bottom of the stream the falsework becomes very expensive.

Safety during erection is very important. When the safety of the falsework is menaced by current, ice-jam, etc. the use of simple structures may even become undesirable and other forms of bridges are used. However, simple bridges are sometimes erected without falsework as cantilever bridges. This has been done on the 416-foot curved top chord bridge over the Columbia River. The simple bridge was made into a cantilever bridge during erection, but after it was erected, it was turned into a simple bridge by cutting the top chords at the ends. This saved the expenses of falsework and avoided liability to trouble from the swift current, but the
truss work was made with stiff bottom chords just as for a cantilever bridge. In this case, an additional amount of material was required so that there was no saving. The bridge was not made a cantilever bridge on account of a curve at one end.

To avoid the swift current and the obstruction of traffic bridges are sometimes erected on trestle bents in shallow water. The trestle bents and trusses are carried to the site by barges. Many large bridges have been erected this way; among them is the 523-foot Channel span of the Ohio Connecting Railway Bridge near Pittsburg, the bottom chord being 150 feet above water.

For renewal, truss bridges may also be erected on the old structure as falsework and the old structure taken down after the new ones are erected or swung. In this case, erection may cost very little, even less than for plate-girders. For renewing ordinary truss bridges, the trusses are, in a majority of cases, erected on falsework by the side of the old structure, and then shifted to the intended points after the structure can be self-supporting.

Attempts have also been made to save money by standardizing the falsework and using it over and over again. This is a good scheme to save money when the bridges to be erected are not far from each other. But when the bridges are far apart, the transportation charges exceed the cost of the falsework and the method is not economical.

As shown above, difficulty of erection increases as the length of the span. But this cost is also influenced by the type. Plate-girders are usually erected at less expense than trusses even for same span lengths. This is because there is very little falsework, if any, required for plate-girders, and the time required is
shorter as most of the construction work is done in the shop. For renewal work, plate-girder bridges are erected in one or two hours for each ordinary span complete. This would delay very few trains, and is an important matter. This time is very much shorter than for erecting pin spans.

According to the report of a committee of the Association of Railway Superintendents of Bridges and Buildings in 1905, the cost of erecting girders is from $4.00 to $10.00 per ton and that for truss work is from $10.00 to $20.00 not including transportation of material, tools and men.

In 1907, the Atchison, Topeka and Santa Fe Railway have erected many thousand tons of steel bridges. The average cost of erection is as follows:

<table>
<thead>
<tr>
<th></th>
<th>$4.63 per ton of which 934 tons are included,</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trusses</td>
<td></td>
</tr>
<tr>
<td>Plate-girders</td>
<td>$5.49 &quot; &quot; &quot; 2784 &quot; &quot; &quot;</td>
</tr>
<tr>
<td>I-beams</td>
<td>$2.88 &quot; &quot; &quot; 2337 &quot; &quot; &quot;</td>
</tr>
</tbody>
</table>

All the structures concerned were in the main lines and all trains ran as regularly as could be expected. The costs are very low, due to the effect of standardization. They seem to indicate that cost of erecting plate-girders is greater than for truss bridges which is contrary to fact. This is due to the removal of old plate-girders for renewal, which constituted a great part of the work.

Art. 8. Styles of Joints.

The style of joint is a very important thing concerning the general construction of a bridge. It has a great influence on the expenses and time required for erection, as well as on the weight and other qualities. The two different kinds of joints are pin-
joints and rivet-joints; and the bridges with these joints are pin-connected and riveted bridges.

The pin-connected bridge is a typical American structure because it is mostly used by American engineers. The members of such a structure are connected together by short cylindrical bars or pins passing through holes and eyes, thus transferring the longitudinal stresses from one member to another by means of shearing and bending stresses in the pin. The compression members are usually built up with plates and angles or channels. And the tension members are eye-bars or rods.

The riveted bridge, on the other hand, is a European type. It is usually adopted by European engineers. Both chords consist of shapes riveted together. When they are too long to be in single pieces they are made in shorter pieces and spliced together in the field so as to make them practically continuous. The web members are riveted to both chords directly or to plates which are in turn riveted to the chords. The first riveted bridge in this country was built in 1859. This was an iron lattice girder built by George E. Gray on the line of the New York Central Railway. It was taken down about 1901.

The first entirely pin-connected bridge in the United States was built by J. W. Murphy, also in 1859, on the Lehigh Valley Railroad at Phillipsburg, N. J. This was a Whipple truss bridge with loop eyes for the tension members. The use of eye-bars is due to J. H. Linville who first built a bridge with them on the Pennsylvania Railroad in 1861.

In the early days pin-joints were used for most truss bridges in the United States, even for spans of 20 and 30 feet. Now
their use is limited to longer spans, and they are not in favor of most engineers for spans less than 200 feet. European engineers have always opposed the use of pin-joints. However, the pin-joint was originated in England and was first used in 1851 for the New Ark Dyke bridge on the Great Northern Railway. This was a Warren truss bridge with two spans of 250 feet each, and had the depth of 17 feet. It had wrought iron tension members and cast iron compression members. It was very heavy. The second pin-connected bridge in England was the Crumlin bridge built in 1853, and the third was a railway bridge over the Thames at Charring Cross in London. These are probably all the pin-connected bridges of any importance that have ever been built in England. They did not fail, but were removed when the load to be carried became too great. They were very flexible and furnished English engineers reason for prejudice against pin-joints. Engineers in the Continent, such as Pauli of Germany, have also tried the use of pin-joints, but they are never in favor of this type of connection. Probably, this is because such special workmanship required for pin-joints was not employed in Europe.

On the other hand rivet joints were formerly not in favor among American engineers. But now the good features of both types of construction are combined. The modern pin-connected bridge has many rivets connecting plates to furnish bearing areas between pins and the truss members. There are many rivets in a pin-connected bridge and the two types of construction may be said to have been united. Besides, riveted bridges are used for short spans while pin-connected bridges are only used for long ones, according to best practice in this country.

There are many points in favor of pin-connected trusses as
well as for riveted trusses. The two styles are then suited to different conditions; the riveted truss for short spans, and the pin-connected truss for spans of 200 feet or more, as adopted by American engineers. (1) The strongest point in favor of the pin-connected truss is that it weighs less for the same specified loading when the span is long. There is considerable saving in the tension members which are eye-bars. Thus, the pin bridge is cheaper. Riveted trusses generally require a little more material, but spans of 200 feet or less, may sometimes be built almost as cheap with riveted joints. A good bridge may be built on either plan, but the total cost usually determines the type of construction. (2) Concentration of material in solid bars and rods is also a point in favor of the pin bridge. Material is spread out in the form of plates and rolled shapes even for tension members for riveted trusses. This is a source of weakness on account of localized stress and corrosion, but it adds stiffness to the members concerned.

(3) Rigidity is the most important point in favor of the riveted truss. This makes it suitable for fast trains and short spans, but for longer spans this advantage vanishes as the live load is much smaller than the dead load, and the vibration due to the traffic is less. The chords in a riveted truss are practically continuous and the rivet joints hold the members of the truss more solidly; and this, of course, makes the truss more like a solid beam. On the other hand, pin-joints make the members of the bridge loosely connected and are regarded by some engineers as flimsy and of poor mechanical construction. (4) But unfortunately, stiff connections produce secondary stresses by bending as the truss deflects, when loaded. It should also be noted that temperature changes on large bridges cause many secondary stresses. With pin-connections such
stresses are much reduced. However, there are secondary stresses due to eccentric loading on the eye-bar or tie-rod.

When a truss deflects the different parts move. Any part may be considered fixed and all the other parts considered to move relative to it. Suppose a pin-joint is represented by Fig. 71 has the pin fixed. Suppose a point $A$ of the eye-bar moves to $A'$. Let $B$ be the point of contact between pin and eye before moving. It is in the axis of the eye-bar. Then $B'$ which is a point not in the axis of the eye-bar after moving if the diameter of the pin is much smaller than that of the eye. This is because the eye-bar does not slip on account of friction. This change of point of contact may cause the load on the eye-bar to be eccentric, thus, producing unit stresses in the bar greater than for concentric loading which excess may be called secondary stress. As the pin and eye are practically of the same diameter the point of contact is in the axis of the eye-bar, and the secondary stress is zero. On the other hand secondary stresses in the riveted truss would be large, as little bending in a member causes serious stresses. The deformations that produce bending are considerable as can be determined by displacement diagrams.

English engineers are inclined to think that riveted bridges have less secondary stresses. An opinion has been expressed that riveted stresses have less secondary stresses for its stiffness. This is not true. The deformation is resisted by bending the members and not because there is less tendency for deformation when the truss is riveted. It should also be remarked, that for sub-panels, pin-joints are very effective in reducing secondary stresses in the
main diagonals due to loads causing deflection by the sub-struts and sub-ties. The matter of secondary stresses is an important one as such stresses sometimes amount to as much as 50% of the principal stresses. Thus, the stresses in pin-connected trusses are more determinate than those in riveted trusses. Deflections and camber in pin-connected bridges allow of more accurate determination than those in riveted trusses.

(5) Ease of erection is an important point in favor of pin-connected trusses. Considerable thought and attention is required of the erecting engineers so that the last members to be placed in the bridge can be inserted easily. Adjustments may be required on the level of the joints from time to time during erection. For pin-connected bridges such adjustments need not be so exact as for riveted bridges. The large number of field rivets for riveted trusses require much more time and labor in the field. This means greater expenses for erection. The longer time required for erection of the riveted bridge is also an objection for renewal as it prevents the passage of trains, but the time required for erecting a pin-connected truss is very short.

The case of the Cairo bridge is a good example to illustrate speed of erection of pin-connected bridges. This was a Whipple truss bridge built across the Ohio River in Southern Illinois in 1889. The bridge had two spans of 518 feet 6 inches each weighing 2,000,000 pounds. Erection was completed by seventy-five men working for one month and three days, including five idle Sundays. The time for removing the falsework from one span to another was also included. The first span was erected in six days only, with the falsework ready before erection.
With modern equipment and the skill developed in the erectors, many pin-connected truss bridges of 200 feet or less have been erected within one day after falsework is completed. This would reduce liability of damage by swift current, floods, ice jams, storms, etc. As the liability of damage due to the conditions of the stream and storm is great, the bridge that can be quickly made self-supporting is preferable. For pin-connected bridges the field rivets are not many. Bolts are used to fasten the bridge temporarily for safety. With sufficient bolts substituting the rivets, trains may be allowed to run over the bridge in case of renewal work. This is done only when the service of the bridge is unusually urgent. Advocates of riveted trusses sometimes argue that the same method of using bolts for rivets for temporary service may be adopted for riveted bridges. This would subject the structure to too much abuse for unequal stresses in different parts of the joints and is not allowable.

As to the strength of joints one system is about as good as the other. All joints are liable to be weak and require more work and attention than other parts of the bridge. Riveted bridges may last longer on account of the different members being united more closely, but for spans of 200 feet or more the plates become thick and the joints are clumsy and weak. The stress in any rivet is also not very certain. This is shown by loose rivets generally being present, and the cause of the looseness of such rivets must be localized stresses. Fortunately such rivets are easily replaced. English engineers believe that riveted joints do not get out of order as readily as pin-joints. They once had the opinion that pin-joints would work loose in the course of time for wear and tear. This opinion has been proved incorrect.
As naturally expected, there are occasionally accidents on bridges due to derailment, breaking of some parts of the train or projection of something in the train. These accidents often injure or even detach some members of the bridge. Experience shows that riveted bridges do not collapse so readily as pin-connected bridges. Such injuries have occurred many times and the injured riveted bridges carried the load. The same injuries on pin-connected bridges would do more damage to lives and property. This is a fact, inducing favor for the use of riveted bridges, but in this connection, it is well to note that many accidents may be avoided by increasing the distance between trusses by a foot or two, thus decreasing the liability of accident.

The relative prices of material and labor also have a bearing on the adoption of riveted and pin-connected bridges. For pin-connected bridges the eye-bars are very carefully forged and the dimensions of the bars are very accurate, but great part of the work is very cheaply done by machines in America. Thus, the use of pin-connected bridges is suitable for conditions in this country. In Europe the use of machines in bridge works is not so extensive, making labor more expensive. On the whole, the amount of labor required for a riveted bridge is less, and riveted bridge suits European conditions better.

The erection of riveted bridges is also unsuitable for localities where skillful labor cannot be secured as there should be more people working for a longer time in the field than for pin-connected bridges. Due to scarcity of labor in the field, England sent many pin-connected Warren truss bridges to India for railroads in the early days beginning about 1858. This was the only thing that
could be done, in spite of the prejudice of English engineers against pin-connected bridges. Unfortunately, pin-joints are not suitable for the Warren truss because the web members have to resist alternate stresses. The pins work back and forth in the eyes and holes. England for the same reason has also sent many pin-connected bridges for railroad use to other dependencies and colonies. Formerly the Warren truss was also frequently built with pin-connections in the United States, but now the Warren truss is invariably built with rivet joints.

As modern traffic is very heavy and fast, the bridges accordingly must be massive and stiff. Stiffness is especially important for short span bridges as the structure is light and hence sensitive to vibration. As the riveted truss is more rigid than the pin-connected truss, short spans are preferable riveted, but for long spans, the live load constitutes the smaller portion of the total load and the vibration is less. Thus, with all the advantage of pin-connection, the long span simple bridge should be pin-connected, while for short spans the trusses should be riveted for stiffness.

According to the best practice, the Warren truss is invariably riveted. This is also because the truss is better suited for short spans which should be riveted. The truss is either with or without sub-verticles. Double intersection Warren trusses are built occasionally. They are also built with sub-verticles to furnish immediate support for shallow floors, but the use of double intersection webbing is not in favor with the best authorities. It is not in the line of progress.

The Pratt truss is used for longer span than the Warren. When built for shorter spans, it is without counters. The main di-
agonals near the middle are made to resist both tension and compression. This is convenient with rivet connections and many such bridges are built.

The Baltimore truss was first constructed as riveted trusses in 1899 by the New York Central and Hudson River Railway with a trough floor which was necessary for the saving of head room in the waterway. Due to the immediate support of the sub-trusses, this type was found very suitable, and many bridges of this type are built for spans from 100 to 200 feet, by the company, aggregating 70,000 tons up to 1902. Many railroads followed the example of riveting the Baltimore truss, until now this is a common practice, but up to 1899 the Baltimore truss was only pin-connected and used for the longer spans.

The curved top chord trusses are seldom riveted because their use is limited to long spans. Hence, the Pettit truss is never riveted.

The character of the counters and lateral systems is also an important thing concerning the general construction of bridges. The immediate care of iron bridges is generally left to people who do not know much about bridges. It is the habit of such people to tighten up everything indiscriminately. It is therefore wise to construct bridges so that such adjustments cannot be made very easily. When counters are over-tightened, they relieve the main rods and chords of a part of the dead load. They will be improperly strained when the train passes the bridge. When such over-tightening is considerable, the main rods and chords are much relieved of the dead load. The counters receive shocks as the train passes and they will finally be broken.
The lateral bracings have been made adjustable in earlier bridges, but in late years they are generally riveted in iron railroad bridges, especially the bottom lateral bracings. This gives the bridges more lateral rigidity which is very good, as trains are generally running fast when they pass the bridges. This also prevents undue tightening. In one occasion, the intermediate sway rods of a through span at a little distance below the top struts had been screwed by the erectors so tight that they had forced the top ends of the intermediate posts out of line. The inspecting engineer was alarmed and surprised that the bridge did not collapse. This was a case of a combination bridge, but the danger in the case of an iron bridge is the same.

For attempts to reduce the loads on bridges into concentrated loads at panel points, and to eliminate uncertain stresses such as secondary stresses, the floors of old bridges used to be suspended by such devices as U-shape hangers. The lateral bracings were also connected to the trusses by loose joints. All the joints were then loose and the structures were flexible, but for heavy and swift traffic, stiff floor connections were necessary. They are better as they do not allow vibration to accumulate as much as do the floor hangers. Moreover, all the lateral and detail connections are made as stiff as possible even for pin-connected bridges.

For modern engine loads and ordinary panel length, the depth of floor beams and stringers become great, the former being generally three or four feet, and the latter two or three feet. When there is plenty head room these depths make no difference, but for many cases head room is important as for example in the case of through bridges in crossing streams and in elevated tracks in cit-
ies. In the latter cases, especially, every inch in decrease of thickness of flooring enables the road to be lower from the natural surface of the ground and such decrease is economy.

The floor is sometimes made thinner, by reducing the depth of the floor beams as described in Art. 5. Sometimes, plates and angles are used to form troughs for the whole flooring. Concrete may also fill the troughs thus formed and the railroad rest directly on the floor for or ballast.

For over head bridges in cities the floor may also be required to be water-proof. Many methods of constructing the solid floor and waterproofing it have been proposed and used. A great deal has been written on the subject of flooring. But only the treatment of the general design of supporting structures is contemplated in this little volume. However, it is well to note that a trough floor is sometimes used for short spans in place of both flooring and supporting structure. The plate-girder is the most suitable form of structure for the use of a thin or a solid floor. For trough floor the longitudinal axis of the trough may be placed perpendicular to the web of the plate-girders, thus enabling the troughs to be very short as desired. For thin open floor with through plate-girders, the floor beams may be placed at any desired distance thus enabling the floor beams to be shallow. When the span is too long for using plate-girders, trusses must be used. The panel needs to be short, but is controlled by the span length and economic depth of the truss. In this case multiple webbing is intolerable, the sub-panel system becomes best. Thus, the Baltimore truss became generally adopted for solid floors as mentioned in the foregoing pages.
Both theory and practice dictate the use of different types of bridges for different span length. For very small spans, rolled shapes are used for superstructure for simplicity of construction, but as the span is larger, economy of material is more important than simple construction. In long span bridges saving of material is attained by all possible means. It is shown in Art. 5 that rails are suitable for superstructures for spans up to 8 feet; I-beams for spans between 8 and 26 feet; and plate girders, for spans between 26 and 105 feet.

The general opinion of engineers in this matter is best shown in the specifications now in use. The general specifications of consulting engineers and bridge companies recommend and the railroad companies require certain styles for certain span lengths. These opinions are the results of experience. It is too complicated to compute for true economy by theory. The use of plate girders is required or recommended from 20 to 100 feet by more than half of the leading specifications. The minimum spans for the use of plate girders is from 18 to 30 feet; and the maximum from 80 to 125 feet. The minimum span, as found by average of twenty leading specifications, is 22 feet; and the maximum span, 101 feet.

In the early days railroad engineers wanted to use plate-girders for spans longer than those desired by bridge manufacturers. Even as late as 1895, railroad engineers wanted to use plate girders for spans up to 100 feet while the manufacturers preferred the maximum span to be about 75 or 80 feet. This was because adequate transportation and erection facilities were more or
less lacking, but as such facilities are now easily available, this difference of opinion disappears. Now, plate girders up to spans of 140 or 150 feet are occasionally built as conditions warrant with the general maximum limit at 100 feet. The use of plate-girders for long spans is not desirable as the structure becomes very heavy. The flanges become very thick and large rivets must be used. Besides, very large rivets do not make solid connections and the plate-girders become weak.

For spans longer than 100 feet trusses are generally used for carrying the railroad. In recent practice, the use of different trusses of the best types according to the order of increasing span length should be (1) Warren truss, (2) Pratt truss, (3) Baltimore truss, (4) Parker truss, and (5) Pettit truss. The Warren type may be omitted from the list if pin-connections are required, but when trusses are to be riveted for span lengths such as those with the use of Pratt trusses, the Warren type is just as good as the Pratt. However, the Pratt is the prevailing type for spans from 100 to 200-foot spans. For spans of about 175 feet or more, the use of inclined top chord trusses is economical as the saving of material by inclining the top chord would be great enough to justify more difficult construction; and for spans exceeding 175 or 200 feet the chords of through bridges are seldom made parallel. Such economy is quite appreciable as illustrated by the ordinates showing the weights of curved and parallel top chord trusses in Fig. 72. The actual economy and use of different types of trusses for different span lengths are thus roughly shown by these figures. The use of the different types of superstructure for different span lengths may be shown by Table II.
Table II. Specification of Type for Different Span Lengths.

<table>
<thead>
<tr>
<th>TYPE OF STRUCTURE</th>
<th>Authority</th>
<th>Proposed by Author</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Northern Pacific Railway</td>
<td>Atchison Topeka &amp; Santa Fe Railway</td>
</tr>
<tr>
<td></td>
<td>Span in Feet</td>
<td></td>
</tr>
<tr>
<td>T-Rail</td>
<td>Upto 8</td>
<td>Upto 8</td>
</tr>
<tr>
<td>I-Beam</td>
<td>10 to 30</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Up to 20</td>
<td>8</td>
</tr>
<tr>
<td>Plate Girders</td>
<td>25 100</td>
<td>26 100</td>
</tr>
<tr>
<td>Deck</td>
<td>26 100</td>
<td>20 100</td>
</tr>
<tr>
<td>Through</td>
<td>40 100</td>
<td>30 105(\frac{1}{2})</td>
</tr>
<tr>
<td>Pratt (Four panel Special)</td>
<td>90 125</td>
<td></td>
</tr>
<tr>
<td>Pratt Truss</td>
<td>130 200</td>
<td>110 210</td>
</tr>
<tr>
<td></td>
<td>100 200</td>
<td></td>
</tr>
<tr>
<td>Warren Truss</td>
<td>110 120</td>
<td>125 250</td>
</tr>
<tr>
<td>With Auxiliary members</td>
<td></td>
<td>125 200</td>
</tr>
<tr>
<td>Single Cancellation</td>
<td></td>
<td>200 250</td>
</tr>
<tr>
<td>Double</td>
<td>200 250</td>
<td></td>
</tr>
<tr>
<td>Curved Top Chord</td>
<td>210 300</td>
<td>175 300</td>
</tr>
<tr>
<td>Baltimore Truss</td>
<td>200 300</td>
<td></td>
</tr>
<tr>
<td>Pettit Truss</td>
<td>250 &amp; up</td>
<td>250 550</td>
</tr>
</tbody>
</table>
Waddell prefers the use of the Pettit truss for spans of 250 feet or more, but some authorities recommend its use for spans a little less than this. Most of the new simple truss bridges of spans greater than 300 feet are of the Pettit type. There are, however, Baltimore truss bridges of spans longer than 300 feet as in the case of the bridge over the Missouri River at Bellefontaine, Mo. This was erected in 1893 with a 440-foot span length.

Formerly, the Whipple truss was very commonly used for long span bridges. But as multiple webbing is not in favor of modern engineers, this type is not used. In the early days many Whipple bridges were built for spans from 400 to 515 feet. The longest bridge ever built of this type was the Cairo Bridge over the Mississippi River with trusses measuring 518 feet 1 3/4 inches from center to center of end pins. This was erected in 1889.

The longest Pratt truss is 255 feet. This is a double track bridge erected in 1906 over the Susquehanna River on the Pennsylvania Railroad. It is at Havre de Grace, Maryland. This is an unusual case where trusses with sub-panels and curve top chords should be used. As carried out in practice, very long bridges should be of the Pettit type. In present days this type is adopted, more than all the other put together, for simple spans of 400 feet or more. The longest bridge of this type is between Louisville and Jeffersonville, Kentucky over the Ohio River, completed in 1894. It is a single track bridge with spans of 546 feet 6 inches from center to center of end pins. Fig. 70 shows the general outline of this truss. This is the longest simple truss span.

Formerly the Warren type was also used for long span
bridges. The panel was shortened by adopting the multiple and sub-vertical systems. But as soon as the advantages of the Pettit truss was fully recognized, they were not used for long spans. The longest Warren trusses measures 521 feet 11 3/4 inches from center to center of end pins. They were trusses with sub-verticals erected over the Ohio River at Henderson, Kentucky in 1885. Thus simple trusses are not built for very long spans. For spans of 550 feet or more Cantilever arch or suspension systems are used.
CHAPTER IV. GENERAL MERITS.

Art. 10. Stiffness.

To meet the requirement of modern heavy fast trains, bridges are not only required to be strong but also stiff. There are many things effecting the stiffness, among which is the weight of the structure for stiffness against vibration. The small steel structures, such as the little T-rail and I-beam bridges as previously described, are very light. The action of the train on the bridge is like that of heavy hammer on a light anvil. They are very shallow and are really not stiff, when the ratio of the deflection and length is taken into consideration, but as the actual deflection is always small for such structures their stiffness is sufficient.

As the plate-girder bridge is heavier than truss bridges, its weight is an element for stiffness, but the stiffness due to the character of the type is even more important. The plate-girder is the stiffest of all the types of simple bridges. This is because the different parts of the plate-girder bridge is most intimately connected by rivets, thus reducing vibration at the passage of the train.

In a plate-girder the deflection of the bridge due to stresses on the flanges alone would be great as a ratio to the length, but the stresses on web is almost nothing on account of the excess of material in that part of the structure. The deflection due to shear in the web members for ordinary trusses is about as great as that in the flanges. In this way the plate-girder is about twice as stiff as a truss of the same span.
The deflection of a truss is a function of the depth and unit stresses on the different members rather than a quantity relating to the general outline. However, the deflection of the truss is proportional to the change of length of every member under stress. The change of length is directly proportional to the initial length. This fact tends to make the trusses stiffer for the smallest length of the members with the same depth. In this case, the sub-panel system tends to make the truss stiff as well as to save material on the flooring and truss members.

The change of length in a truss member is inversely proportional to the area of the cross section, but the types with smaller cross-sectional areas is more economical, and the value of stiffness is inappreciable, compared with that of economy. The change of length is directly proportional to the stresses on each member and the smaller stresses tend to make the deflection of the truss less. In this case the stresses that are carried most directly to the abutment tend to produce the least deflection. Moreover, the stiffest truss of a given type is the most economical as proven by Turneaure. Thus, economy and stiffness tend to go together as far as general outline is concerned, except for the cross-sectional areas of the members. In this connection, there is a tendency for the Pettit truss to be stiffest. The Baltimore truss would also be stiffer than the Whipple truss of the same weight, but such consideration is only of secondary importance, as the difference of stiffness is not great enough to control the adoption of types.

Riveted connections are the most important thing in imparting stiffness to the truss. This is due to the solid rivet connections for the joints as discussed in Art. 8. Steel bridges
of 200-foot span or less are riveted, as a rule; and this rule is due to the desire for stiffness. The stiffness gained by the use of riveted joints is so important that engineers are led to build a great majority of truss bridges riveted, as most bridges are of spans less than 200 feet.

According to present practice, most floor connections and lateral bracings are riveted for the sake of stiffness. There are many things affecting the stiffness of the trusses of a bridge among which is the flooring. Solid floors are good to make the bridges rigid as they do not allow up and down motions of the flooring. On the other hand, wooden stringers are very flexible and make the whole bridge vibratory. They have been used considerably in the early days, but are used no more. Steel floors are fairly good for stiffness and those with deeper stringers are less vibratory.

The length of panel also has an influence on rigidity of the bridge. Panel lengths about equal to the circumference of locomotive wheels are favorable for vibration which are mostly due to accumulating effects of centrifugal forces of the counter weights of the engine. This is about 17 feet 6 inches. On the other hand panel lengths of 25 feet is not so favorable for vibration. This question is important because the effect of vibration is important, often amounting to as much as 20 per cent of the stress in Pratt trusses of spans from 100 to 200 feet.

Art. 11. Economy of Material.

Economy of material is the most important thing that
SPANS AND WEIGHTS
OF
STANDARD BRIDGES
OF
A. T. & S. F. Ry

A = Deck Girders of Economic Depth
B = " Least
C = Thru With Deep Floor
D = " Shallow

Load: 2 - 139 Ton Locomotives Followed by Uniform Load of 3200 lbs per foot

Fig. 72.
influences the selection of a type of the bridge, except for very short spans when simplicity of construction is sought. It is well to know about the weights of bridges. It is a very easy matter to determine the weight of a structure after it is designed, but such weight cannot be determined accurately without having the design. Many formulae have been proposed for finding the weights of bridges of different types before they are designed, but such formulae are very rough and can only be used for trial design. They do not furnish reliable information for the choice of type.

Comparison of weights for the different types of bridges can be made easiest by examining the weights of bridges of different types for different span lengths. Fig. 72 is a chart showing the weights of the bridges that are built on standard plans of the Atchison Topeka and Santa Fé Railway, made in the year 1903 as described in Art. 5. By examining the chart the weights of all bridges are shown to be increasing with increasing span length. This increase is great for small increments of the span length when the span is long. The variation is much like that of a parabola as stated for beams in Art. 1. Deck bridges are shown to be more economical than through bridges for any span of any type. It is unfortunate that bridges of different types for same span lengths and loading are not available so as to compare the difference of type, but if the reader only imagines the lines extended as the broken lines in the chart, it is shown that curve top chord bridges are lighter than Pratt bridges, that Pratt bridges are lighter than Plate-girders, and that plate-girder bridges are lighter than I-beam bridges near the maximum and minimum spans.

The weights plotted on Fig. 72 are results of very careful
SPANS AND WEIGHTS
OF
STANDARD BRIDGES
OF
NORTHERN PACIFIC RY

Load: 2 14.6 Ton Locomotives
With Train Load of 4000 lbs per Foot

Span in Feet

Fig 73
designing. They are the shipping weights of many bridges by which the contractors are paid by the railroad company. They are the estimated weights plus 2 1/2 per cent to protect the contractor against uncertainty of variation.

Fig. 73 is a chart showing the weights of the standard bridges of the Northern Pacific Railway in the year 1900. It shows that Plate-girder bridges are lighter than I-beam bridges, and that lattice or Warren trusses with sub-verticals are lighter than plate-girders, but it indicates that the Lattice bridges are also lighter than pin-connected Pratt bridges. In all cases, deck bridges are lighter than through bridges of the same type for the same span length.

However, the weights of bridges do not necessarily determine their relative cost. This is because the cost of the members of a bridge is not strictly proportional to the weight. For instance, the increase in sectional area of a member does not increase the cost of shop work. The increased cost of erection is almost nothing. And the only increase in cost is due to the increased amount of raw material and freight expenses. For short span bridges, plate-girder and beam bridges are even more economical than truss bridges in spite of greater weight. This is because the former are easier to manufacture and erect.

When both the weight and pound price of a bridge is known the exact cost of the structure can be determined. The pound price depends greatly upon the style of the bridge on account of different requirements of manufacture. It also depends upon the cost of material and erection. It is different at different times, depending upon market conditions. As stated by Waddell, the average
pound prices for carbon steel bridges erected in the United States in the year 1909 were as follows:

- Plate-girder spans: 4.0 cents.
- Riveted truss spans: 4.5 cents.
- Pin-connected Pratt truss spans: 4.5 cents.
- Pin-connected Pettit truss spans: 5.0 cents.

Thus the total cost of a bridge is very hard to calculate, and can only be found by results of experience, but the types generally adopted are the best for the different conditions met in practice, and custom is a guide in this case.


Great deal of care and money are necessary for a railroad to maintain the bridges. The bridges are constantly watched. Thorough inspections of all the bridges are necessary, and are generally made as often as every three months. Many railroads paint their bridges completely every five years. Besides, repairs are made occasionally, especially for weak parts.

The ease of maintenance depends more upon the plausibility of design, detail construction, material and conditions effecting the bridge rather than the general outline or form of structure. The life of the bridge depends upon the attention paid to maintenance. However, riveted joints render the structure easy to maintain, and they are preferrable in this respect. The rivets should be tight and the bridge should be rigid to let them remain tight. As localized stresses in a flexible structure lossens the rivets, many more loose rivets are found in flexible bridges than
stiff ones. The different parts should also be well proportioned to prevent localized stress which tend to loosen the rivets.

The plate-girder is the stiffest form of structure and the stresses are distributed most uniformly. In all probability, it is the easiest type of bridges to maintain, and is the most durable. As Mr. T. C. Clark said in a meeting of the American Society of Civil Engineers in the year 1895, "Experience shows that the durability of plate-girders is greater than that of any other form of construction."

Plate-girders are also easier than trusses to brace up or reinforce to carry a greater load. Thus, its life is prolonged. As often happens, all the parts except the flanges of plate-girders are too weak for the overload. Plates can be added to the flanges of plate-girders to brace it up. In one case in the Boston and Main Railway, two 12 by 1/2 inch plates were added to the angles of the flanges of a 60-foot plate-girder bridge and the structure became quite strong to carry the load greater than that for which it was originally designed.

As riveted trusses are stiff with tight joints, they are next to the plate-girders in durability. The joints are very easily repaired by replacing the rivets when they become loose. Besides, riveted trusses, especially the multiple lattice trusses, do not collapse so readily as pin-connected bridges, thus their durability against accident is greater.

As pin-connected trusses are the most flexible with loose joints, they are not very durable. They are also hard to repair and sensitive to injury, collapsing more readily than riveted truss, but the question of durability has less influence on the types of
bridges than many other questions, as the difference of life is not very great and ease of maintenance is relatively unimportant regarding the difference of type.

The life of railway bridges is mostly determined by unexpected overloading. When the load to be carried would cause the stresses in the bridges exceed the safe working unit stress, bridges have to be repaired or renewed. In most cases the stresses in all the members of the girders are nearly the same for economic design, and renewal of the whole bridge is necessary for overloading. In general, the life of a bridge is shortened to about 20 years by unexpected overloading, which is nearly the same as the time for complete renewal of a locomotive which is about 20.8 years as given for the Northern Pacific Railway. However, many bridges have been in service for as long as 25 years, especially the riveted lattice girders on the New York Central Railway constructed about 1870.

Following are some statistics taken from Engineering Contracting:

<table>
<thead>
<tr>
<th>Item</th>
<th>Name of Railroad</th>
<th>Location of Bridge</th>
<th>When Built</th>
<th>Life in Years</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Chicago, Milwaukee &amp; St. Paul</td>
<td>Rock River</td>
<td>1884</td>
<td>18</td>
</tr>
<tr>
<td>2</td>
<td>Wabash</td>
<td>Sagamon River</td>
<td>1885</td>
<td>21</td>
</tr>
<tr>
<td>3</td>
<td>Chicago, Burlington &amp; Quincy</td>
<td>Big Rock Creek</td>
<td>1881</td>
<td>22</td>
</tr>
<tr>
<td>4</td>
<td>Illinois Central</td>
<td>Big Muddy River</td>
<td>1889</td>
<td>13</td>
</tr>
<tr>
<td>5</td>
<td>Illinois Central</td>
<td>Tennessee River</td>
<td>1888</td>
<td>17</td>
</tr>
<tr>
<td>6</td>
<td>Chicago &amp; Northwestern</td>
<td>Kinnikinnic River</td>
<td>1880</td>
<td>19</td>
</tr>
<tr>
<td>7</td>
<td>Pierre Marquette</td>
<td>St. Joseph River</td>
<td>1887</td>
<td>17</td>
</tr>
<tr>
<td>8</td>
<td>Grand Trunk</td>
<td>Niagara River</td>
<td>1877</td>
<td>19</td>
</tr>
<tr>
<td>9</td>
<td>Chicago, Milwaukee &amp; St. Paul</td>
<td>Minomnee River</td>
<td>1886</td>
<td>17</td>
</tr>
<tr>
<td>10</td>
<td>Central Railway of New Jersey</td>
<td>Newark Bay</td>
<td>1887</td>
<td>17</td>
</tr>
</tbody>
</table>

Average--- 18.1
The average life of the ten structures shown about is 18.1 years. The uncertain variations are also suggested by the lives of several bridges.

However, the bridge that has the longest life stood up for 80 years from 1823 to 1903 although it was not in service a great part of the time. This was the first iron simple bridge ever built as described in Art. 2. It was the Gaunless River bridge on the line of the Stockton and Darlington Railway at West Auckland, England.

It is almost unnecessary to say that unfavorable conditions shorten the life of railway bridges. This, again, is independent of the type of bridge except for types whose parts can not be conveniently painted. The effects of fogs and rain and of brine on bridges and structures which carry trains with cargoes of salted meat, is a great problem for the engineers. In this case, the floor with the steel protected from the brine is the most endurable.


Aesthetics are also something which should concern one in the design of the railroad bridge, but the least important. For highway bridges that are seen by many people, towers are often erected at the abutments to produce a pleasing effect. Decorations are also sometimes placed on highway bridges, but for railway bridges, neither adoption of weighty decorations nor expensive towers is justifiable. Railway bridges are seen more at a distance than close by and neither the effect of towers nor the decorations can be had at a distance. Moreover, railway bridges are not seen by very many people and their appearance becomes unimportant. Plates and angles
are often made into curved outlines for the sake of appearance. This is bad practice as it involves great cost of manufacture.

Any type that is strong and economical is by no means offensive to the eyes. The trusses of multiple webbing seem to be the least beautiful. The plate-girder is not ugly, and neither are the Pratt and Warren trusses. The curve top chord trusses are beautiful, especially the Parker and Pettit trusses. The bowstring truss is the most beautiful of all. As the curved top chord trusses are strong and economical, strength, economy and beauty go together, making a good combination of qualities.
CONCLUSION.

There are many things having an influence on the usefulness and economy of the different types of railway bridges, of which the theory of the beam is a very important one. Such things as the distribution of stresses in the different fibres of the same members and the distribution of total stresses in the different members are very important for economy and strength. The principles of inclined chord, and short panels by the use of suspenders, sub-trusses and multiple webbing are also important on account of their use to economize material and to increase the possible span length.

As the important principles that make good truss bridges were not known at the beginning, bridges were not built on a scientific basis. At that time modifications of earlier forms of bridges were made and thus resulted structures somewhat like arched and suspension bridges. Afterward many different types were devised and the best were retained, while the poorer types were rejected and came into disuse. Many different forms of the beam have also been used as tubular, cellular, rolled and built up beams and cast iron girders with and without wrought iron reinforcement; but now only the rolled and built-up beams are considered good and used.

The development of the beam is due to English engineers but the development of the truss is due to American engineers, especially to Whipple. Knowing the secret of truss bridges, most of the trusses were of the panel system. These are the Howe, Pratt, and Whipple trusses. First wood was used, then cast iron and wrought iron; and when steel became cheap, most of the railway bridges were built with it. With the use of this material, more economical types can be used to advantage, and bridge engineering was pushed
forward very rapidly; but for the last twenty-five years very little improvement has been made on the general form of simple bridges, and improvement is mostly made in the details of constructions. In the early days, each bridge engineer had his own types and methods of construction; but now, not only the general design, but even the details are more or less the same throughout this country. Moreover, many railroads have standard designs with which hundreds of bridges are built, and this method is found very economical.

The elevation of the track with reference to the waterway determines the use of deck or through bridges, but as deck bridges are usually cheaper, it should be adopted whenever possible. Moreover, the depth of floor may be made such as to reduce expenses on the track, and for this reason the railroads have different classes of bridges with different depths of flooring.

For the different types of bridges and different conditions of the stream, different methods of erection are employed. As a general rule, the cost of erection of I-Beams and plate-girder bridges is the least, and the cost of short truss bridges is less than that of longer ones. The cost of erection of a pin-connected bridge is less than that of a riveted bridge.

There are many advantages and disadvantages of the riveted and pin-connected bridges of which the most important in favor of the former is stiffness and the most important in favor of the latter is economy and ease of erection. As the weight and speed of the train are great, the riveted truss is preferrable for spans of 200 feet or less, thus leaving the longer spans for the pin-connected bridge.

As the weight is one of the most important things effecting economy, it is always made as small as possible and more attention
is paid to this item as the span becomes longer. Maintenance and life of the structure are also important, the former depending upon good detail design and construction, and the latter depending upon the overloading of the structure.

All general conditions being considered, rolled sections are the best for very short spans and plate-girders for spans a little longer, up to about 100 feet; parallel chord trusses for spans from 100 to about 200 feet; and curved top chord trusses, for spans from 200 to 550 feet. For the present, most of the bridges are of the plate-girder and Pratt type; and as the types now in use are very good ones, the development of simple bridges is only the line of detail construction for the present.