The general design of the deck spans is the double intersection Pratt Truss.

The two systems being entirely independent of each other throughout (see Plate III). Where the diagonals cross the vertical posts, there is a pin running through the post, making the ties in two lengths. It is a matter of regret that this practice has been used indiscriminately by the Engineers in the States, without any regard as to its pernicious effects. This has been the case, for example, to such large structures as the Platsmouth bridge, in which there are posts 50 ft. long on centres, divided into half lengths by the ties crossing at their middle, without any provision for the effects of distortion due to strain in the members; the effect of a load coming on a structure framed in this manner can be easily shown (see Fig 1., Plate III). Supposing the trusses to be cambered when there is no strain in any member, the intersection of the tie is at some point below the centre of the post. Now, when the load comes on the span, the chords tend to become horizontal, the posts tend to become vertical, bringing the intersection of the ties with the posts more and more towards the centre of the post, until finally, when the entire camber is taken out of the truss, the intersection must necessarily be at the centre of the post; the amount of this movement depends on the length of the panels and the depth of the truss. In the 240 feet spans of the Lachine bridge, this movement amounts to about ¼". and has been provided for by making the holes in the posts 1" larger than the pin, thus allowing ample movement for the pin when the load comes on the bridge. This movement can be noticed in a structure at any time where the pin is free to move, as in the Lachine Bridge. Where the pin is not free to move the distortions must necessarily take place in the members themselves; and, moreover, I question this practice where it has been done with a view to figure the posts for half their total length, and consider them as fixed ended where they are held by the diagonals at the centre.

The next portion of the bridge to be considered are the two 269 ft. spans and two 408 ft. spans, forming four continuous spans over five supports (see Plate IV). There were two designs proposed besides the one that was finally adopted (see Plate V). The design as adopted is known in Mr. Smith's office as the "Flying Cantilever," and was first proposed for the Storm King bridge over the Hudson River, in State of New York. As used in the Lachine Bridge it is, properly speaking, no cantilever bridge, as the spans are continuous. The cantilever principle is used here for erecting the bridge only, which is built out from the piers on each side, the ends being joined at the centre when the final coupling is made, and the spans become continuous over five supports. The advantages of the cantilever principle are only in saving in the erection, there being no saving in the weight, as we merely make a different distribution of the material than we would in ordinary disconnected spans. In a continuous girder there is necessarily a saving in the weight over the piers, as was the case in the Lachine Bridge; but the saving in the mode of the erection is the principal item to be considered here. The advantage of using two centre piers instead of one would have been a considerable saving in the cost of erection, but not sufficient to counterbalance the increased cost of the extra masonry; this was the principal reason why one centre pier was only used instead of two, as shown in Fig 1, Plate V.

In speaking of cantilever bridges, it might be here stated that the first cantilever bridge built in America was the Kentucky River Bridge built by Mr. C. S. Smith in 1876. Mr. Smith also built the Minnehaha cantilever in 1881, long before the Niagara cantilever was ever thought of. The Kentucky River Bridge is a wonderful structure, from the fact that it is really the first continuous girder that was ever built in this country, and is remarkable also from the fact that instead of being continuous over four supports (See Plate VI.) it has its points of contra-flexure fixed by cutting the chords after the bridge was erected. In a letter of Mr. Smith's, written two years before the bridge was built, he says: "I feel so confident of my calculations of the continuous girder that I now propose to cut the chords at their points of contra-flexure, thus fixing these points beyond a question of doubt." This statement was the forerunner of the Kentucky River Bridge, in which the points of contra-flexure were fixed at. * (See Plate VI.) These points of contra-flexure could have been fixed in the river arms instead of the shore arms, and it is a curious fact that they should not have been fixed in the river arms, as was subsequently done by Mr. Smith in the Minnehaha cantilever, where the point of contra-flexure is fixed in the centre of the river span, there being two shores arms and two river arms without any mid span hung from the ends of the river arms, as in the Niagara and St. John cantilever bridges.

### THE CONTINUOUS GIRDER.

In any beam continuous over any number of supports, when any flexure takes place $\frac{1}{S} = \frac{M}{EI}$ in which:

M, is the bending moment at any point in the beam.

S, is the radius of curvature of the beam at that point.

E, is the modulus Elasticity of the material.

I, is the moment of inertia of the cross-section of the beam at that point.

By assuming all the supports to be level; and assuming "E" and "I" to be constant, the theorem of Three Moments may be obtained, and is given in all text books on Applied Mechanics. However, it was not until September, 1875, when Professor Mansfield Merriman gave, in the London Philosophical Magazine, the formulæ for a beam continuous over any number of level supports which are at all practicable. These formulæ are as follows:

Formulæ for obtaining pier moments and reactions, as applied to the four continuous spans in the Lachine Bridge.

—— n × l —— ← a → P —— l —— —— l —— —— n × l ——

A (1) Span No. 1, 268¼' A (2) Span No. 2, 408' A (3) Span No. 3, 408' A (4) Span No. 4, 268¼' A (5)

"m" is the number of any pier.

"r" is the number of any pier loaded span.

M is any pier moment.

S is the total number of spans = 4.

We have for Pier Moments when m < r + 1.

$$M_m = \binom{C_m}{l_r} \frac{A\, c_{s-r+2} + A^1 c_{s-r+1}}{C_{s-1} + 2(n+1) C_s}.$$

When $m > r$ we have,

$$M_m = \binom{C_{s-m+2}}{l_r} \frac{A\, c_r + A^1 c_{r+1}}{c_{s-1} + 2(n+1) c_s},$$

in which
$$A = P\, l_r \{2k - 3k^2 + k^3\}, \quad A^1 = P\, l_r \{k - k^3\}, \quad k = \frac{a}{l_r}.$$

P denoting the load in any span.

$l_r$ denoting the length of that span.

a = distance from nearest left hand support to the load "P," which is necessarily a concentrated load.

$$c_1 = 0;\ c_2 = 1;\ c_3 = -(2 + 2n) = -3{\cdot}3151.$$
$$c_4 = 1 + 3 + \{n \times (1 + 1)\} = + 12{\cdot}2616.$$

SHEARING FORCES.

$$S_r \text{ (in loaded span)} = \frac{M_r - M_{r+1}}{l_r} + q.$$

$$S_{r+1} \text{ (in loaded span)} = \frac{M_{r+1} - M_r}{l_r} + q^1.$$

$$S_m \text{ (in unloaded spans)} = \frac{M_m - M_{m+1}}{l_m}.$$

$$S^1_m \text{ (in unloaded spans)} = \frac{M_m - M_{m-1}}{l_{m-1}}.$$

in which $q = P(1 - k)$; $q^1 = P \times k$.

$S_r$ denotes the shearing force immediately to the right of the nearest left hand support, and $S_r + 1$ denotes the shearing force immediately to the left of the nearest right support of the loaded span.

$S_m$ and $S'_m$ apply to the unloaded spans in the same manner.

The above formulæ are given by Dubois in the "Strains in Framed Structures," page 135, but unfortunately the signs + and − should be reversed.

The principles of the design for the four continuous spans, upon which the calculations were based, are the strains from dead weight which are calculated as a cantilever each way from "W" (see Plate IV). After the dead weight is swung complete and proper adjustments are made by means of adjustible ties each way from "W," and adjustible beds at the ends of the balancing spans at "A"; the section "XY" on top chord is riveted in place when the four spans act as continuous as far as live load is concerned. The calculations for live load strains were then made in accordance with the formulæ before given for a girder continuous over level supports, and the two were combined.

The objections to any continuous girder are: 1st—the modulus of elasticity "E" is not constant; 2nd—the moments of inertia "I" is not constant; 3rd—the supports are not necessarily level. These objections will be discussed in order.

1st.—The modulus of elasticity, as is well known, has wide margins of variation in the same material, but by rigid inspection of the material at the mill this variation may be reduced to a minimum. Mr. Bouscaren gives the margin for variation of the modulus of elasticity of iron at per cent. For mild steel, which is a far more homogeneous metal