SOO-HOO

Economic Development

of Bridge Construction

Civil Engineering

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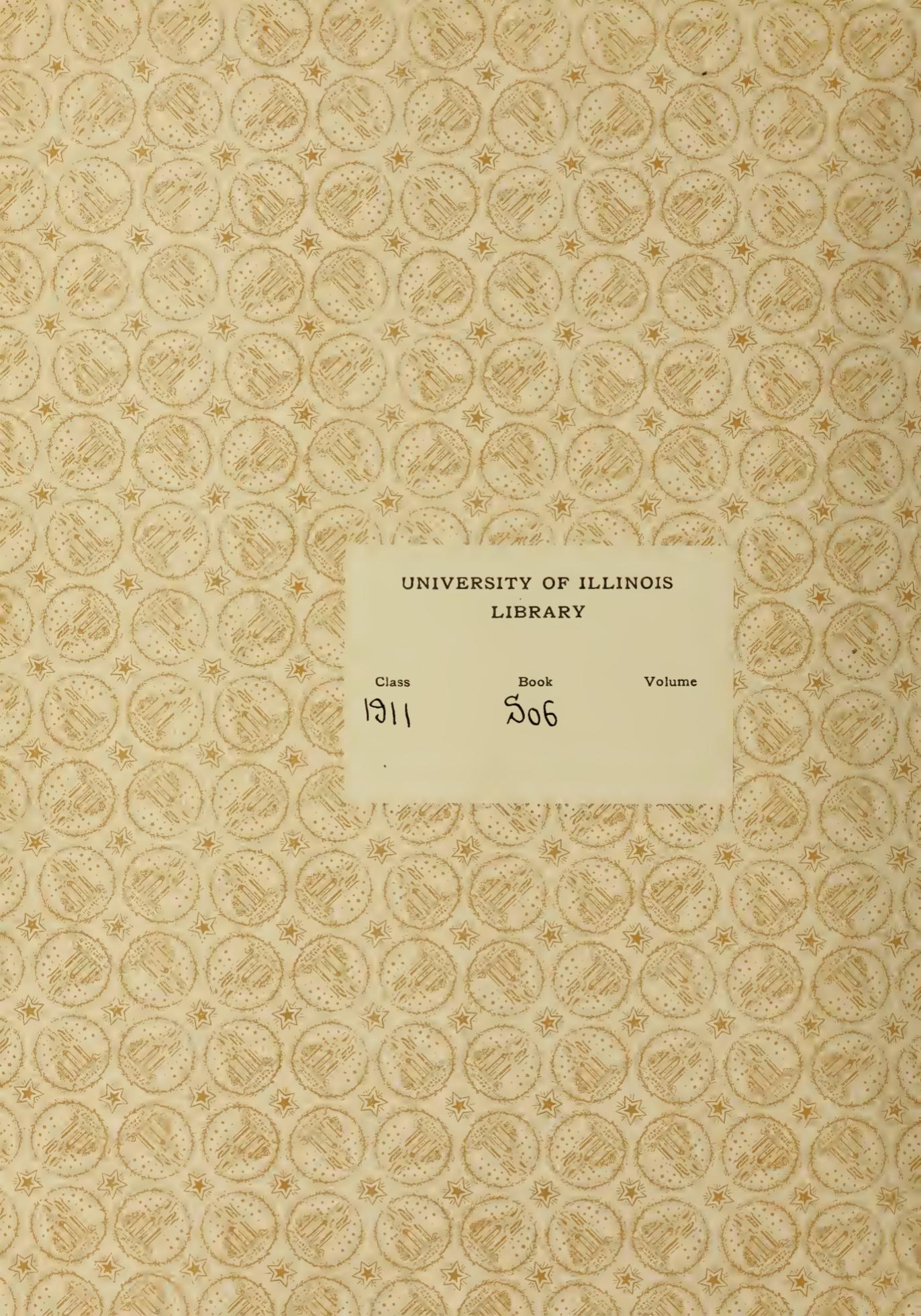
ECONOMIC DEVELOPMENT OF BRIDGE CONSTRUCTION

BY

PETER SOO-HOO
A. B., LELAND STANFORD, JR., UNIVERSITY, 1910

THESIS

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PREFACE.

The relative merits of the different types of simple rail-
way 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 differant
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 some-
times called ingot iron.

The importance of the subject lies in the fact that certain
types of steel railway bridges are most economical and that differ-
ent 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 stand-
ardized. 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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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 ex-
cept for the applications. All the theories concerned are the prin-
ciples of parallogram 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 econom-
ical and scientific. It is a science as well as an art.

CHAPTER 1. DEVELOPMENT OF TYPES.

Art. 1. Controlling Principles.

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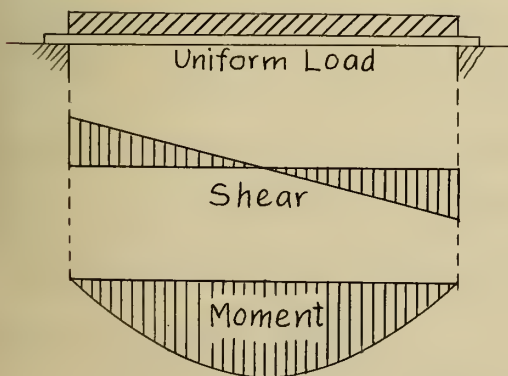
Let M denote maximum bending moment; L, span length; w, load per unit length; S, allowable unit stress for the material of the beam;

Uniform Load

Shear

Moment

Fig. 1.

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 $M = \frac{1}{8} w l^2$, $S \frac{I}{c} = M$, and the sectional modulus $\frac{I}{c} = \frac{w l^2}{8 S}$

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 $ad = \dfrac{Wl^2}{8S}$
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 pre- caution. 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.
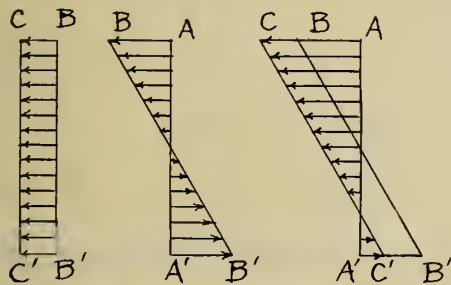
Economy is attained by having the stress uniform throughout

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,
$$\frac{P}{a} = S - k\frac{l}{r}$$

P denotes the longitudinal axial force in pounds acting on the
column; a, cross-sectional area of column in square inches; S, unit

stress 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 $k\frac{l}{r}$ 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-homogeniety 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

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 tri-
angles 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.

Inclination Angle in Degrees.

Fig. 4.

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 Cd be the truss member in question; V be the verticle force, and α be the angle of inclination. Then, the stress,

$$S = V \operatorname{cosecant} \alpha$$

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°. 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 verticel. 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 effected 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 tripple 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

Fig. 5.

Fig. 6.

Fig. 7.

Fig. 8.
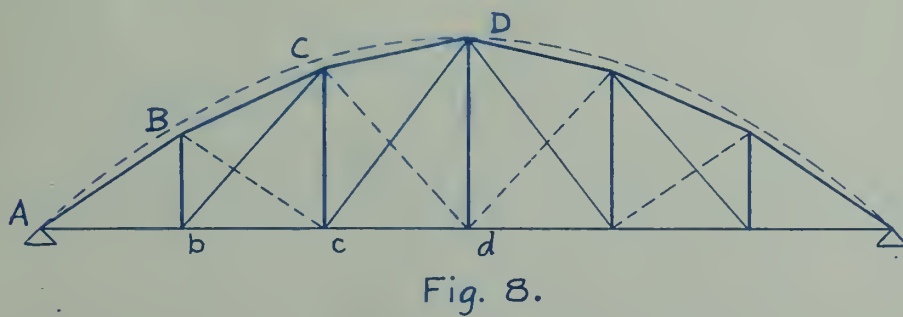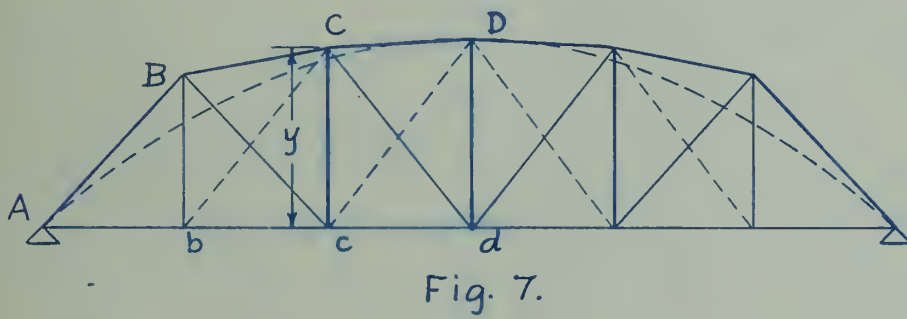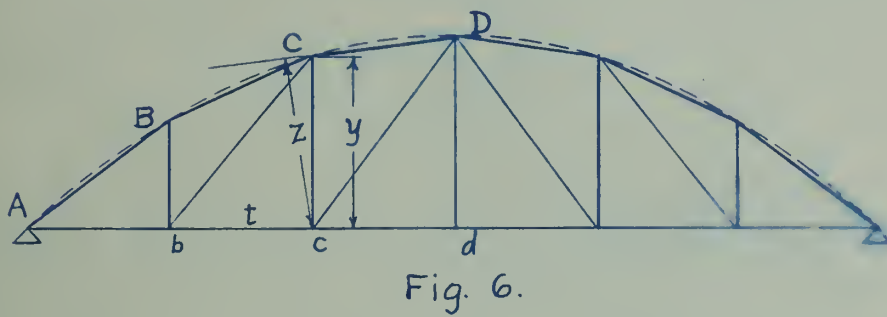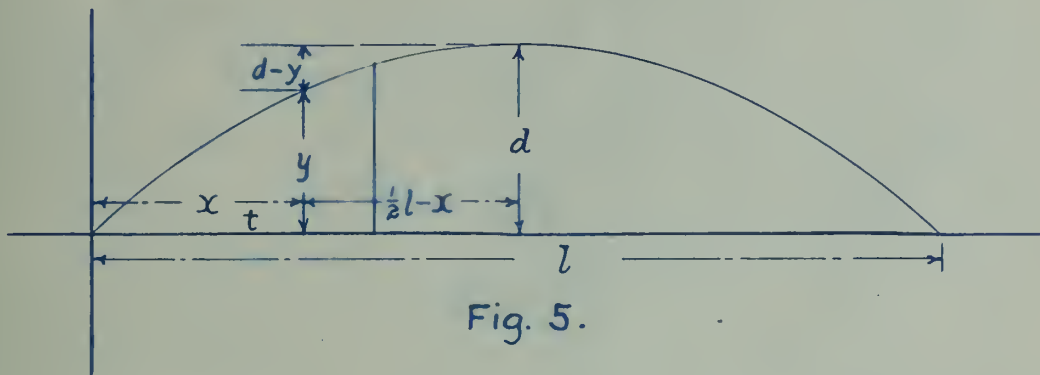
the loading like that for maximum chord stresses in a long span.
Let w be uniform load per unit length; and y be the heighth of the
panel point; C which is at distance x from the left end of the
span (see Fig. 5.) Let l be the length of span; d be the maximum
depth of the truss; and S be the stress in 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, $(\frac{1}{2}l - x) = k(d-y)$ 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,

$$S = \frac{wl^2}{8d}$$

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 thorughout. 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 differ-
ence 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 verticle 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,

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

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 begin-
ning 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, dicussed 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.
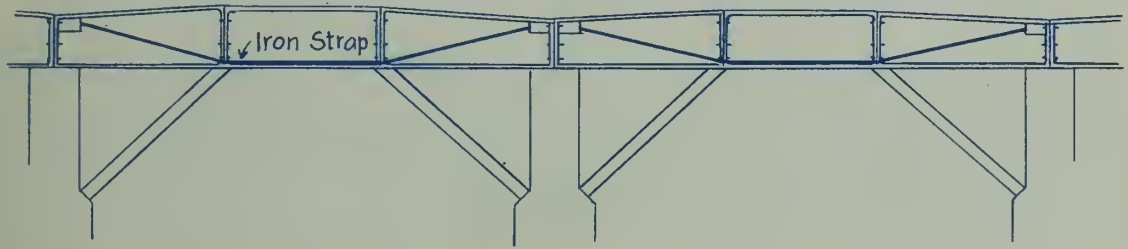
Fig. 9.
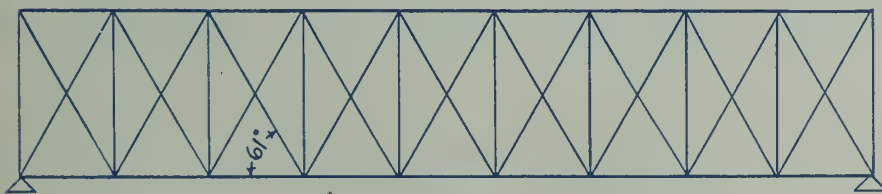
Fig. 10.

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 goverment 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

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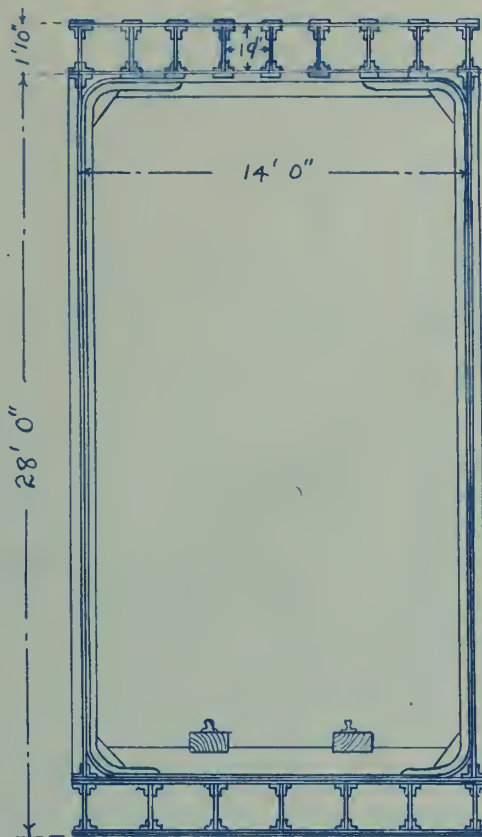
Fig. 11   Cross-section of Tube of Britania Bridge.
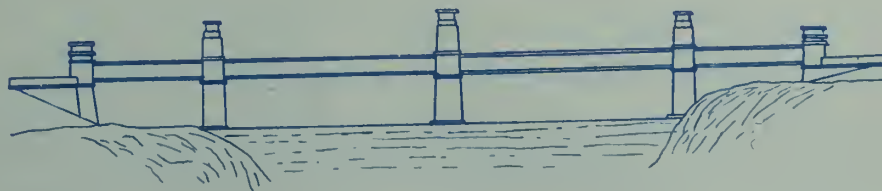
Elevation of Bridge.
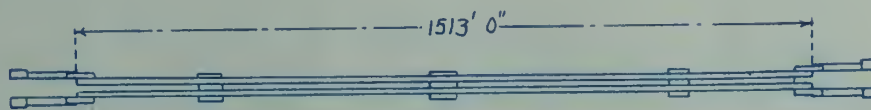
Plan of Bridge.

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 Brittannia 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 scaffords 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.
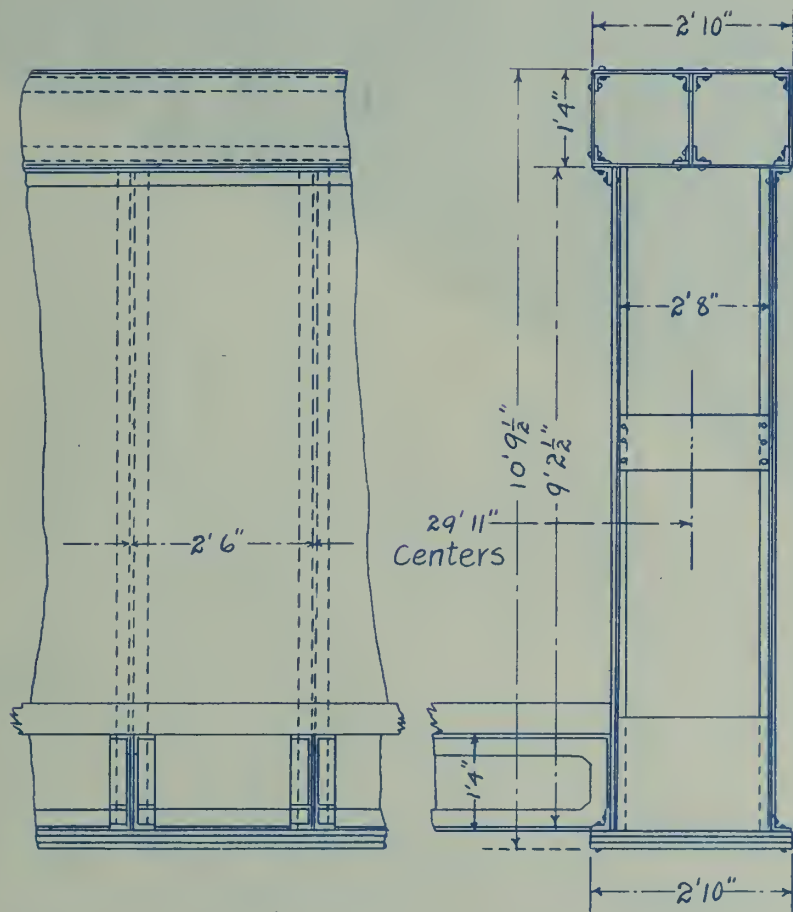
Sectional Elevation.   Cross Section.

Fig. 13   Girder of Bridge over River Swale at Maunby, England.
Length 155 Feet.
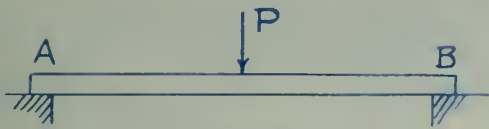
Fig. 14.
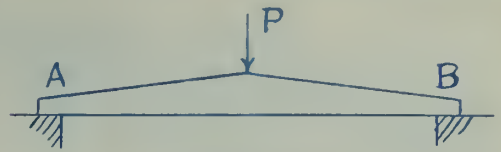
Fig. 15.
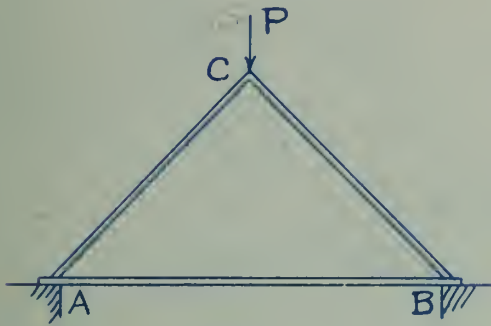
Fig. 16.
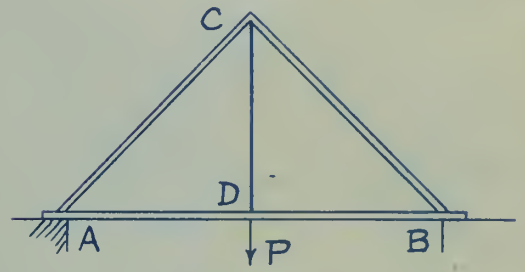
Fig. 17.
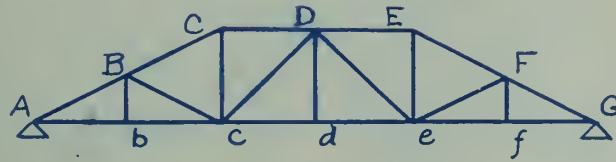
Fig. 18.
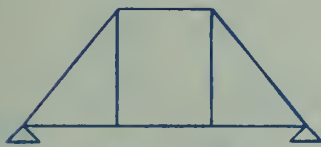
Fig. 19.
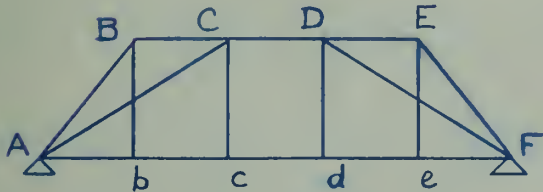
Fig. 20.

Fig. 22.

Fig. 21.

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 verticle walls were made of thin plates to make the structure light. As the verticle 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 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. 18, which represents the outline of this bridge, the elementary King-post trusses, ABcbA, cDedc, and eFGfe are easily seen as are also the halves of trusses, ACc 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.
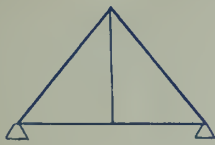
Fig. 24.
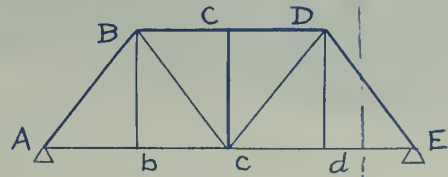
Fig. 28.
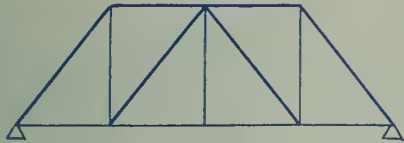
Fig. 25.
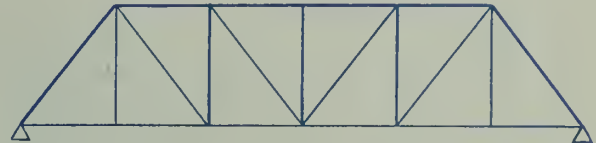
Fig. 29.
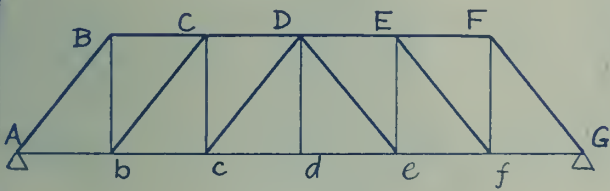
Fig. 26.
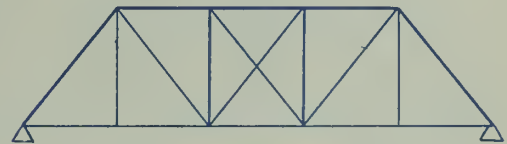
Fig. 30.
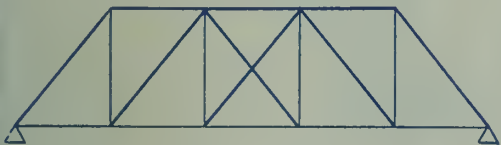
Fig. 27.
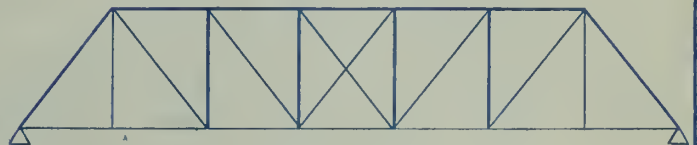
Fig. 31.
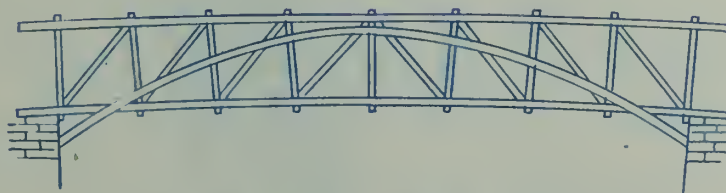
Fig. 32   Burr   Truss.

During the Seventeenth and Eighteenth Centuries many wooden bridges have been built of such general outline. In 1758 Gruben-man built a great timber bridge of 366-foot span on this plan. Such a type is uneconomical for the numerous long braces are sub-jected 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 preceeding 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 gen-
eral 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 pro-
gress 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
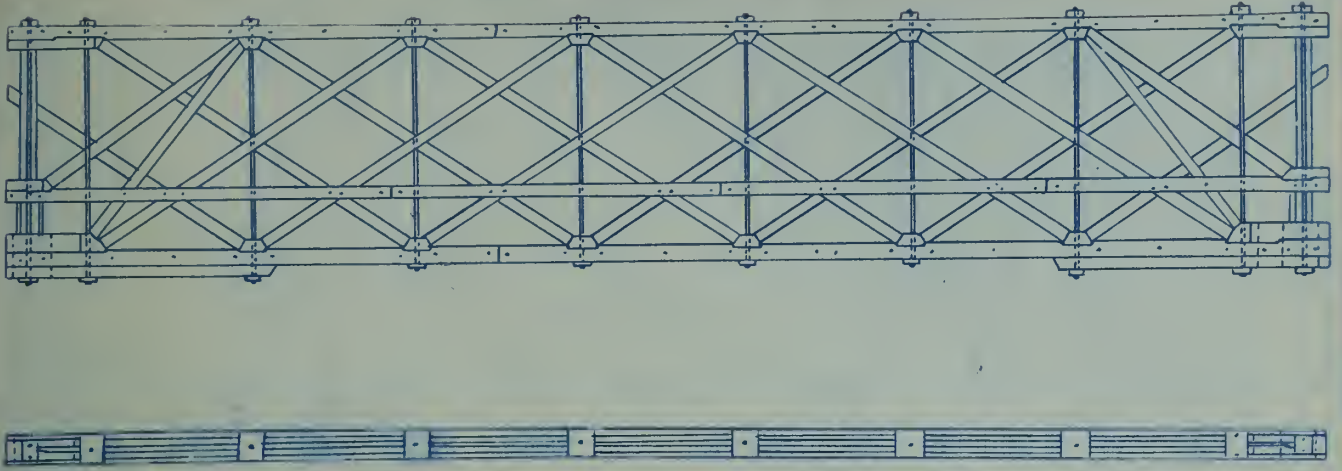
Fig. 33 Original Howe Truss.
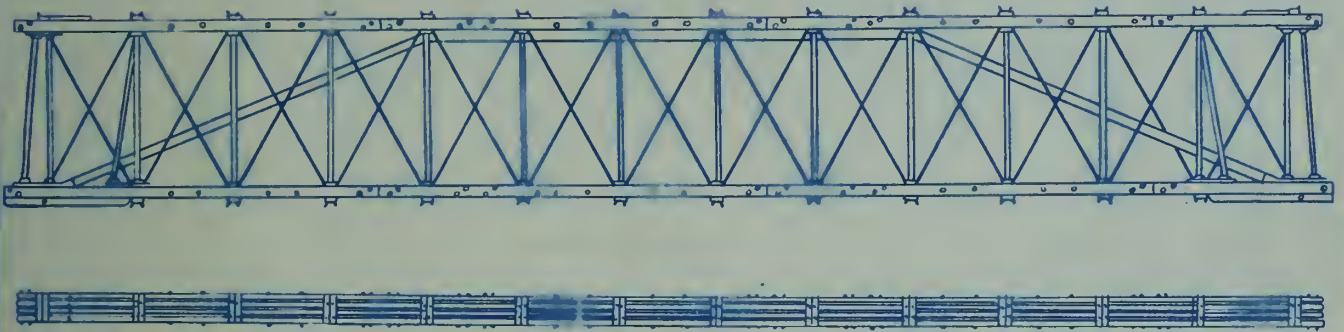
Fig. 34 Original Pratt Truss.
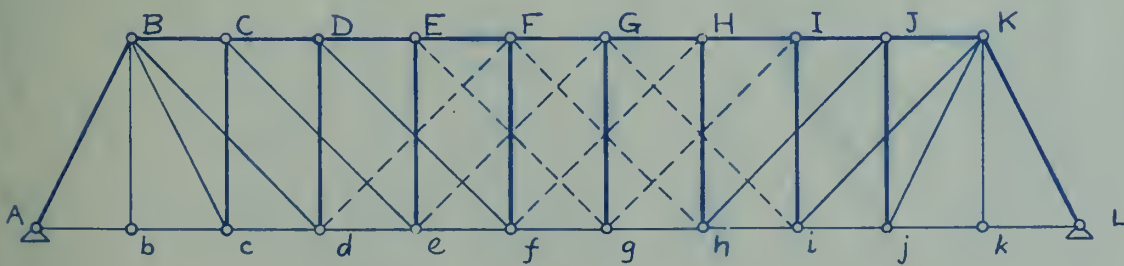
Fig. 35 Whipple Truss.

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 recog-
nized, 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 in-
vention 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 pub-
lished 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 investi-
gations 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 supressed 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 Whipples important little book. Among which was the omission of the vertical end posts for through bridges. Previous to 1847, the verticle 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".

340' 3¾"
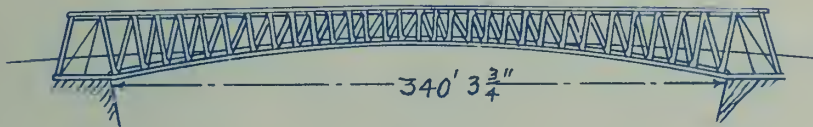
Fig. 37 Colossus Bridge.
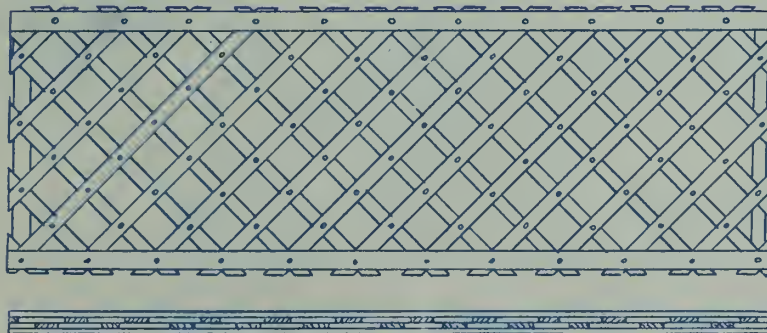
Fig. 38 Portion of Town Truss.

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 Schuykill 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 Schuykill 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 peo-
ple 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 con-
sisted of timbers of uniform dimensions, and the truss has been
built without iron holts, 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 rail-
road 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 stress-
es 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-
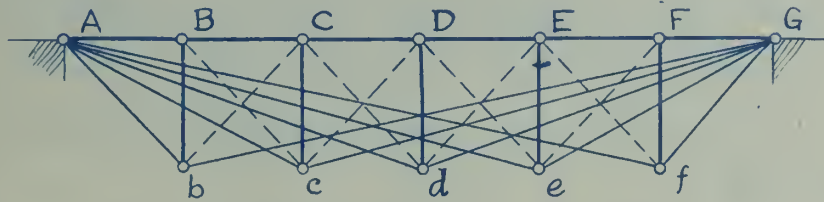
Fig. 39   Bollman   Truss.
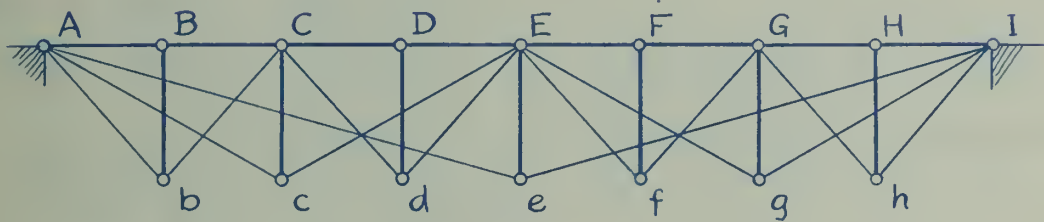
Fig. 40   Fink   Truss.
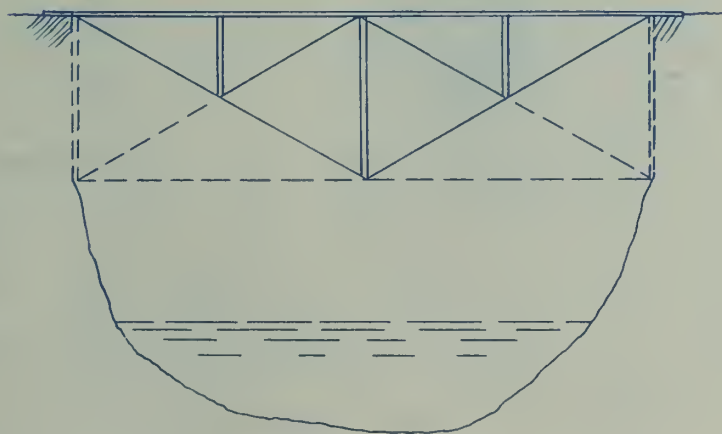
Fig. 41   Fink's   Military   Bridge.

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 abut-
ments 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 1880. The 124-
foot span Bollman truss bridge built in 1852, across the Potomac Ri-
ver at Harper's Ferry, Virginia, is the most famous bridge of this
type and is a good example. The trusses had eight panels 18 feet
deep. The top chord and posts were of cast iron and the inclined
members were of wrought iron. It carried railroad traffic for for-
ty-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 AeI 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-

Fig. 42.
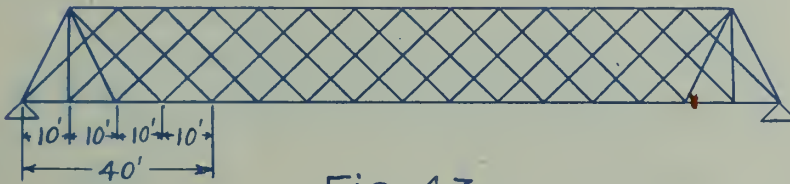
Fig. 43.
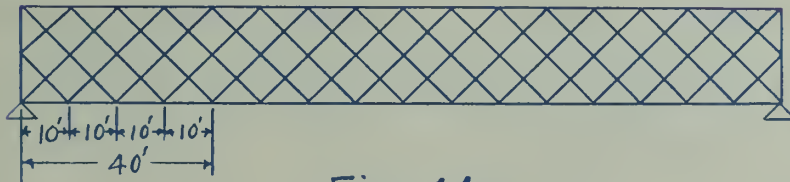
Fig. 44.
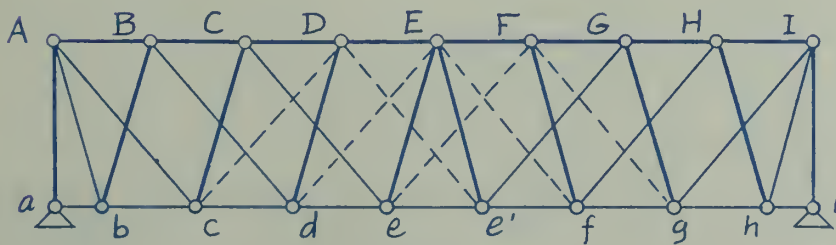
Fig. 45.

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 subjecteed 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 Whiple 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.

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 1880, 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 1850. 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 1859. The one in the United States was built in the year 1846 and 1847 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
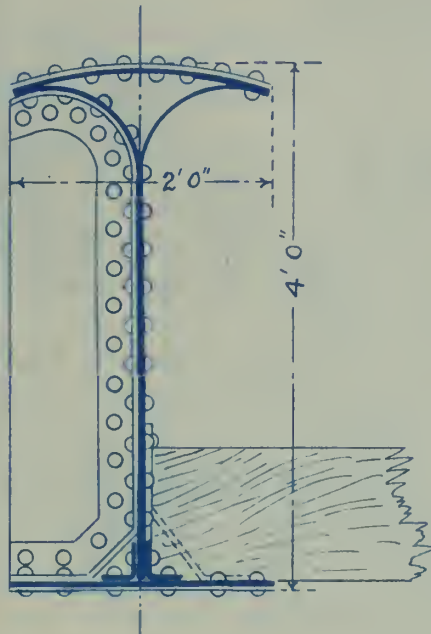
Fig. 46   Section of Girder Over Staindrop Road.
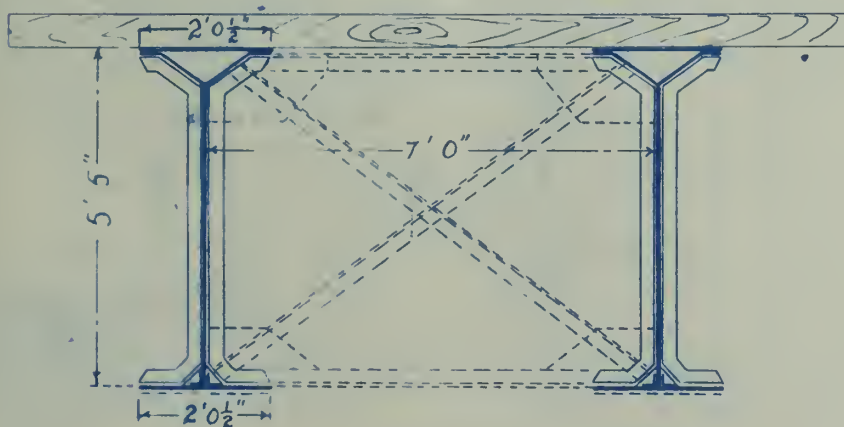
Fig. 47   Section of Musgrave Bridge.

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 bot-
tom being more than is necessary for other types.

For spans of about 200 feet or less, girders with box sec-
tions 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

Table I. Single Track Railway Girder Bridges.

No.	Span Bet. Centers of Bearings	Type	Date of Erection	Over What River	Location	Name of Engineer	No of Panels	Depth	Live Load per Foot in Pounds 2,240#	Weight of Trusses in Tons of 2,240#	Remarks
1	307	Whipple	1877	Susquehanna	Havre de Grace, U.S.	Phoenix Br. Co.	17	35'	2,240	215	Pin, Rolled iron.
2	330	Tubular	1859	St. Lawrence	Montreal, Canada	Robert Stephenson		22', 30'	2,240	686	
3	342	Whipple	1870	Ohio	Parkersberg & Belleaire, U.S.	J.H. Linville	22	33'	3,000	338	Pin, Rolled iron.
4	345	Lenticular	1862	Rhine	Mayence, Germany	Gerber	13	24'6", 49'2"	2,890	359	Riveted.
5	368	Warren	1870	Ohio	Louisville, U.S.	Albert Fink	24	46'	2,600	479	Top chord cast iron Rest Wrought iron, Pin.
6	375	Whipple	1877	Kentucky	Dixville, U.S.	C. Shaler Smith	20	37' 6"	2,037	425	Pin, Rolled iron.
7	396	Warren	1870	Ohio	Louisville, U.S.	Albert Fink	28	46'	2,600	623	Pin, Top chord cast iron, Rest rolled iron.
8	397	Close Lattice	1856	Vistula	Dirschau, Germany	Lentze		28'6"	2,128	838	Rolled iron.
9	400	Tubular	1848		Conway, England	Robert Stephenson		25'6"	2,240	1,112	
10	415	Whipple	1871	Ohio	Cincinnati, U.S.	J.H. Linville	20	41'6"	4,500	830	Pin, With highway.
11	420	Lattice	1861	Inn	Passau, England		23	27'	2,210	327	Riveted, Rolled iron.
12	455	Lenticular	1859		Saltash, England	I. K. Brunnel	12	30', 60'	2,240	945	
13	460	Tubular	1850	Menai Str.	Britannia, Wales	Robert Stephenson		22'9", 30'0"	2,240	1,553	
14	515	Whipple	1877	Ohio	Cincinnati, U.S.	J. H. Linville	20	51'5"	1,818	1,176	Pin, Rolled iron.

longitudinally and transversely. The construction was most diffi-
cult 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 ad-
ded 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 construct-
ion.

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 Insti-
tute 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 popu-
lar on account of the difficulty of construction. However, the type
was used in Europe. The lattice girders also were not in this coun-
try 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
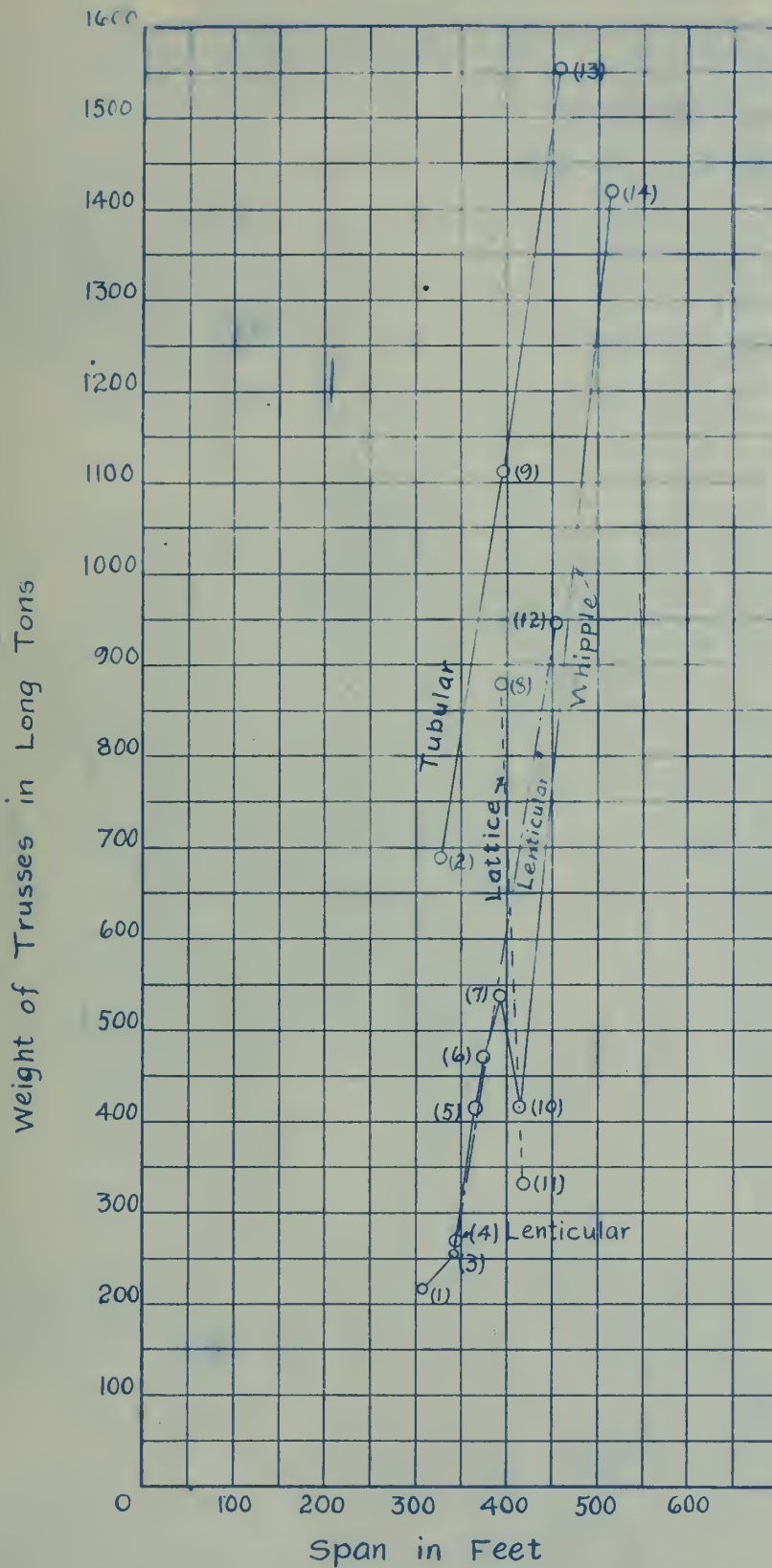
Fig.48 Relation of Weight and Span of Bridges Given on Table I
For Live Load of 2,240 Pounds Per Linear Foot.

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 struct-

ures were considered unusual in magnitude, at the time of their er-

ection.

Fig. 48 is a chart showing the weights of the fourteen bridg-

es 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 varia-

tion of the weights for spans of nearly the same length due to dif-

ferent 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 consid-

ering the condition in one of the important roads. In the year
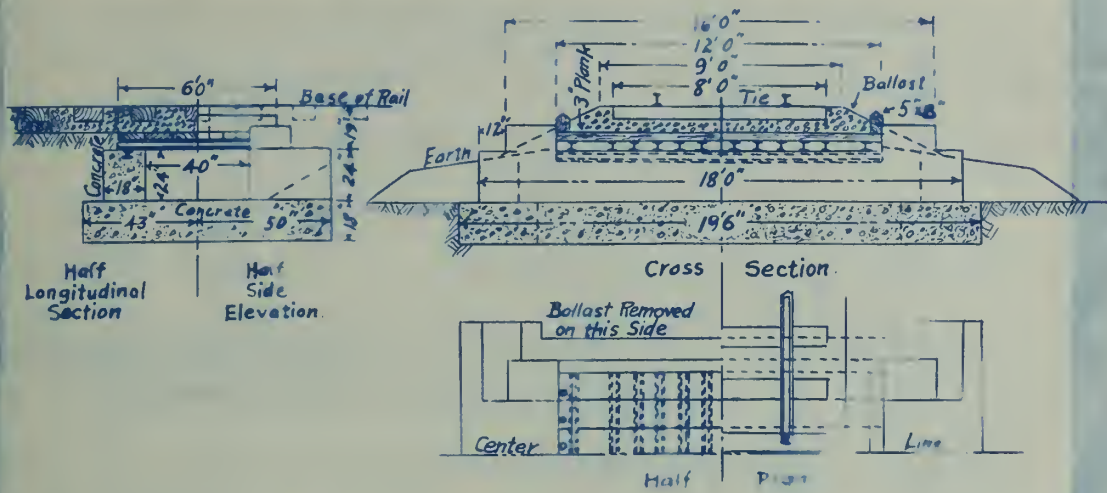
Fig. 49 Rail Top Culvert, A. T. & S. F. Ry.
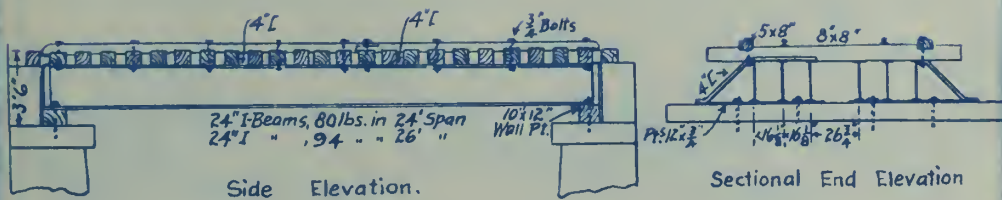
Fig. 50 I-beam Bridges of 24 Ft And 26 Ft Span.

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 to-
gether. 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 bend-
ing 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 some-
times made less than the economic depth to reduce the level of the
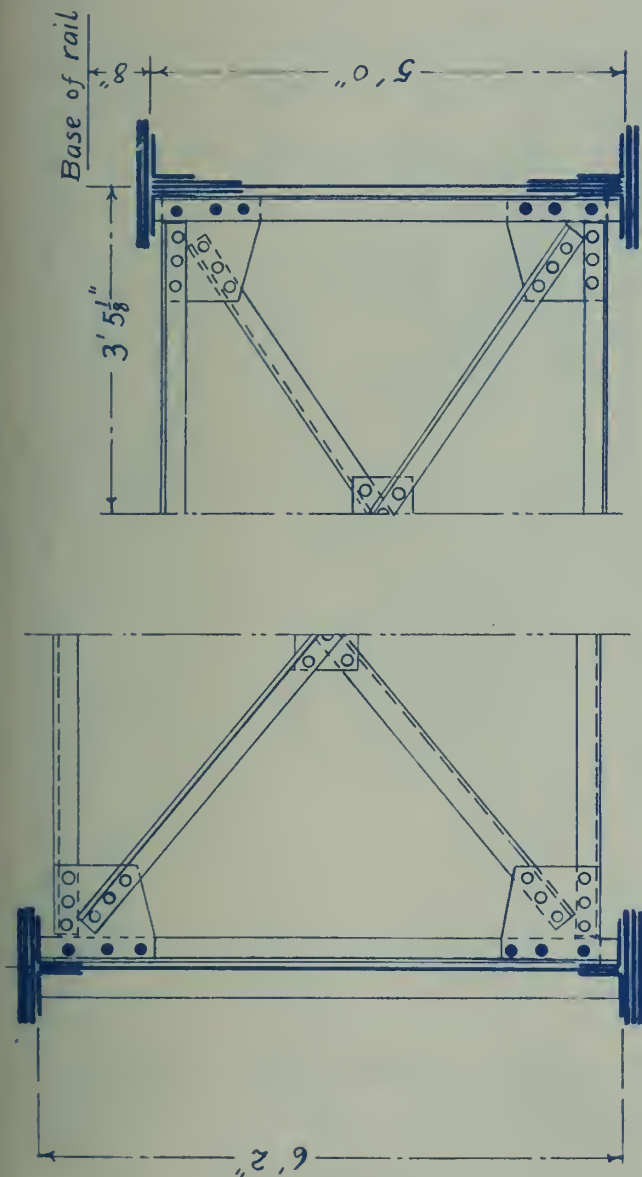
Class A
Semi-Sections of Deck Plate-Girder Spans Showing
Intermediate Cross Frames.
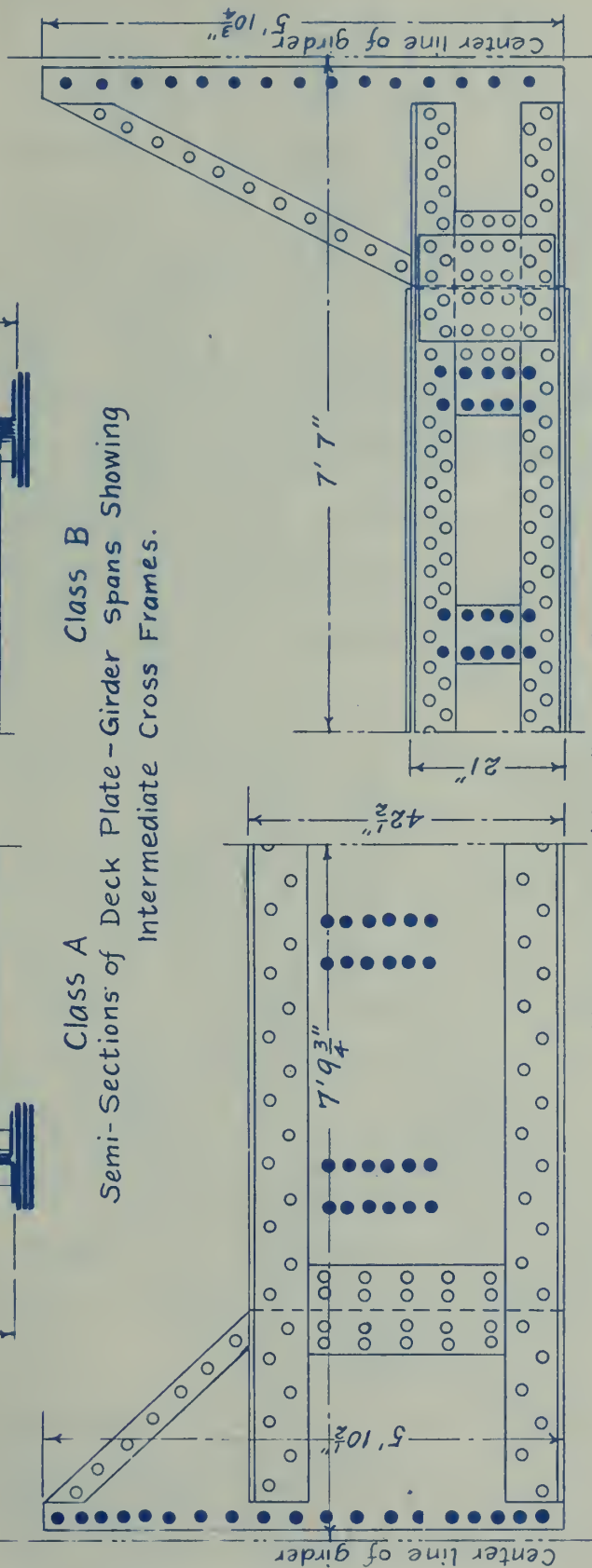
Class B
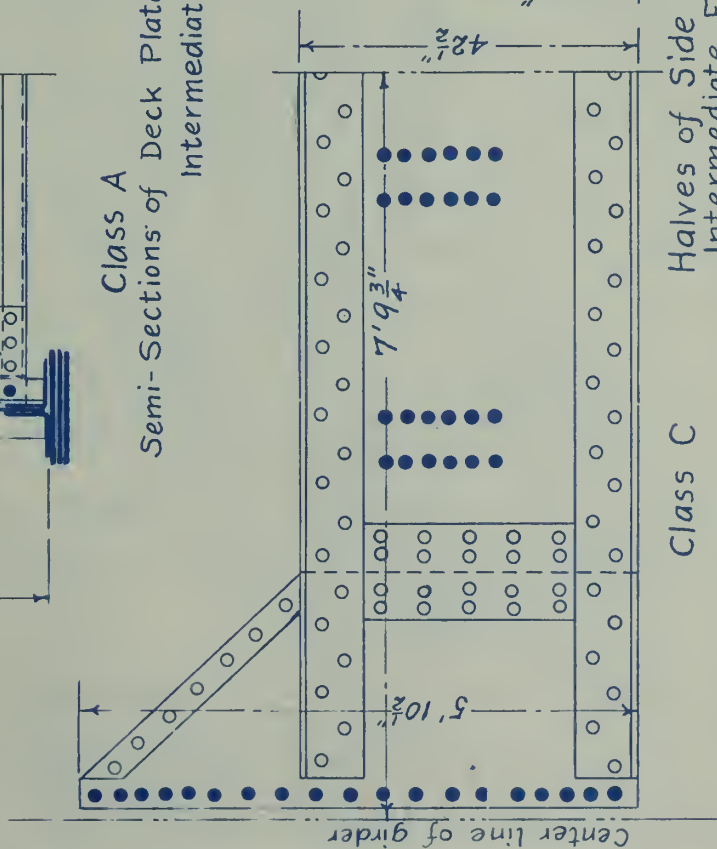
Class C
Halves of Side Views of
Intermediate Floor Beams of Through Plate-Girder Spans.

Class D

Fig.51 Showing Parts of 64-Foot Standardized Plate-Girder Spans of Atchison, Topeka
and Santa Fé 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 num-
ber 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 satis-
factory.

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 of-
ten used to render immediate support of flooring to shorten the pan-
els, 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 ten-
sion 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 in-
clined 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 un-
suitable as it is hard to construct such joints for both tension

Fig. 52.

Fig. 54.

Fig. 53.

Fig. 55.
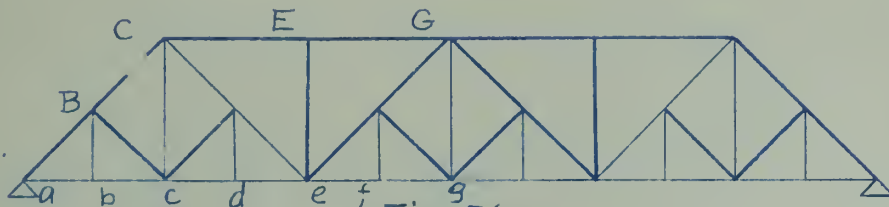
Fig. 56.
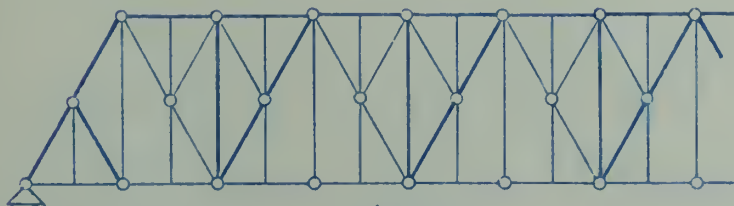
Fig. 57.

and compression. This is one reason why the Warren truss is gener-
ally 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 em-
ploys 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 an-
gles, 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 alter-
nate 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 popu-
lar 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 Louis-
ville, 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 web-
bing discussed in Art. 4.

The Howe truss was originally intended for wooden construct-
ion 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 occasion-
ally, 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 War-
ren and Pratt trusses. The main diagonals slope upward toward the
middle of the truss, and the stresses in these diagonals are com-
pressive for positive shear. As wood is suitable for compression
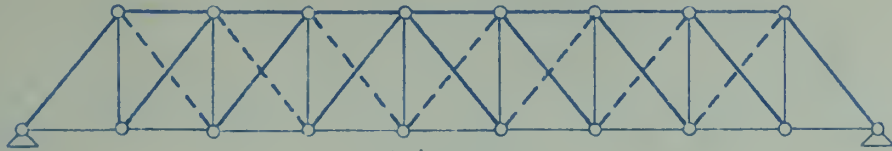
Fig. 58.
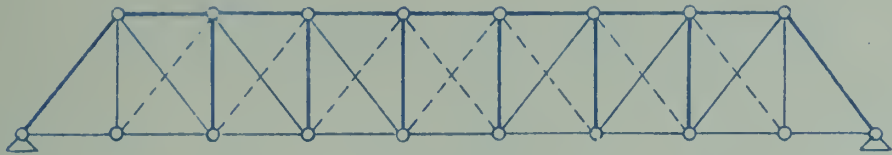
Fig. 59.
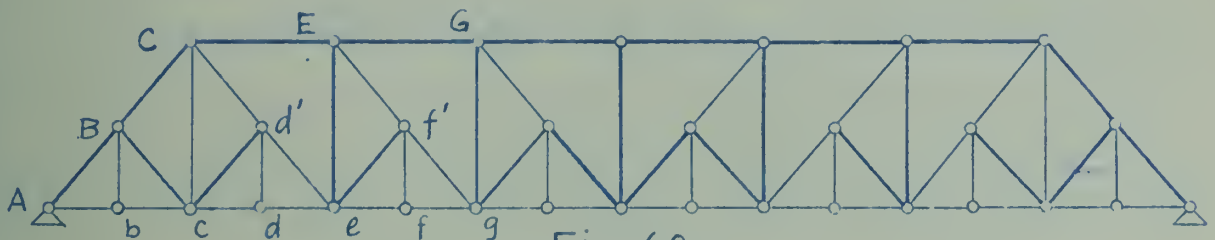
Fig. 60.
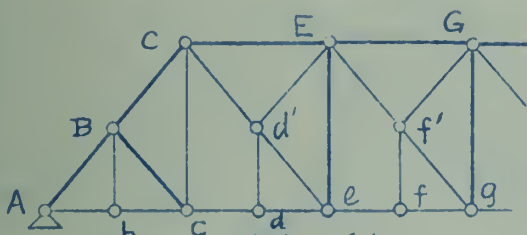
Fig. 61.
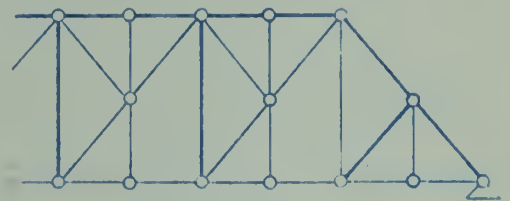
Fig. 62.
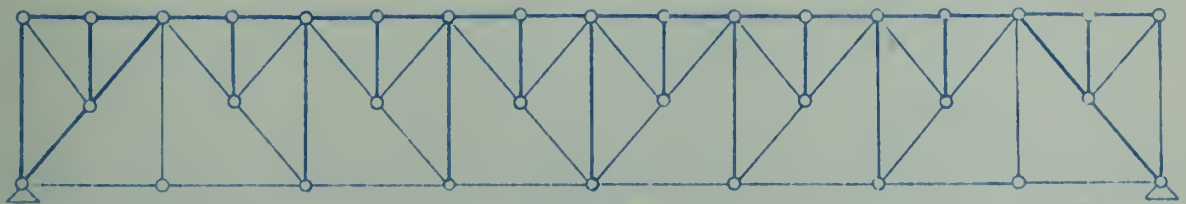
Fig. 63.

Fig. 64.
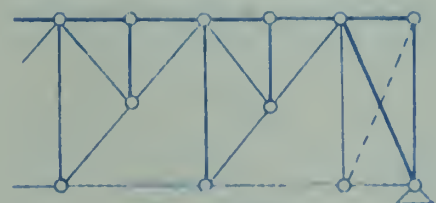
Fig. 65.

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 verticle 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 Have 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, particuliarly 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 necess- ary when the maximum compression due to live load exceeds the ten- sion 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 presnt 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 Pennsyl- vania 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 abailable 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 pro-
portion. 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 De-
partment of the Pennsylvania Railroad in 1871. The Baltimore Bridge
Company were the first to build bridges of this type. They adver-
tised 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 al-
most 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 ad-
dition 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 compres-
sion 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 in-
clined end post only extends over one panel and its slope is very
great as appears in Fig. 65. Fig. 65 also shows a possible arrange-
ment 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 design-
ing 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 di-
rectly to the piers, besides the sub-struts have about twice the a-
rea 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 contin-
ued from the bottom to top chords. This occurs in a single track
bridge of the Cleveland, Painesville and Eastern Railroad at Wil-
loughby, Ohio, built in 1896.

Being merely a special form of the Pratt truss, the Balti-
more truss has all its good qualities. The struts of the main truss
being perpendicular to the chords are the shortest; the main diago-
nal 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 ar-
ranged 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 re-
garded 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.
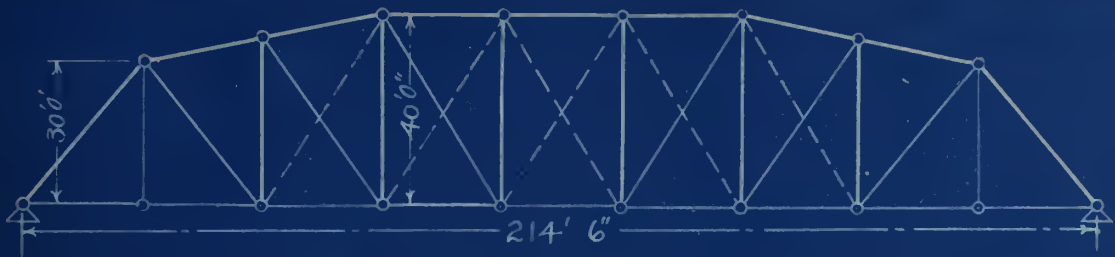
Fig. 66 A. T. & S. F. Ry Standard 214-Foot Bridge.
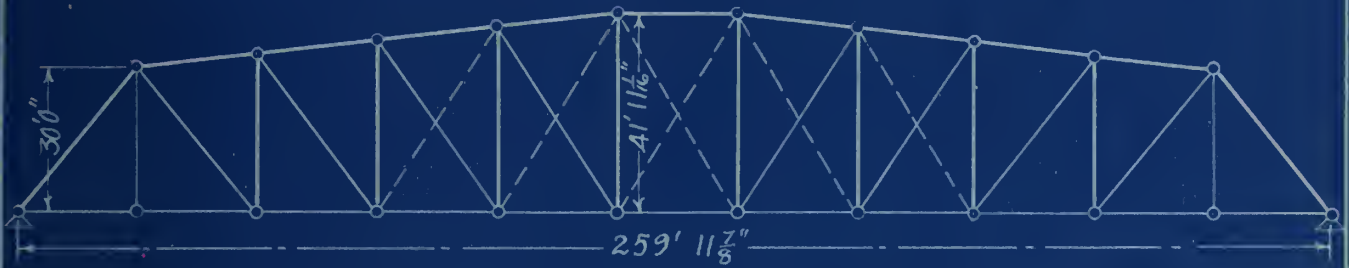
Fig. 67 A. T. & S. F. Ry Standard 360-Foot Bridge.
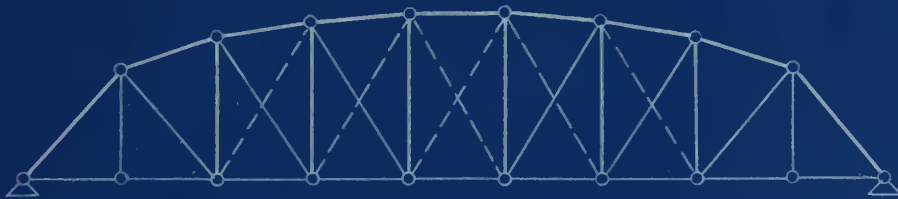
Fig. 68 Parker Truss.
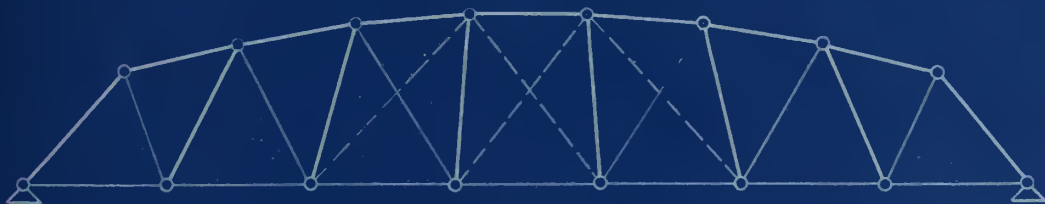
Fig. 69 Pegram Truss.
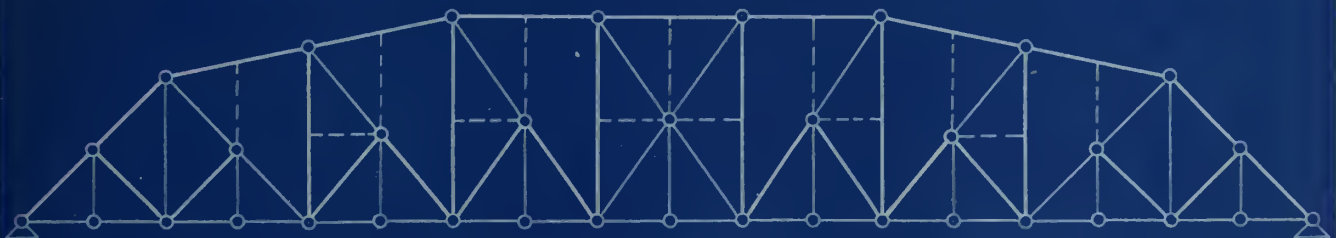
Fig. 70 Pettit Truss.

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 supports 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 di-
agonals, 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 col-
umns near their middle. (1) The sole function of the auxiliary mem-
bers 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 secon-
dary bending stresses. Thus, there is a tendency to build the Pet-
tit 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 re-distribution 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 beginn-
ing, 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, To-
peka 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 ef-
fect 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 lit-
tle 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, Tope-
ka 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 satis-
fies the condition for best economy with truss bridges. Taking the
question of clear head room and the grade of the approaches in con-
sideration, 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:--

Trusses	$4.63 per ton of which	984 tons are included,
Plate-girders	$5.49 " " " "	2784 " " "
I-beams	$2.88 " " " "	2837 " " "

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 mem-

bers. It was very heavy. The second pin-connected bridge in Eng-

land was the Crumlin bridge built in 1853, and the third was a rail-

way 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 flexi-

ble 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 dif-
ferent conditions; the riveted truss for short spans, and the pin-
connected truss for spans of 200 feet or more, as adopted by Ameri-
can engineers. (1) The strongest point in favor of the pin-connect-
ed truss is that it weighs less for the same specified loading when
the span is long. There is considerable saving in the tension mem-
bers 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 con-
tinuous and the rivet joints hold the members of the truss more sol-
idly; 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.
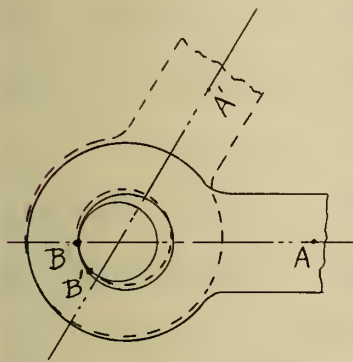
Fig. 71.

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 a-
gainst 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. For-
merly the Warren truss was also frequently built with pin-connect-
ions 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 ac-
cordingly must be massive and stiff. Stiffness is especially import-
ant for short span bridges as the structure is light and hence sen-
sitive 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 to-
tal 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 inva-
riably 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 im-
mediate support for shallow floors, but the use of double intersec-
tion 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 railway 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 re-
quired 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 contem-
plated 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 pan-
el needs to be short, but is controlled by the span length and eco-
nomic depth of the truss. In this case multiple webbing is intoler-
able, the sub-panel system becomes best. Thus, the Baltimore truss
became generally adopted for solid floors as mentioned in the fore-
going pages.
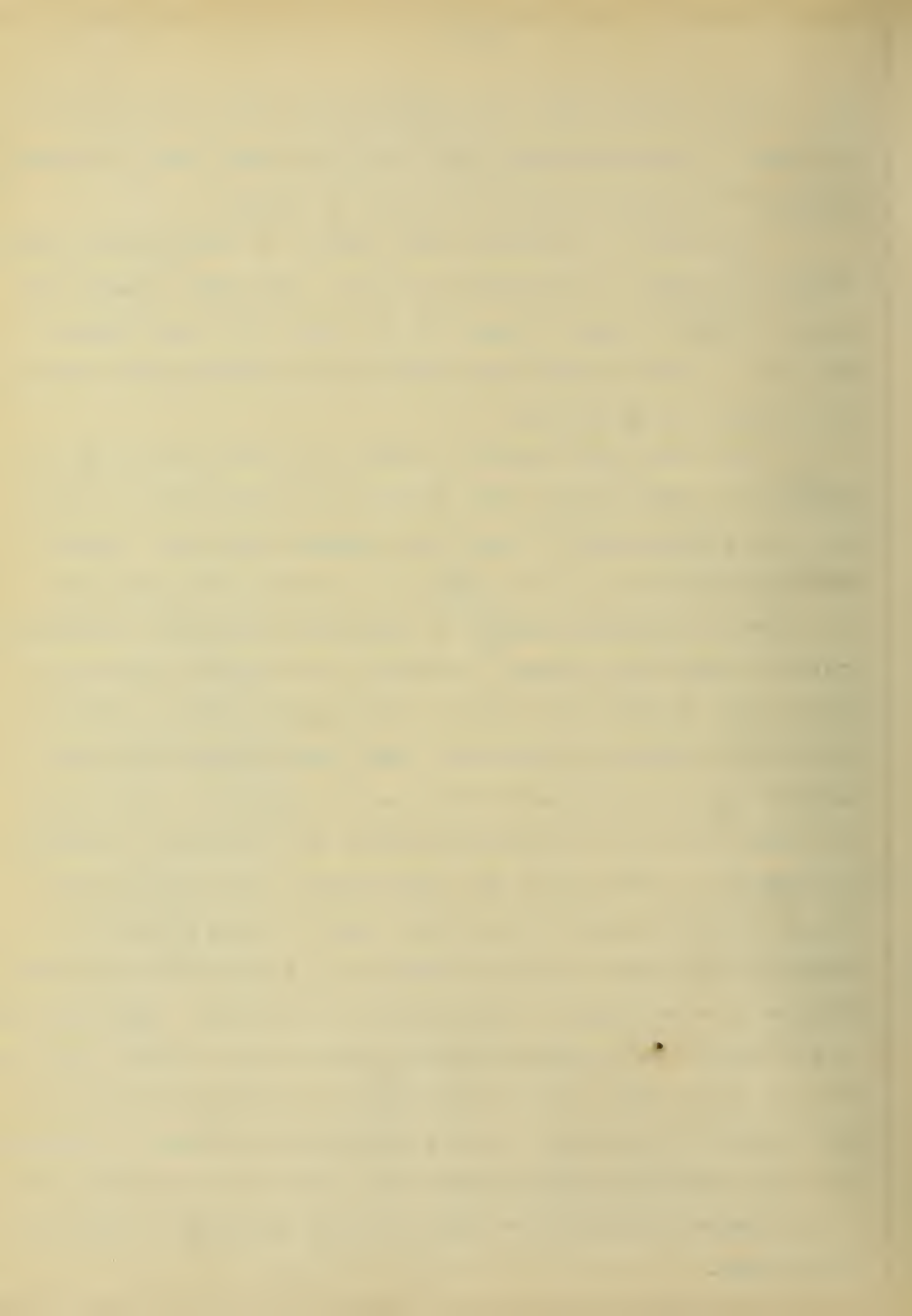
Art. 9.   Span Length.

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 occassionally 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.

TYPE OF STRUCTURE	Authority			
	Northern Pacific Railway	Atchison Topeka & Santa Fé Railway	J. A. L. Waddell	Proposed by Author
	Span in Feet			
T-Rail		Up to 8		Up to 8
I-Beam	10 to 30	8　26	Up to 20	8　26
Plate Girders { Deck	25　100	26　100	20　100	20　100
Plate Girders { Through	40　100	30　105½	20　100	30　100
Pratt (Four panel Special)			90　125	
Pratt Truss	130　200	110　210	125　250	100　200
Warren Truss { With Auxilliary members	110　120		125　250	
Warren Truss { Single Cancellation			125　200	
Warren Truss { Double　"			200　250	
Curved Top Chord		210　300		175　300
Baltimore Truss				200　300
Pettit Truss			250 & up	250　550

Waddell prefers the use of the Pettit truss for spans of 250 feet or more, but some authorties 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 errected 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, Kentuckey 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 stiffest
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 proport-
ional 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 stiff-
est 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 im-
parting 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 effecting 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 floor-
ing. 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 vibra-
tory.

The length of panel also has an influence on rigidity of the
bridge. Panel lengths about equal to the circumference of loco-
motive 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

Weight of Bridge in Thousand Pounds.

Span in Feet

Thru Pin Curved Top Chord

Thru Pin Pratt

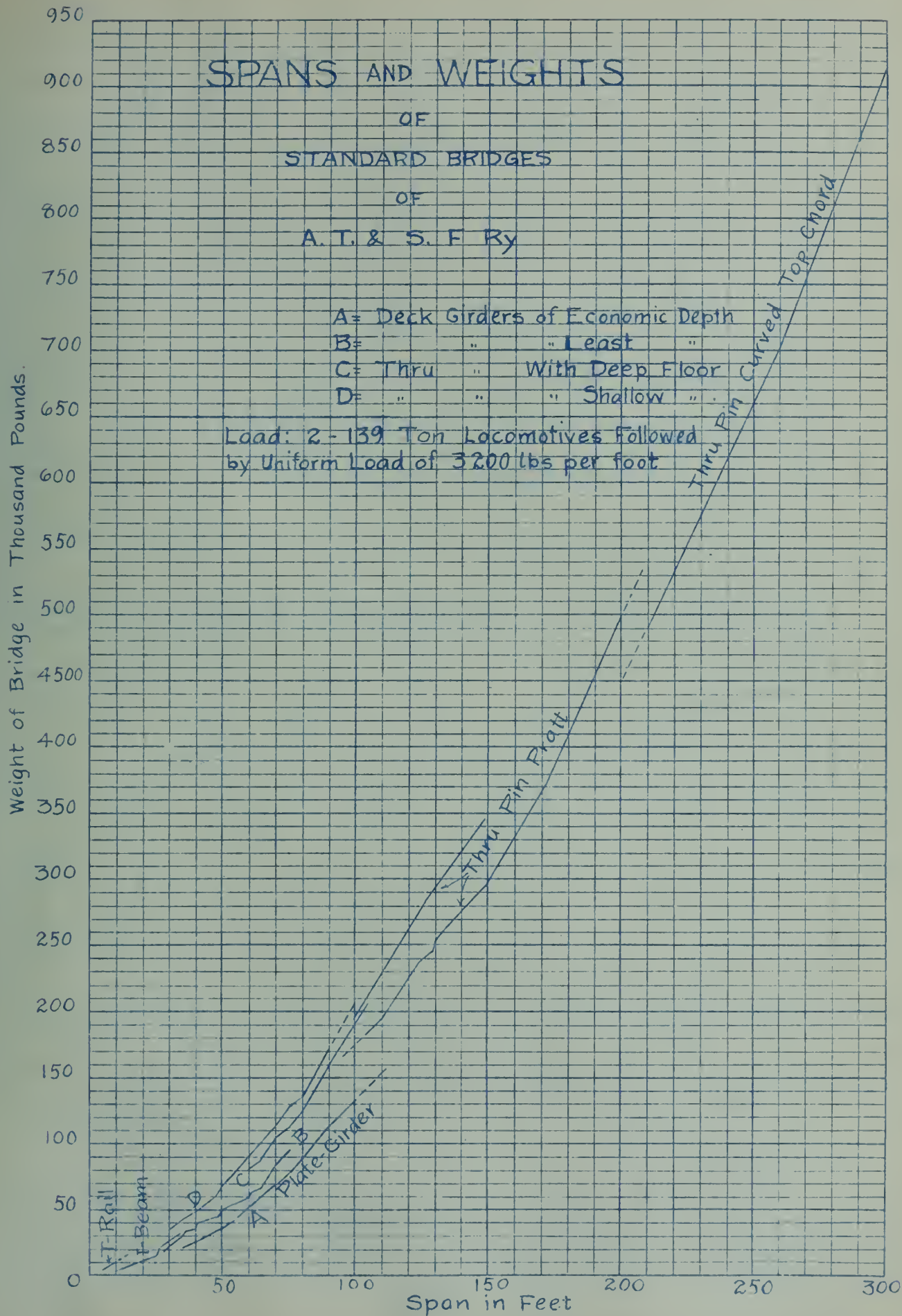Plate-Girder

T-Rail

I-Beam

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 differ-
ence 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: 2 146 Ton Locomotives
With Train Load of 4000 lbs per Foot

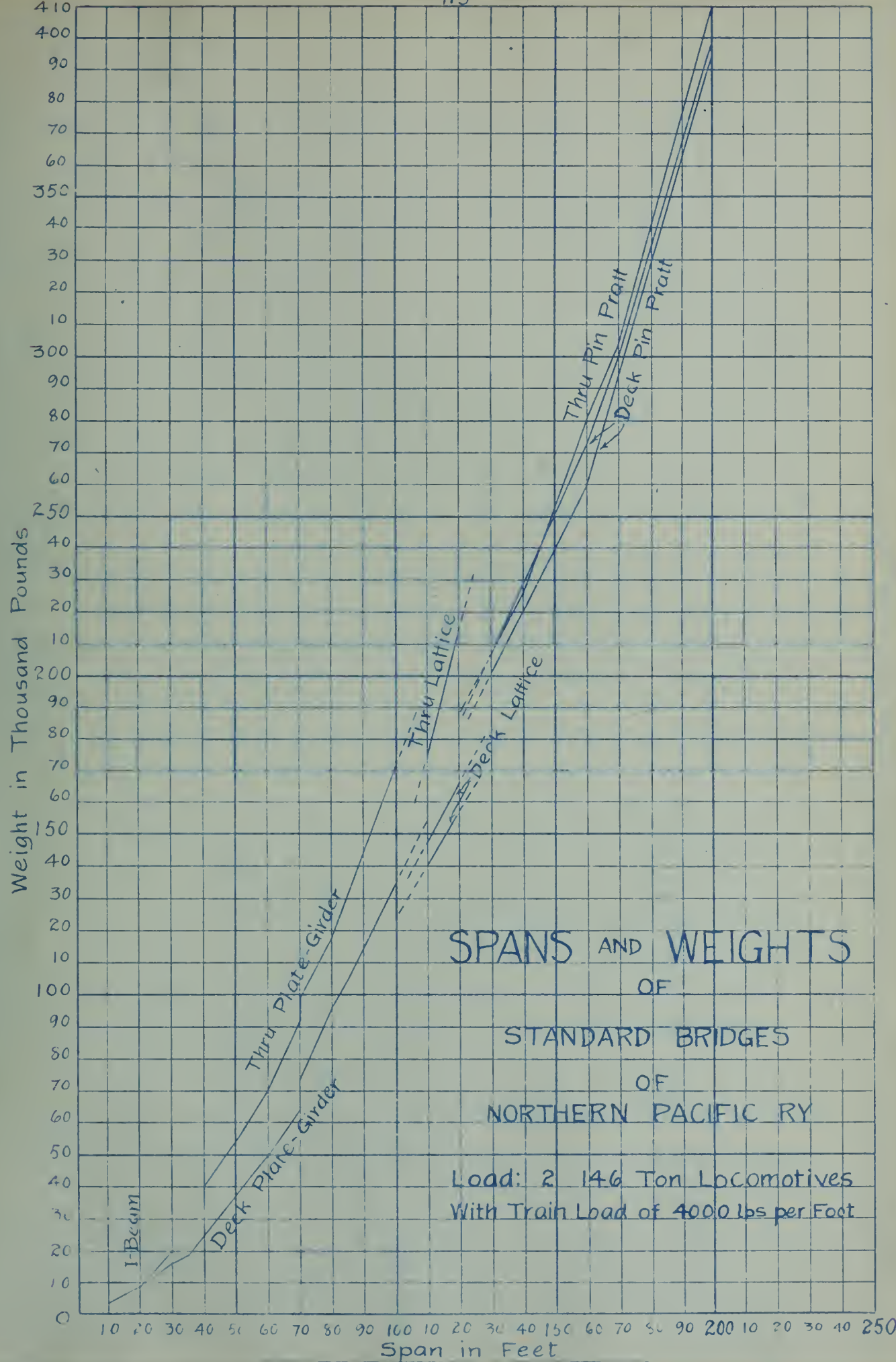Weight in Thousand Pounds

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 strickly 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 noth-
ing. 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, depend-
ing 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 "

     Pin-connected Pratt truss spans------4.5 "

     Pin-connected Pettit truss spans-----5.0 "

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.

Art. 12.   Ease of Maintenance.

     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 mainte-
nance. 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 dura-
bility 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 unexpect-
ed overloading. When the load to be carried would cause the stress-
es 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
Volume 30 page 277:-- :

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

Art. 13. Aesthetics.

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 ugle, 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.