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OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

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INSTITUTED 1852.

## TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XIV.—November, 1885.)

### THE CANTILEVER BRIDGE AT NIAGARA FALLS.

By CHARLES C. SCHNEIDER, M. Am. Soc. C. E.

READ MARCH 4TH, 1885.

WITH DISCUSSION.

The interest which has been taken by the profession in the construction of the Niagara Cantilever Bridge has led the writer to prepare this paper, in which he has endeavored to give a faithful account of the construction, and a full description of the work in all its details, and which is most respectfully submitted to the Members of this Society.

#### PRELIMINARY NARRATIVE.

On October 13th, 1882, the writer received a letter from the Central Bridge Works, of Buffalo, N. Y., requesting him to make a preliminary estimate for a double-track railroad bridge of 900 feet clear span, for the purpose of ascertaining the probable cost of bridging the Niagara River below the Falls, near the Railroad Suspension Bridge, intimating that a braced arch reaching from cliff to cliff might be the proper design for the proposed structure.

Not being in possession of a profile of the bridge site, the writer was unable to decide upon a plan; but, from the nature of the stream, concluded that a structure in that locality should be so designed that the river span would be self-sustaining during erection. Having only a slight recollection of the formation of the gorge, he thought that a hinged arch, to be erected on the cantilever principle, would be the proper design, and made a rough sketch of one of 600 feet clear span, assuming a profile, but requesting that an approximate profile should be made to assist in deciding on a feasible plan by which the chasm could be spanned without the use of false works for the river span.

The writer having previously, in the spring of 1882, designed the Fraser River Bridge for the Canadian Pacific Railway (see Plate XLIX), where similar conditions existed, then had occasion to investigate that class of structures which are best adapted for bridging streams the nature of which precludes the use of false works for erection, and to make himself thoroughly familiar with the subject.

According to the writer's request, the Central Bridge Works had an approximate profile made of the site of the proposed bridge, which was received by him on October 23d. After careful consideration, the writer decided that the cantilever plan would be the most feasible and economical for this particular location, sketched out a skeleton design, and made an approximate estimate for the same, which he submitted to the Central Bridge Works, with the recommendation that steel be used for the entire structure, with the exception of the floor beams and stringers, which should be made of wrought iron. An exact fac-simile of this first plan is shown in Plate L.

The general dimensions and some of the detail arrangements of this plan were subsequently changed to suit the final location and other conditions, but the general features of the plan have been substantially carried out.

This design was afterwards worked out more in detail; strain-sheets were made, the estimate revised, and a tender submitted by the Central Bridge Works to the Niagara River Bridge Company for the construction of the entire work.

This tender was considered by the Board of Directors of the Niagara River Bridge Company, and found satisfactory; the preliminary designs accompanying the bid were referred to Mr. Charles H. Fisher, M. Am. Soc. C. E., Chief Engineer of the New York Central and Hudson River

*Frazer River Bridge. Canadian Pacific Railway.*

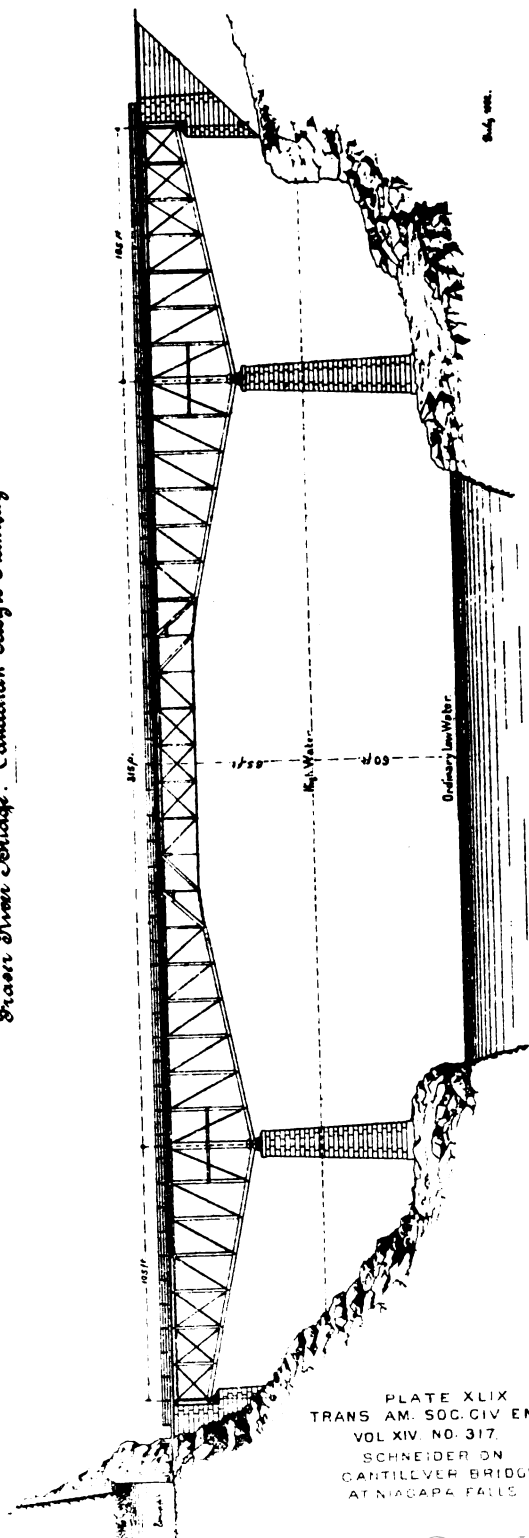
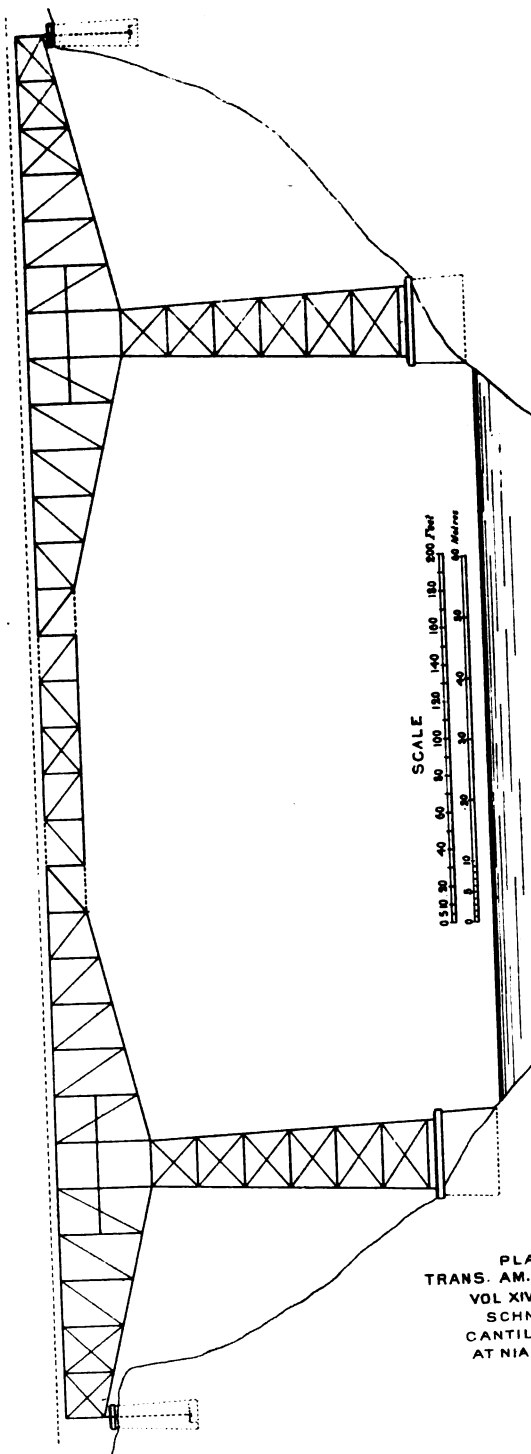


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PRELIMINARY DESIGN.

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Railroad, who acted at that time as consulting engineer for the bridge company; he considered the plan feasible, and approved of it. The contract for the whole work was accordingly awarded to the Central Bridge Works, of Buffalo, on April 11th, 1883, on condition that the structure be completed on December 1st of the same year, and be built according to specifications which had been agreed on between the Niagara River Bridge Company, under advice of their consulting engineer, and the contractors.

These specifications are given in Appendix No. 1.

The contracts for the following portions of the work were sublet by the Central Bridge Works:

Excavation and masonry to Dawson, Simmes & Mitchell, Niagara Falls, Ont.

Beton foundations to John C. Goodridge, Jr., New York City.

Steel and iron compression members for towers to Kellogg & Maurice, Athens, Pa.

On April 26th the writer had an interview with Mr. Cornelius Vanderbilt, President, and Mr. James Tillinghast, Vice-President of the Niagara River Bridge Company, when he was informed of his appointment as chief engineer, and requested to take charge of the work immediately.

The writer appointed Mr. A. R. Trew as his principal assistant, and subsequently Mr. James A. Bell and Mr. B. F. Betts were employed as assistant engineers.

An accurate survey and profile of the bridge site had been made previously by Messrs. G. C. Cunningham and James A. Bell. The writer commenced at once to perfect the plans and work out the details of the structure, with the kind assistance of the Engineer of the Central Bridge Works.

In making the final plans it was found necessary to make some changes from the original design. The length of the cantilever arms, as well as that of the intermediate span, had to be changed to suit the final location of the piers. The end-posts on the shore ends of the cantilevers were made inclined, similar to the river ends, for the purpose of bringing the anchorage masonry above ground, thus saving the labor of rock cutting, besides having better access to the anchorage. As time was a very important factor in this work, and having been convinced by previous experience of the uncertainty of obtaining steel

of satisfactory quality in so short a time as was needed for this work, the writer concluded to limit the use of steel to pins and heavy compression members, such as tower-posts, center-posts, compression-chords and end-posts of the cantilevers. It was also decided to use a double system of diagonals for the cantilevers, although the writer does not ordinarily advocate double intersection. It was done, however, in this case, to have an intermediate support for the posts, some of which are very long and had to be made in two sections; and also, with reference to convenience of manufacture, to shorten the eye-bars for the main ties.

The general plan of the structure as it was built is given on Plate No. LI.

Work was commenced at the bridge site on April 15th, and the material for the superstructure was ordered by the contractors as soon as the plans were far enough advanced; work at the bridge site as well as in the shops was pushed along vigorously under the very able management of the Central Bridge Works, until the bridge was completed and opened for traffic on December 20th, 1883, about eight months from the commencement of the work.

#### LOCATION AND SURROUNDINGS.

The bridge which is the subject of this paper crosses the Niagara River about two miles below the Falls and about 300 feet above the Railroad Suspension Bridge, and connects the village of Suspension Bridge, New York, and the town of Niagara Falls, Ontario.

Previous to the building of this bridge, the Niagara was spanned by two railroad bridges, viz.: the Railroad Suspension Bridge, near which the new bridge is located, and the International Bridge, crossing the Niagara at Black Rock, one of the suburbs of Buffalo; the latter was formerly used by the Michigan Central Railroad, and as both bridges were controlled by the Grand Trunk Railway, it was deemed desirable for the interests of the Michigan Central Railroad to have an independent crossing, for which the site of the new bridge was selected.

The bridge connects on the American side by a short approach with the tracks of the Niagara Falls branch of the New York Central and Hudson River Railroad. On the Canadian side it was necessary to build a branch line, known as the Welland Cut-off, which intersects the Niagara Falls branch of the Michigan Central Railroad at the "View,"



S

NIAGARAN SIDE

PLATE LI.  
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posite the Falls, and joins the main line of the Michigan Central Railroad at Welland, Ont., having a total length of 14 miles.

The Niagara River forms the outlet of Lake Erie into Lake Ontario; it is 36 miles long and forms a part of the boundary line between the United States and Canada. It contains many islands, of which Grand Island is the largest, and at about 22 miles from its source occurs the great cataract of Niagara Falls.

The strata from the base of the escarpment at Lewiston (about 7 miles from Lake Ontario) up to the Falls are represented by eight divisions, of which the bottom is a red shaly sandstone and marl which forms the bank at Lewiston and extends to Lake Ontario. Above this is a gray quartzose-sandstone; this layer forms the bed of the lower rapids, and the river cuts through this layer at the whirlpool. The third layer is a red shaly sandstone, which, with the one below it, forms the Medina sandstone group, which shows at the river bank just below the Suspension Bridge. Above this is a thin layer of green shale; we then come to a compact gray limestone, then to a soft argillo-calcareous shale, which is known as the Niagara shale; above this is a compact limestone, known as the Niagara limestone, which is the stratum on which the anchorage piers of the bridge rest.

The bridge spans a chasm of about 850 feet in width between the bluffs and 210 feet depth to the surface of the water. The banks on both sides of the river have a slope of about 45 degrees from the water's edge to about 50 feet below the top of the cliff, above which they are vertical. The river is about 425 feet wide at the bridge site and the water has a velocity of about 16½ miles per hour in the center of the river, forming whirlpools and eddies along the American shore. On account of the rapid current of the stream it has been impossible to ascertain the correct depth of the river, but it is estimated to be 50 to 80 feet in the center.

The sloping banks of the river at the bridge site consist of a mass of large boulders and broken rocks from the hard limestone layer which forms the upper stratum, mixed with earth and *debris*. The limestone has been undermined by the action of the water, which has cut away the argillaceous stones, leaving the limestone overhanging; this stone falling into the gorge in large masses has formed a natural rip-rap from the ferry below the Falls to the Suspension Bridge, and has prevented further erosion of the softer stones. Below the whirlpool there are no hard rocks in the bed of the river.

In making the excavations of the pits near the water's edge it was found that the material removed consisted wholly of the strong limestone above mentioned. The indications are that no bed rock could have been found for the foundations of the piers which support the towers, unless the excavations had been extended to the bottom of the river. If the bridge had been founded on the shale of Medina sandstone, it would have been necessary to have rip-rapped it with immense rocks, as nature has already done at the bridge site and up to the Falls. The erosion of the softer stones and the rip-rapping of the banks by the falling of the large stones can now be seen on the east bank of the river below the Suspension Bridge; the river cutting into the ledge until the rock of the limestone group breaks and falling forms a rip-rap which protects it from further wear. Just below the site of the foundation on the east bank, fallen limestones have made a raceway, which has been a site for a mill, and has remained in its present condition unchanged from the time of the oldest records.

There is a sufficient accumulation of earth on the broken rocks to support vegetation, and from the Suspension Bridge to the Falls the banks are heavily wooded to the water's edge with trees, many of them of considerable size, showing that this bank has been undisturbed for a long period.

#### SUBSTRUCTURE.

Work on the foundations was commenced on April 15th, 1883, by excavating the pits for the river piers which were to form the supports for the towers. This work mainly consisted in excavating and removing loose rock and *debris*. A great deal of blasting had to be done, as most of the pieces of rock forming the banks of the river were of such size that they could not be handled whole, and many of them extended far into the banks. The materials excavated were deposited into the river in front of the pits; much of it was washed away again by the current of the water, only the largest pieces remaining. This work was done simultaneously on both sides of the river. It was at first supposed that somewhere near the water's level a ledge of solid rock might be struck, and the intention was to place the masonry directly on the rock on foundations prepared by making a smooth, level surface on the same. As the work of excavating progressed it became more and more apparent that the formation which was found in the sloping banks was likely to extend some distance below the water's level, and as the work

of excavating into the banks was attended with great difficulties and dangers, it was decided to excavate only deep enough to reach a good compact layer of large rocks and far enough back into the banks to form pits large enough to hold the masonry. At about the water's level on each side of the river, a good bed of irregular-shaped, but very large, rocks, was found, closely bonded and interlocked with each other, which the writer considered an excellent foundation for the weight which would be thrown upon it by the structure and masonry.

At the suggestion of the contractors it was decided to leave the irregular shape of the bottoms of the pits and prepare a bed for the masonry by filling these pits to an average depth of about 8 feet with "Beton Coignet," made by John C. Goodridge, Jr., of New York City. The foundations were prepared by removing all the small pieces of rock and debris and carefully cleaning out the interstices and cavities, but leaving the irregular surfaces as found. The beton was prepared on top of the cliffs and let down into the pits in chutes; it was well rammed into the crevices, thus fitting all irregularities in the sides as well as on the bottom and bonding all the rocks into one solid monolithic mass. This method will distribute the pressure uniformly over the entire area of the foundations and transmit a portion of it to the adjoining rocks, thus increasing the actual bearing surface of the footings to a large extent. The size of the foundation for each pair of piers, although of irregular shape, averages about 40 feet in length and 25 feet in width, making an area of about 1 000 square feet.

The weight thrown upon one pair of piers by the superstructure, including live load and wind pressure. is.....	2 348 000 pounds.
Weight of masonry. ....	2 800 000 "
" " beton.....	782 000 "

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Total weight on one foundation.....5 930 000 pounds, which makes 5 930 pounds per square foot, or about 41 pounds per square inch; this it will be seen is within the safe limits. As the actual area of the foundations, owing to the method above described, is practically a good deal larger than the one assumed in the calculations, the pressure per square foot is still further reduced.

The beton beds were leveled off at an elevation of 11 feet above the water level to receive the masonry; they average about 8 feet in thickness.

After the beton foundations were nearly completed and work on the masonry on the American side had commenced, a question came up regarding the stability of the foundations.

As the bridge was to be used by the Michigan Central Railroad, Mr. H. B. Ledyard, the President of that road, requested the late Mr. E. H. Phelps, Chief Engineer of the Michigan Central Railroad, to make an examination of the foundations. Mr. Phelps after making an examination reported that he was not perfectly certain as to the stability of said foundations, being of the opinion that the structure could be absolutely safe only on solid rock.

Mr. Cornelius Vanderbilt, President of the Niagara River Bridge Company, then sought the opinion of Mr. Charles H. Fisher, M. Am. Soc. C. E., Chief Engineer of the New York Central and Hudson River Railroad. Mr. Fisher also made an examination and partly supported Mr. Phelps' views, and suggested that, as he did not wish to rely on his own judgment solely in a case of such importance, the opinions of other engineers of acknowledged authority should be obtained.

In accordance with Mr. Fisher's suggestion Mr. J. Tillinghast, Vice-President of the Niagara River Bridge Company, selected Mr. George S. Morison, M. Am. Soc. C. E., and Mr. Charles Macdonald, M. Am. Soc. C. E., of New York City, as experts, who visited the bridge site and made an examination on July 6th. They gave their opinion in a written report, and expressed themselves as satisfied with the stability of the foundations.

A few days afterwards, Mr. John A. Wilson, M. Am. Soc. C. E., of Philadelphia, Mr. A. W. Stedman, Chief Engineer of the Lehigh Valley Railroad, and Mr. Theodore Cooper, M. Am. Soc. C. E., of New York City, who were at the time in Buffalo on other business, were also requested to make an examination of the foundations and give their opinion. They made their examination on July 10th and reported in favor of the foundations.

The official reports of both commissions of engineers are given in Appendix No. 2.

The masonry was built of Queenston limestone laid in cement in courses of about 2 feet rise, excepting the filling and backing, for which Black Rock stone was used.

The piers which support the towers are 12 feet square under the coping and have a batter of  $\frac{1}{4}$  inch to the foot; each pair of piers is connected

by a wall 4 feet thick on top and battering the same as the piers. These piers, together with the beton foundations, are shown on Plate LII.

The drawing also illustrates the underlying and surrounding rock formation.

The masonry for these piers was laid with a derrick placed at the end of a platform, 180 feet above the water's level, on top of a temporary trestle, which trestle was used afterwards for the erection of the towers and shore arms of the cantilevers. The stones were lowered from the cliff to the platform by a derrick erected above, and from there were run out on hand cars on a track on top of the trestle to the other derrick at the end of the platform, which was used for laying the stones. These derricks had 50-foot booms and were each provided with a hoisting engine working a single wire rope.

Work on the foundations was commenced on April 15th, and the beton filling on the American side was commenced on June 6th, and on the Canadian side on June 13th. The beton foundations were completed on June 26th on the American side and on July 13th on the Canadian side. Laying stone was commenced on the American side on June 26th and the piers finished on August 20th; on the Canadian side the masonry was commenced on July 14th and completed on September 3d.

The following table gives the quantities of beton and masonry, in cubic yards, used in the piers which support the towers.

	BETON.	MASONRY.
American side, north pier.....	226.8 cub. yds.	696.4 cub. yds.
"    "    south pier.....	291.4 " "	687.9 " "
Canadian side, north pier.....	257.0 " "	696.9 " "
"    "    south pier.....	378.7 " "	706.6 " "
Totals.....	1 153.9 cub. yds.	2 787.8 cub. yds.

The anchorage piers are located on top of the cliffs and are 11 feet x 37 feet 6 inches under the coping, with a batter of  $\frac{1}{4}$  inch to the foot. They form the supports for the shore ends and also act as counter-weights for the cantilevers. In order to utilize the whole weight of the masonry for the anchorage, they are built on a platform consisting of twelve wrought-iron plate girders 2 feet 6 inches deep and 38 feet long; these plate girders

on eighteen 15-inch I beams, through which the anchor bars pass. The lower ends of these bars have screw ends and are provided with nuts which screw up against a wrought-iron washer plate; the upper ends of the anchor bars are formed into loops which connect on the pin around which the rockers oscillate. It is evident that with this arrangement the weight of the whole mass of masonry in the anchorage piers is brought into account.

Each anchorage pier contains on an average 454 cubic yards of masonry, weighing, together with the iron-work, about 2 000 000 pounds. As the uplifting force from the cantilever under the most unfavorable position of loading is 612 000 pounds, we have a factor of safety of about  $3\frac{1}{2}$  in the weight of the anchorage against the overturning of the cantilevers.

The foundations for the anchorage piers were prepared by excavating the rock to such depth as to give the piers the required height; the pits were then partly filled up with masonry to form a bed for the I beams and plate girders, leaving cavities under the washer plates which open on the outside to gain access to the nuts on the anchor bars. The masonry for the anchorage piers was of the same class as that of the tower piers.

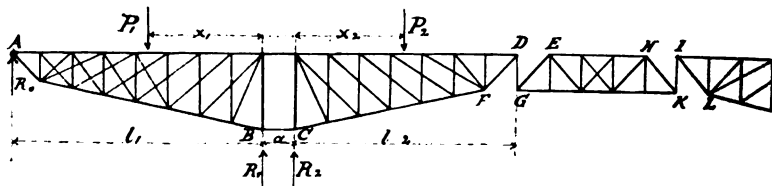
The details of one of the anchorage piers are shown on Plates LIII and LVIII.

#### SUPERSTRUCTURE.

The structure carries a double track; it consists of two cantilevers which rest on iron towers, and whose shore ends are anchored to masonry piers, the river ends being connected by an intermediate span.

The principal dimensions of the structure are as follows: Distance between centers of anchorage piers, 910 feet 2 $\frac{1}{2}$  inches; length of each of the cantilevers, 395 feet 2 $\frac{1}{4}$  inches between centers of end-pins; and length of the intermediate span, 119 feet 9 $\frac{3}{4}$  inches between centers of end-pins.

In order to define the principles of the system upon which the calculations were based, we will consider the structure as shown in the accompanying diagram, with the members *DE*, *FG*, *HI* and *KL* removed:



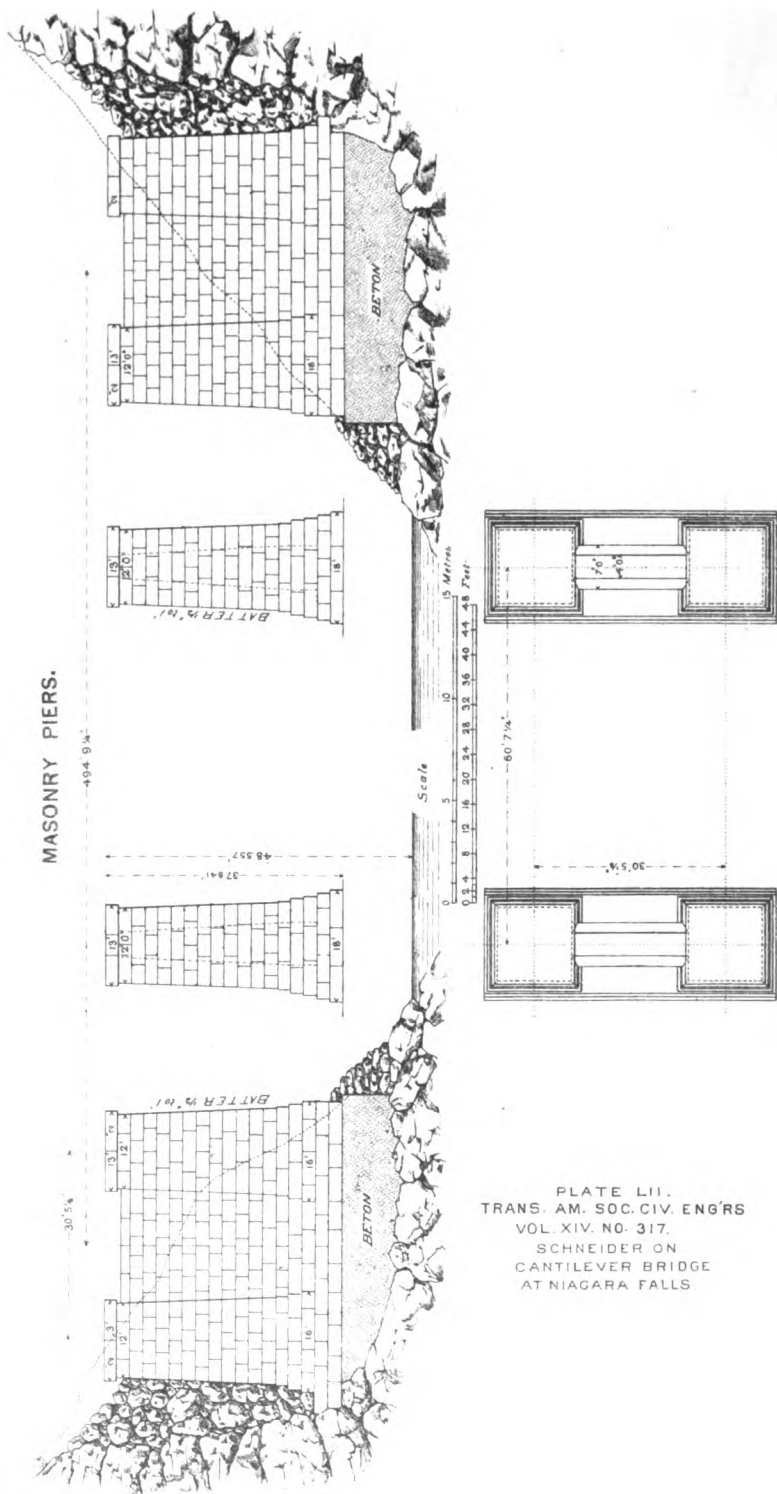


PLATE LII.  
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It is evident that the middle span  $GK$  is similar to a girders supported at both ends and free to move horizontally. In the finished structure the members  $DE$ ,  $FG$ ,  $HI$  and  $KL$  are put in, but have oblong pin-holes at the points  $E$ ,  $G$ ,  $H$  and  $K$ , so that a limited longitudinal motion can take place at these points. Those members are necessary to transfer the wind strains from the center span to the cantilevers. This arrangement eliminates the temperature strains, and makes it possible to determine the strains in the middle span with the same accuracy as in any ordinary truss.

The cantilevers being supported on three points,  $A$ ,  $B$  and  $C$ , would, if the web systems of the trusses were continuous from end to end, form a system in which the strains are ambiguous (beam continuous over three supports). However, by omitting the diagonals in panel  $BC$ , it is evident that no other strains can be transmitted between  $B$  and  $C$  than moments, the points of support being practically reduced to 2, and the shearing strain in panel  $BC$  becoming 0, the strains become well defined and can be determined with precision. If  $P_1$  denotes the resultant of the loads in the left cantilever arm and  $P_2$  the resultant of the loads acting upon the right cantilever arm,  $x_1$  and  $x_2$  their respective distances from the supports  $B$  and  $C$ , and  $R_0$ ,  $R_1$  and  $R_2$  the reactions at points  $A$ ,  $B$  and  $C$ , we have the following simple equations for the reactions :

$$R_0 + R_1 + R_2 - P_1 - P_2 = 0 \dots\dots\dots (I)$$

$$P_1 (l_1 - x_1) + P_2 (l_1 + a + x_2) - R_1 l_1 - R_2 (a + l_1) = 0 \dots\dots\dots (II)$$

$$P_2 - R_2 = 0 \dots\dots\dots (III)$$

$$\text{Hence: } R_0 = \frac{P_1 x_1 - P_2 x_2}{l_1} \dots\dots\dots (IV)$$

$$R_1 = \frac{P_1 (l_1 - x_1) + P_2 x_2}{l_1} \dots\dots\dots (V)$$

$$R_2 = P_2 \dots\dots\dots (VI)$$

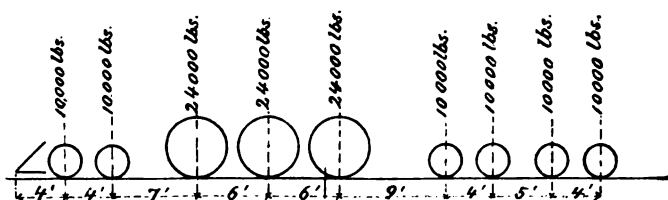
If the assumption is correct, that there is no shearing strain in the panel over the towers, the moments in section  $BC$  must be in equilibrium, or the following condition must be fulfilled:  $R_0 l_1 - P_1 x_1 = P_2 x_2$ . By substituting the value of  $R_0$  from equation IV, we obtain  $P_2 x_2 = P_1 x_1$ , thus proving that no condition of loading can produce a shearing strain on panel  $BC$ . The equations also show that if  $P_2 x_2$  is larger than  $P_1 x_1$ , the reaction at  $A$  becomes negative; the cantilever therefore must be anchored down at  $A$ .

After the reactions at  $A$ ,  $B$  and  $C$  are determined, the calculations of strains in the various members composing the cantilevers offer no difficulties.

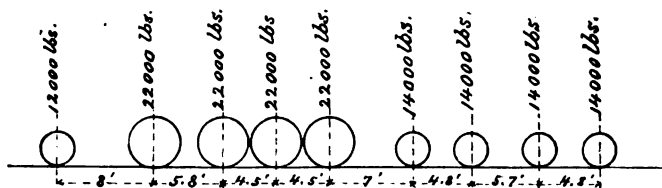
In order to eliminate the temperature strains in the shore arms of the cantilevers, as well as their effect upon the towers, the supports at  $A$  must be movable horizontally. To accomplish this, the shore ends of the cantilevers are supported on rockers, consisting of short links, capable of resisting tension as well as compression, hinging on pins which rest on pedestals, and are anchored to the masonry.

In proportioning the structure, the following moving load was assumed, as per specification agreed upon between the contractors and the Bridge Company, viz.: A train on each track, headed by two 66-ton locomotives, having 72 000 pounds on three pair of drivers, spaced 6 feet between centers, followed by a train load of 2 000 pounds per lineal

foot. The following is a diagram of the locomotives assumed for the calculations :



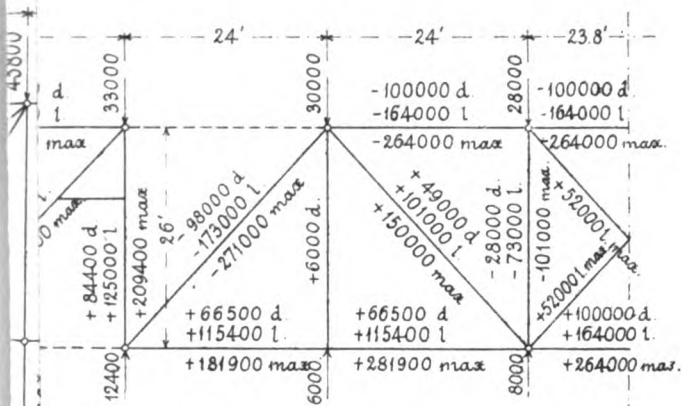
The floor system, however, has been proportioned for 78-ton consolidation engines, which is in excess of the loads called for in the specifications. This was done in view of the possibility that at some future time consolidation engines might be used on the bridge, which would in practice and under ordinary traffic not increase the strains on the members of the structure above those the bridge has been calculated for, but would increase the strains on the floor system materially. The diagram of the engines assumed for the calculation of the floor system is given below :



The lateral system is proportioned to resist a wind pressure of 30 pounds per square foot on a train surface of 10 feet vertical height, plus the same pressure per square foot upon the exposed surface of both trusses and the floor system ; the wind pressure on the train surface being considered as a moving load.

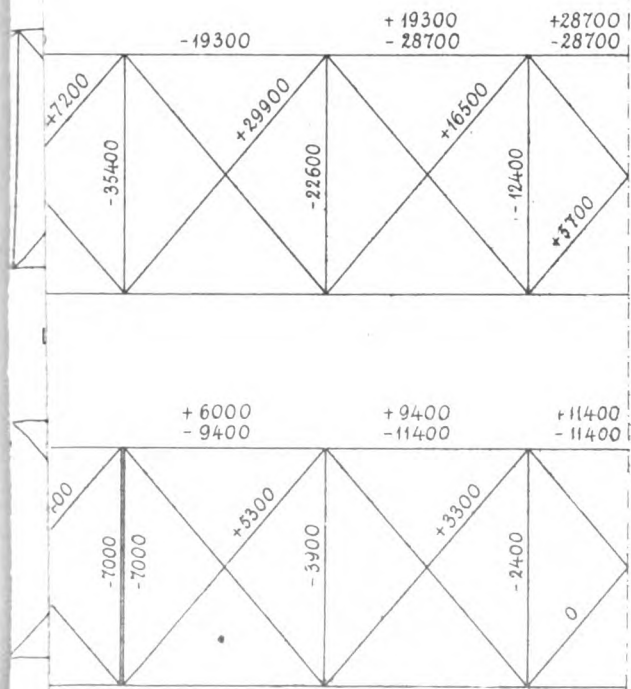
The strains produced by the above-mentioned loads and the strains due to the dead load of the structure, as computed from the actual shop weights applied at the several panel points, are shown in the strain-sheet on Plates LIV and LV ; the dead-load strains being marked *d*, the live load strains *l*, the wind strains *w*, + denoting tension and — compression. The maximum strain produced by this load was intended to be limited to  $11\,000 \left( 1 + \frac{1}{3} \frac{\text{minimum strain}}{\text{maximum strain}} \right)$  pounds per square inch on the section

of the steel compression members, and to  $8\,000 \left( 1 + \frac{1}{3} \frac{\text{min. strain}}{\text{max. strain}} \right)$  pounds on the iron members in tension. The above formulae are modifications of Weyrauch's formula derived from Woehler's experiments. The actual weights of the finished structure, however, turned out to be a little different from those assumed in the first calculations, and as the dead load is an important factor in a structure of this kind, the strains per square inch of section on the different members were thus in some cases increased, in others decreased. The bending strain on the steel pins was limited to 20 000 pounds per square inch on the extreme fiber, the strains on each bar being supposed to be concentrated on its center. The maximum tension allowed on the lateral rods was 15 000 pounds per square inch.



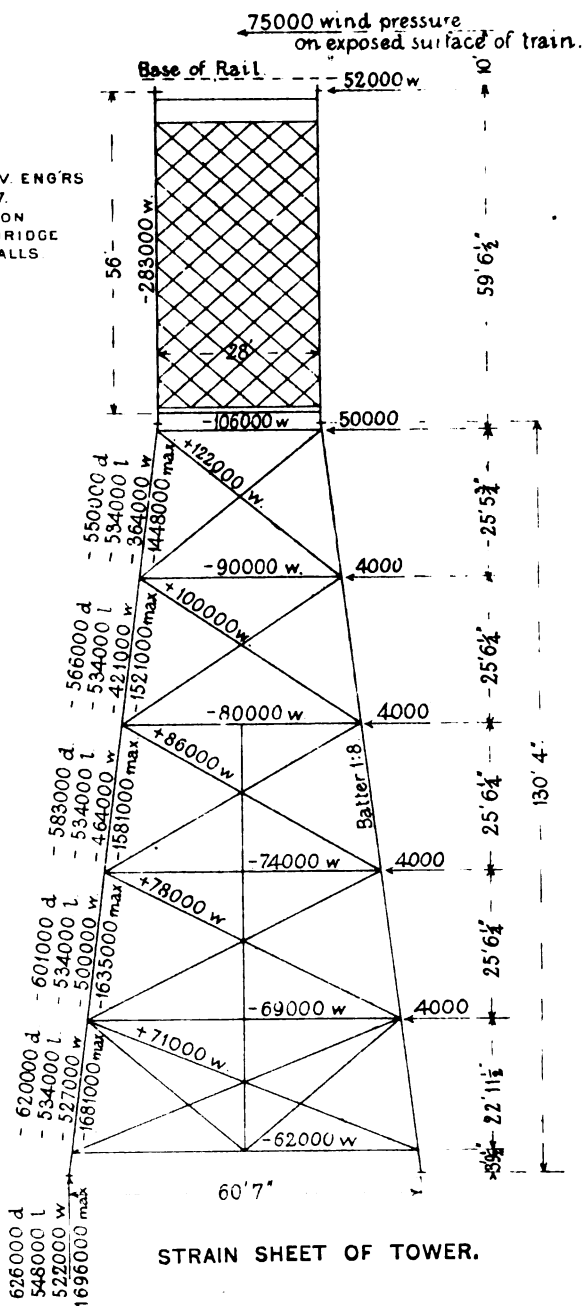
+ denotes tension.  
 - " compression.  
 trains resulting from dead load  
 " " " " " live "

Center line of bridge.



Strains in pounds  
 + denotes tension.  
 - " compression.  
 d - Strains resulting from dead load.  
 l - " " " " " live  
 w - " " " " " wind pressure.

PLATE LV  
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 AT NIAGARA FALLS



The tower-posts, lower chords, center and end-posts of cantilevers, pins and top-castings for towers are of steel; all the other parts of the bridge are of wrought-iron, excepting the shoes for the tower-posts, filling rings, washers and hand-rail posts, which are of cast-iron.

The towers are 132 feet  $6\frac{1}{2}$  inches high from top of masonry to center of lower chord of cantilevers. Each tower is composed of four main posts, which are connected to each other by horizontal struts, and braced diagonally by tie-rods. These posts rest on cast-iron shoes at the bottom, which are supported by the masonry piers. The tops of the posts consist of steel castings, which support the cantilevers on  $7\frac{1}{4}$ -inch steel pins. The tower-posts have a batter of 1 in 8 at right angles to the axis of the bridge, and a batter of 1 in 48 parallel to the axis of the bridge. The distance between centers of posts at their base is 60 feet  $7\frac{1}{4}$  inches at right angles to the line of the bridge, and 30 feet  $5\frac{1}{4}$  inches parallel to it. On top, where the posts form the support for the cantilevers, the distance between their centers is 28 feet at right angles to the line of the bridge, and 25 feet parallel to it. The tower-posts are formed of steel plates and angles; each post consisting of two side plates 26 inches wide, varying in thickness from  $\frac{1}{4}$  inch to  $\frac{3}{4}$  inch, 4 angles  $4'' \times 4'' \times \frac{3}{8}''$ , a cover plate  $30'' \times \frac{1}{4}''$ , two filling plates between the angles of  $18'' \times \frac{3}{8}''$  and two balance plates  $5'' \times \frac{1}{4}''$  on the side opposite the cover plate, on which side the posts are double laced with bars  $2\frac{1}{2}'' \times \frac{1}{4}''$ . The horizontal struts which form the connection between the posts divide the towers into five sections of nearly equal height. The struts running at right angles to the center line of the bridge consist of two channels with the flanges turned in, laced top and bottom; they are inserted through the open side of the tower-posts and connected to them by  $3\frac{1}{4}$ -inch steel pins. The horizontal struts running parallel to the center line of the bridge consist of 4 angles  $4'' \times 4'' \times \frac{1}{8}''$ , double-laced on all four sides, and are riveted to the web plates of the posts. In the three lower sections of the towers the horizontal struts running at right angles to the line of the bridge are intersected by a vertical strut, consisting of two 10-inch channels, laced on the open sides. The various panels formed by the post and horizontal struts are braced diagonally by square tension rods attached to steel pins and provided with sleeve nuts for adjustment.

The general plan of one of these towers is given on Plate LVI and the details are shown on Plate LVII.

Each cantilever consists of a shore arm 195 feet  $2\frac{5}{8}$  inches long, one panel of 25 feet over the tower and a river arm of 175 feet length. The cantilever trusses are divided by vertical posts into panels of 25 feet, with the exception of the end panel of the shore arm, which is 20 feet  $2\frac{5}{8}$  inches; they have a double system of diagonals, and are spaced 28 feet between centers; they are 56 feet deep over the towers, 26 feet over the last vertical post at the river end, and 21 feet over the last vertical post at the shore end.

The upper chords of the shore arm receive alternate tensile and compressive strains; loads which are applied between the anchorage and the tower produce compression, and those applied to the river arm or intermediate span produce tension. As the largest strains are in tension, the upper chord of the shore arm has been constructed as a tension member composed of eye-bars mainly, but has a compression member packed in between the chord-bars. The eye-bars are 8 inches wide and vary from  $1\frac{1}{4}$  to  $1\frac{3}{4}$  inches in thickness.

The compression member consists of two web plates  $18'' \times \frac{1}{4}''$ , and four angles  $3\frac{1}{2}'' \times 5'' \times \frac{1}{4}''$ , double latticed top and bottom. This strut is made in lengths of one panel, with pin joints at each end.

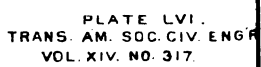
The upper chord in the tower panel consists of fourteen 8-inch eye-bars.

The upper chord of the river arm, which is, like that over the tower, strained in tension only, is composed entirely of eye-bars, 8 inches and 6 inches deep.

The lower chords and inclined end-posts of the cantilevers are steel compression members, which are composed of plates and angles; each member consisting of two side plates 24 inches wide, varying in thickness from  $\frac{1}{2}$  to  $\frac{3}{4}$  inches, and four angles  $4'' \times 4'' \times \frac{1}{4}''$ ; the section is increased by filling plates on each side between the angles and additional side plates, according to the required area. No cover plates were used, so as to preserve a symmetrical section, the pins passing through the neutral axis of the member, which the writer considers to be very essential in compression members in order to develop the full strength of the same.

The vertical posts over the tower supports are of steel, and composed of two plates  $20'' \times \frac{1}{4}''$ , and four angles  $4'' \times 4'' \times \frac{1}{4}''$ , double-laced; they are made in two lengths and spliced in the center.

The intermediate vertical posts are formed of two 12 or 15-inch chan-



nels with the flanges turned outwards and double-laced. The three posts next to the tower are made in two sections and spliced.

The main ties are eye-bars 6 and 7 inches wide, and from  $1\frac{1}{4}$  to  $1\frac{1}{2}$  inches thick; they are made in two lengths, with the exception of the end ones, and are coupled on a pin which passes through the posts at their intersection with the main ties.

The end panels of the shore arms are provided with counter ties, which consist of two bars  $6'' \times 1\frac{1}{4}''$  in the last panel, and square bars, varying from 2 to  $1\frac{1}{2}$  inches, in the adjoining three panels. At both ends of the cantilevers, the last two sets of ties are connected on the pin which passes through the intersection of the center line of the end-post with that of the lower chord by means of short links. These links are interposed for the purpose of defining the strains in the ties attached to them, their center line determining the direction of the resultant.

All the principal connections are made by steel pins of  $5\frac{1}{4}$ ,  $6\frac{1}{4}$  and  $7\frac{1}{4}$ -inch diameter.

The vertical posts over each tower are stiffened by a horizontal strut between them at the center, and another strut attached to the same pin and connected with the intermediate pin of the next post and the pin connection of the first set of main ties.

The floor beams are riveted to the vertical posts just below the upper chord, and at these places the two channels forming the vertical posts are connected by a web plate instead of lacing. The floor beams also rest on brackets made of two angles riveted to the inside of the posts, which brackets are also used for the connection of the sway-rods; but this bearing is not estimated in calculating the strength of the connection. The floor beams at the river ends of the cantilevers are suspended from the pin; their ends being attached to short eye-bars, rigidly connected together by a web plate and angles, and to the lower ends of which the suspenders carrying the intermediate span are connected. A rigid connection is also made between these floor beams and the inclined end-posts of the cantilevers by a short strut, consisting of two 10-inch channels, laced.

The wind and sway bracing is arranged in the following manner: Horizontal struts are attached to the main pins of the lower chord and to the intermediate pins in the vertical posts, connecting the posts transversely; these struts consisting of two 8-inch channels, laced.



The connections are made by small pins, which pass through the ends of the struts and through U-nuts screwed to the ends of the main pins. The bends which are thus formed by each pair of vertical posts and the horizontal struts and floor beams, are braced transversely by diagonal rods; the only exceptions are the posts over the tower supports, which are braced transversely by angle-iron lattice-work. The inclined end-posts are also braced transversely by diagonal rods. The connections of the lower lateral rods are on the pins which pass through the U-nuts and the ends of the lateral struts. The upper lateral rods are connected on pins which pass through angle-iron lugs and plates riveted to the floor beams at the upper flange.

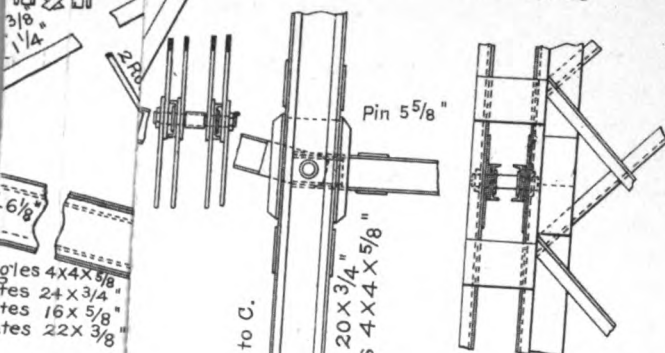
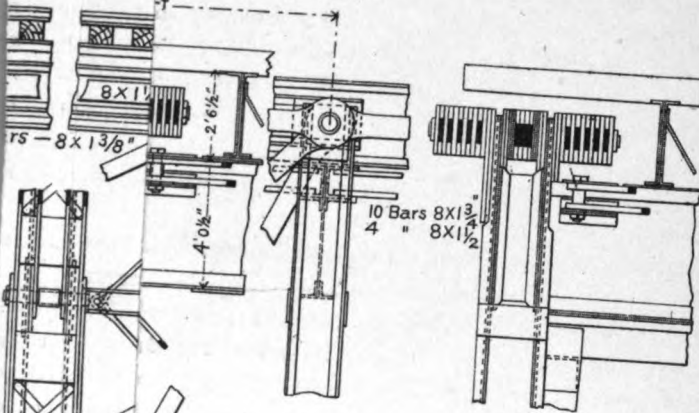
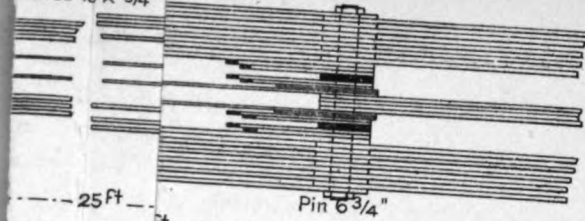
The lateral connections are all made on turned steel pins, and all rods are supplied with sleeve-nuts for adjustment.

The cantilevers are supported over the towers on pins in steel castings, while they are connected at their shore ends to the anchorage piers by rockers. The anchor rods are  $3\frac{1}{4}$  inches square, the upper ends of which are formed into loops, through which pass the pins to which the rockers are connected.

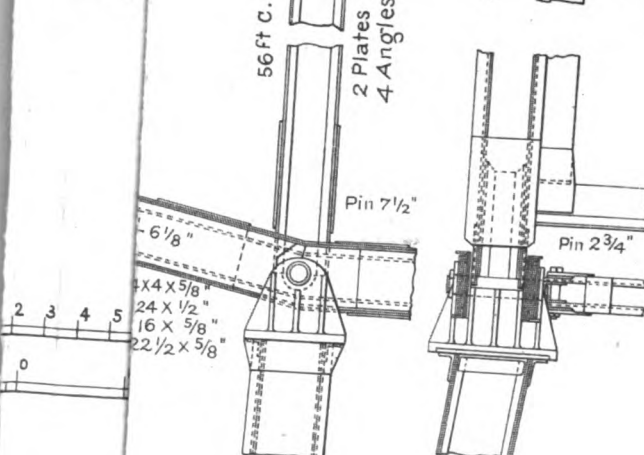
The detail plans of the cantilevers are given on Plates LVIII and LIX. Cross-section over tower is shown on Plate LXI.

The intermediate span is 119 feet  $9\frac{1}{2}$  inches long between centers of end-pins, and 26 feet deep, and is divided into five panels of 24 feet each, excepting the center panel, which is 23 feet  $9\frac{1}{2}$  inches. The trusses are spaced 28 feet between centers, the same as the cantilever trusses. The upper chord consists of two 15-inch channels and one cover-plate,  $24'' \times \frac{3}{8}''$ , doubled laced on the under side. As the lower chord of the intermediate span had to resist compressive strains during erection, it being erected on the cantilever principle, the same as the river-arms of the cantilevers, it was designed as a compression-member, consisting of two 12-inch channels, laced top and bottom. In the center panel the lower chord consists of six 5-inch eye-bars of  $1''$  and  $1\frac{1}{8}''$  thickness. The vertical posts are similar to those of the cantilevers, composed of two 12-inch channels each, laced. The first panel-suspender, which acts only as a support for the lower chord, consists of rods  $1\frac{1}{4}$  inch square, the upper part consisting of two flat bars,  $14'' \times 1\frac{1}{4}''$ , with a web plate riveted between them to form an attachment for the floor beam. This floor beam has a bracket attached to each side connecting with the pin. The diagonals are 4-inch eye-bars, and the center panel is provided with

Angles  $3\frac{1}{2} \times 5 \times \frac{5}{8}$ "  
Plates  $18 \times \frac{3}{4}$ "

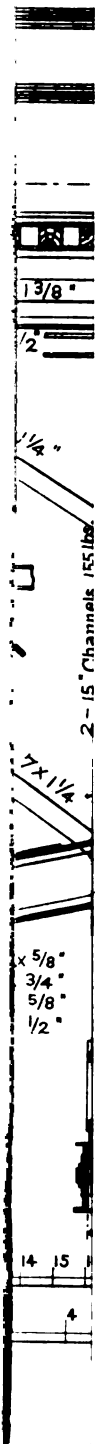


Angles  $4 \times 4 \times \frac{5}{8}$ "  
Plates  $24 \times \frac{3}{4}$ "  
Plates  $16 \times \frac{5}{8}$ "  
Plates  $22 \times \frac{3}{8}$ "



Angles  $4 \times 5 \times \frac{5}{8}$ "  
Plates  $24 \times \frac{1}{2}$ "  
Plates  $16 \times \frac{5}{8}$ "  
Plates  $22 \times \frac{1}{2} \times \frac{5}{8}$ "





counters. The lateral and transverse bracing for the middle span is arranged similar to that of the cantilevers, with the exception of the top lateral bracing over its end panels, which consists of angles riveted to the floor beams and stringers. As these are the panels which have the expansion joints, the angle-iron braces were substituted for the tie-rods, to give lateral stability without interfering with the working of the expansion joints.

The detail plans of the intermediate span are given on Plate LX.

The iron-floor system consists of four lines of stringers placed  $6\frac{1}{2}$  feet between centers, resting directly on the transverse floor beams at each panel point. The bottom flange of the stringer is riveted to the top flange of the floor beam. The stringers are spliced at their joints and braced transversely by brackets made of flat iron. Each set of stringers is stiffened at the center by angle-iron frames riveted between the webs of the stringers. The stringers of the end panels rest at the shore end, on sliding plates on the anchorage piers; the stringers being connected transversely at that place by angle-iron frames similar to those which stiffen them at their center, but extending across the whole width between the trusses and connected to the end-pins, forming the end-top lateral strut. The stringers are proportioned for 78-ton consolidation engines besides their own weight and that of the timber floor. Each stringer consists of a web plate  $30'' \times \frac{3}{4}''$ , and four angles  $5'' \times 3\frac{1}{4}'' \times \frac{1}{2}''$ .

The maximum bending strain on the extreme fiber of the stringers produced by the above-mentioned loads is 8 100 pounds per square inch.

The floor beams consist of a web-plate  $48'' \times \frac{3}{4}''$ , with four angles  $6'' \times 6'' \times \frac{1}{4}''$ , and cover-plates  $14'' \times \frac{1}{2}''$ , 15 and 22 feet long. The web-plate is stiffened by angle-iron stiffeners under each stringer support.

The floor beams are proportioned for the same engine load as the stringers, besides the weight of the floor. The maximum bending strain on the extreme fiber of these beams is 7 660 pounds per square inch. As the floor beams are also acting as top lateral struts, and besides are used to stiffen the structure transversely in connection with the transverse rods, it was thought advisable to give them an excess of strength, and therefore the fiber strain was limited to the one above mentioned.

The floor of the superstructure consists of oak ties  $9'' \times 9''$ , 12 and

16 feet long, resting on the stringers. They are spaced 18 inches between centers, and are notched down 1 inch over the stringers. The track is laid with 60-pound steel rails. There are four lines of oak guard-timbers, spaced 4 feet from the center line of each track; they are notched down 1 inch on the ties and are bolted to every third tie by  $\frac{1}{2}$ -inch bolts. The spaces between the outer guards and ribbons are planked with 2-inch pine plank. An iron hand-rail is placed on each side of the floor, consisting of cast-iron posts, spaced 6 feet apart and bolted to the ribbons, carrying four lines of  $1\frac{1}{4}$ -inch gas-pipes. The floor, including rails and hand-rails, is estimated to weigh 795 pounds per lineal foot, assuming the weight of timber at 4 pounds per foot, board measure.

The quantities of material for the superstructure are given in the following table:

	Steel.	Wrought Iron.	Cast Iron.	Total.
	Lbs.	Lbs.	Lbs.	Lbs.
Towers.....	445 170	238 232	87 075	770 477
Cantilevers.....	822 156	2 370 478	18 500	3 211 134
Intermediate span.....	4 735	289 693	545	294 973
Anchorage.....	.....	177 840	3 192	181 032
Hand-rails.....	.....	16 438	17 136	33 574
	1 272 061	3 092 681	126 448	4 491 190

Timber floor, oak..... 135 400 feet, B. M.

“ “ pine..... 18 300 “ “

153 700 “ “

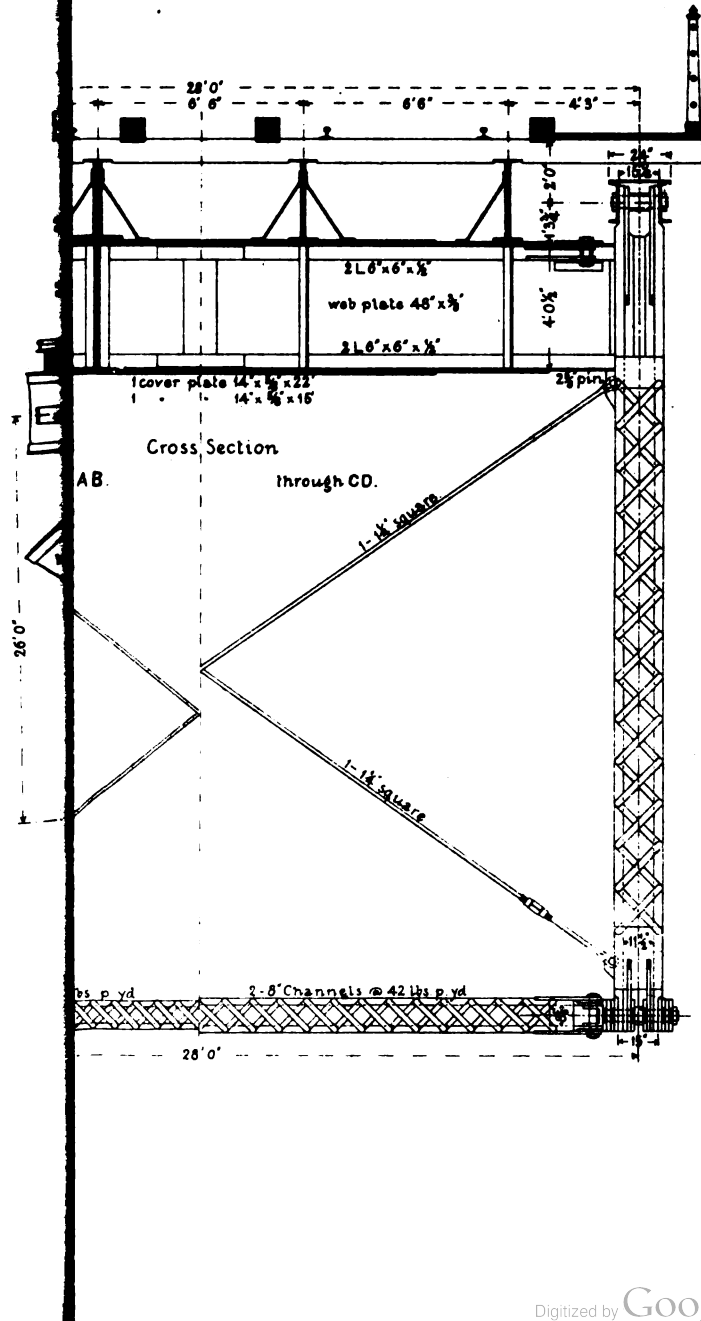
#### MATERIALS FOR SUPERSTRUCTURE.

The iron used in the construction of the bridge was manufactured by Messrs. Atkins Brothers, of Pottsville, Pa., and Graff, Bennet & Co., of Pittsburgh, Pa.

The steel was made by the Spang Iron and Steel Company, Limited, of Pittsburgh, Pa., with the exception of the steel used for the pins and the steel castings, which were made by the Cambria Iron Company, of Johnstown, Pa.

All materials were manufactured into finished members at the shops

PLATE LX  
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of the Central Bridge Works, at Buffalo, excepting the compression members for the towers, which were made in the shops of Messrs. Kellogg & Maurice, of Athens, Pa.

The work in the shops was inspected by Mr. Jacob Jung.

The heads of the eye-bars were formed by die-forging, which is done by piling pieces of scrap on the end of the bar, which is then heated to a welding heat and hammered under a steam-hammer into a die which has the shape of the eye-bar head. Some of these finished eye-bars were tested in the Government testing machine at the Watertown Arsenal. A record of these tests is given in Appendix No. 3.

The material was inspected at the mills by Mr. W. F. Zimmermann, of Pittsburgh, Pa., who made all the sample tests which are recorded in Appendix No. 4.

The records of the tests of accepted steel are accompanied by a partial chemical analysis furnished by the manufacturers. In selecting the material the specifications were strictly adhered to. The following change, however, was made from the original specification for the steel: The original specification required the sample bars to show a reduced area at the point of fracture of 35 per cent.; this was changed to 25 per cent.

The steel test bars were all rolled from 4" × 4" test ingots into 1" rounds.

According to the writer's experience with steel for structural purposes which had to be made according to a specification, there have always been considerable delays, and this case was no exception to the rule. The records of the tests will show that the steel which has been accepted was of a good uniform quality. There were 245 heats made by the Spang Iron and Steel Company, of which 109 heats were accepted and 136 rejected.

#### ERECTION.

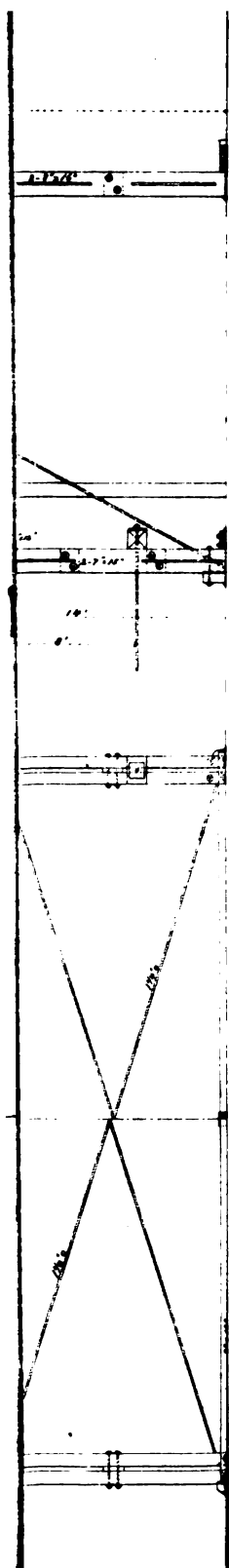
The towers were erected by means of derricks placed at the ends of the same false-works which had been used to handle the stone for the masonry. The material was lowered with a derrick from the cliff upon hand-cars, and run out to the derrick at the end of the false-works which were used for the handling of the iron in erecting the towers. The various pieces composing the towers were lowered and placed in their respective positions by this derrick, and the towers thus completed in sections.

The erection of the tower on the American side was commenced on August 29th, and on the Canadian side on September 10th. The tower on the American side was completed on September 8th, and on the Canadian side on September 18th. After the towers were erected, upper false-works were put up on top of the trestles, and the shore arms of the cantilevers erected. The shore arms being completed, a track was laid on the completed shore arms, and the erection of the river arms commenced.

The river arms of the cantilevers were erected with a traveler, constructed for that purpose by the contractors. This traveler, the details of which are shown on Plate LXII, was built in place on top of the shore arm. The traveler consisted of a substantial wooden framework braced with iron rods. It was supported by cast-iron wheels 14" diameter and 8" face on steel axles; these wheels run between timber guides on the outside stringers of the permanent bridge. The traveler was anchored to the three last transverse floor beams of the completed portion of the structure by iron clamp-hooks, hinged to a bolt which passed through the timbers, and was provided on the top with screw-end, nut and washer plate. These clamp-hooks caught under the upper flanges of the floor beams; by screwing up the nuts the clamps held the traveler firmly down to the truss. The traveler was 66 feet 6 inches long from out to out, projecting 40 feet beyond the completed truss, with an extension on the rear end reaching to the next panel point, where it was anchored by the clamp-hooks. The extreme height of the traveler was 21 feet, and the width over all 38 feet 6 inches. The traveler was provided with 2 derricks; power was furnished by a Copeland & Bacon hoisting engine with 7½ inches diameter of cylinder and 8-inch stroke, and a vertical tubular boiler 2 feet 6 inches by 7 feet. The engine was connected by proper gearings and shafts, shown on the plan (Plate LXII), with the winches around which the hoisting ropes were passed. The material was conveyed on hand-cars run on a track in the center from the shore end to the traveler. The derricks were so arranged that they could lift the material from the car and lower the pieces into their respective places in the structure, or transfer the same to one of the tackles suspended from a cross-beam on top of the traveler. Those travelers were at first placed on the panels over the towers, and the adjoining panels were then erected in the following manner: The members composing the panel were lowered into place and held in position







until the connections were made and the panel thus completed. The traveler was then moved forward one panel length; this was done, after the clamp-hooks were loosened, by passing ropes, which were fastened to the front floor beam, over the sheaves *A A* (see Plate LXII) back to the winches.

For the safety and convenience of the workmen while connecting the different parts, a hanging platform was used suspended from the traveler by 1-inch iron rods, which were adjustable by having short links of 3 feet length attached to their lower ends, this distance being equal to the rise of the lower chord in each panel length; the height of the platform could be adjusted by removing one set of links at each advance.

Plate LXIII shows the bridge during construction; the false-works used for the erection of the shore arm of the cantilever are shown in place. The traveler is in position for the erection of the third panel from the tower; a chord section is just being put in place. The heaviest members handled with this traveler weighed about 12 000 pounds.

The intermediate span which connects the cantilevers was also erected with the travelers, the lower chord (with the exception of the center panel) being made a compression member. The last position of the travelers was over the end panels of the cantilevers, leaving a space of 40 feet between them. After a portion of the intermediate span had been erected, a number of timber beams were laid across on top between the travelers, bridging the open space, and forming false-works from which the last connections were made.

The first metal of the cantilever shore arm on the American side was placed on the false-works on September 25th, and erection completed on October 15th. The erection of the cantilever shore arm on the Canadian side was commenced on October 8th, and finished on October 22d. The traveler on the American side was completed on October 25th, and erection of the river arm commenced on October 28th. The traveler on the Canadian side was completed on October 31st, and erection of the river arm commenced on November 4th. The last connection was made on November 22d. at 11.55 A. M. The travelers were then taken down and the false-works removed. The laying of the tracks was commenced on November 31st, and the first track completed on December 6th, when the bridge was crossed for the first time by Mr. G. H. Burrows, Division Superintendent of the New York Central and Hudson River Railroad,

in his pony engine, having on board with him the engineers of the bridge. The whole structure was completed on December 19th.

The entire superstructure was raised under the direction of Mr. S. V. Ryland, Superintendent of Erection for the Central Bridge Works.

#### TESTS.

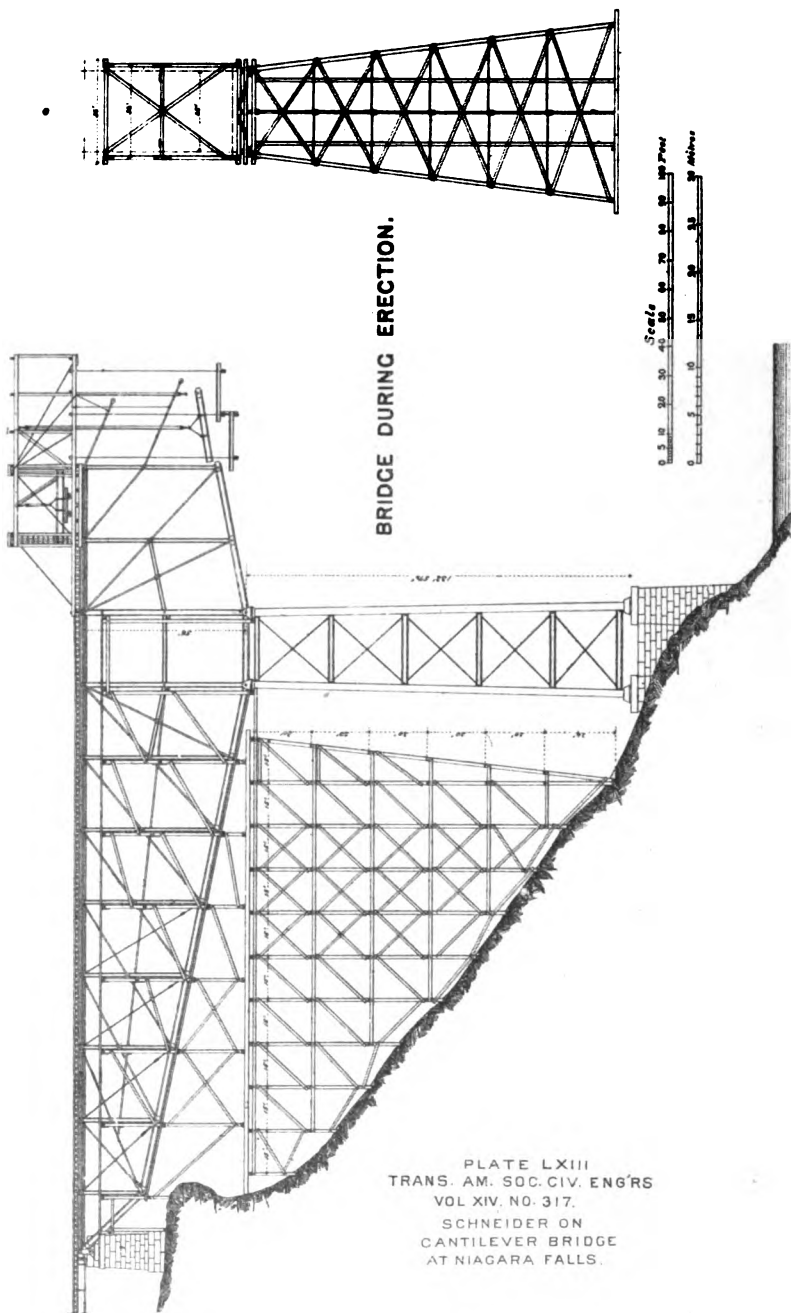
On December 20th the bridge was formally opened for traffic, and tested in the presence of a great number of engineers who were invited for the occasion. The tests were conducted by a committee consisting of Messrs. George S. Morison, M. Am. Soc. C. E., Theodore Cooper, M. Am. Soc. C. E., and Charles Macdonald, M. Am. Soc. C. E., of New York City, and Mr. Thomas Ridout, Engineer, Department of Railways and Canals, of Ottawa, Canada.

As the observations made on the day of the opening were not considered absolutely reliable on account of the inclemency of the weather, the committee expressed a desire to make some additional tests some time during the following spring or summer.

According to agreement the 9th of June was selected for the additional tests; the necessary preparations having been completed, the same engineers met again at the bridge site and made the tests, the results of which are given in the Report of the Testing Committee in Appendix No. 5.

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A reproduction of a photograph of the Niagara Cantilever Bridge after completion, is given on Plate LXVII.



## APPENDIX No. 1.

## SPECIFICATION FOR BRIDGE OVER NIAGARA RIVER AT SUSPENSION BRIDGE.

## GENERAL.

The bridge will be constructed for two railroad tracks, and will consist of two pairs of cantilevers, each 380 feet between the centers of end-pins, and supporting a fixed span between their inner ends, 125 feet between centers of end-pins; these dimensions may be varied to suit final location. The cantilever spans at their land ends shall be anchored to and supported by stone abutments, and at their centers by steel piers, as shown on general plan; the latter shall rest upon stone piers.

## PLANS.

Full detail plans showing all dimensions will be furnished to the engineer of the Bridge Company for his approval before the work of construction is commenced, and any modification required by him to insure a safe and substantial structure as contemplated and described in these specifications, shall be made. The work will be done in all respects in accordance with these plans, and such modifications thereof as may be required by the engineer.

## EXCAVATIONS.

Pits for the foundations of the piers and abutments shall be excavated to the depths and dimensions directed by the engineer in charge, and shall be made smooth and level on the bottom.

## MASONRY.

The masonry in piers and abutments shall be built of sound, durable limestone, of quality approved by the engineer, and will be first-class ashler work.

All coping stone shall have beds and joints well bush-hammered and cut true to the square, and shall be laid in the work so as to form a quarter inch joint throughout. Each face stone shall be rock-faced with edges pitched to straight lines and no projections exceeding 2 inches. The beds to be bush-hammered throughout and the end joints for 12 inches back. Stone to be laid on their natural beds in regular courses with 1 header to every 2 stretchers, and bonding not less than 12 inches with contiguous courses. No header to be less than  $4\frac{1}{2}$  feet long and no stretcher less than  $2\frac{1}{2}$  feet wide. No stone to be less than 12 inches thick. The whole to be set level, well bedded in mortar, and laid to a  $\frac{1}{4}$  inch joint. The backing and filling to be of large-sized stone, laid in courses corresponding with the face stone, but 2 courses may fill up one of the faces, provided no stone less than 9 inches thick is used; the broadest bed to be laid undermost and to have a good bearing on the stone below.

A draft of 2 inches wide shall be cut at all angles in the masonry.

Not more than 2 unfinished courses shall be allowed at one time in any wall, and, as a rule, 1 course must be wholly finished before another is begun.

Cement shall be of the best quality hydraulic, newly manufactured,

and well packed and protected until required for use; none to be used until accepted by the engineer. Any cement condemned by him must be immediately removed from the work.

All sand used in mortar must be sharp and clean, and free from loam and vegetable matter.

The general proportions of sand and cement for common mortar will be 2 of sand and 1 of cement, and for pointing mortar 1 of sand and 1 of cement. It shall be used immediately after mixing, and any that has commenced to set or has been mixed more than half an hour shall be rejected and thrown away. Each course of masonry when laid shall be thoroughly grouted with thin mortar of the same quality as the above.

All outside joints will be raked out to a depth of 1 inch and neatly pointed.

#### LOADS.

The structure shall be proportioned in all its parts to sustain the maximum strains produced by the following loads:

1st. The dead weight of the structure itself.

2d. A train on each track, headed by two 66-ton consolidated engines, having 72 000 pounds on 3 pairs of drivers spaced 6 feet between centers, followed by a train weighing 1 ton per lineal foot.

3d. A wind pressure of 30 pounds per square foot on a train surface of 10 square feet per lineal foot of bridge, plus the same pressure per square foot upon the vertical surface of one side of the structure and track, exposed in any direction.

#### MATERIALS FOR SUPERSTRUCTURE.

The posts of the towers, and the posts, bottom chords and pins of cantilevers, shall be of steel. All other members shall be of wrought iron.

#### STRAINS.

In iron members the tensile strains allowed shall be:

In main ties and chord bars for live load.	10 000 lbs. per sq. in.		
“ counters.....	8 000	“	“
“ laterals and sway bracing.....	15 000	“	“
“ chord bars for dead loads.....	12 500	“	“
“ chord bars for wind strains.....	12 500	“	“

In compression the allowed strain shall meet the requirements of Gordon's formula.

In steel members the tensile strain per square inch allowed shall be as follows, viz.:

$$11\,000 \left( 1 + \frac{\text{minimum strain.}}{\text{maximum strain.}} \right)$$

This result shall be used in Gordon's formula for compression as the numerator of the fraction in place of 8 000.

#### MATERIALS.

All material shall be subject to inspection, at all times during its manufacture, by the engineer and his inspectors, and any machine or other tests of material that he may desire shall be made without charge.

All steel shall be manufactured by the open-hearth process. Bessemer steel will not be accepted.

Steel used in compression members, pins and steel plates, shall contain not less than  $\frac{3}{100}$  nor more than  $\frac{1}{100}$  of 1 per cent. of carbon, and less than  $\frac{1}{100}$  of 1 per cent. of phosphorus.

Sample test bars of  $\frac{1}{2}$  of an inch in diameter shall bend 180 degrees around that diameter without sign of crack or flaw.

The same bars tested in a lever machine shall show an elastic limit of not less than 50 000 pounds per square inch, and an ultimate strength of not less than 80 000 pounds. It shall elongate at least 15 per cent. in a length of 8 inches before breaking, and shall have a reduced area of 35 per cent. at point of fracture.

The iron used in tension members shall be double refined iron. Small samples having a minimum length of 8 inches shall show an elastic limit of at least 26 000 pounds and an ultimate strength of 50 000 pounds per square inch, and shall elongate 15 per cent. and show a reduction of 25 per cent. at point of fracture. The fracture shall be of uniform fibrous character. When tested in full-sized bars, a reduction of from 5 to 15 per cent. will be allowed, according to the size of the bars. Small samples taken from plate and shape iron shall show an elastic limit of 24 000 pounds and an ultimate strength of 47 000 pounds per square inch; shall elongate 10 per cent.; and show a reduction of area of 15 per cent. at point of fracture.

#### WORKMANSHIP.

In riveted work the holes shall not be punched more than  $\frac{1}{8}$  inch larger than diameter of rivet to be used, and when the parts are assembled a cold rivet shall pass through without reaming; no drifting will be permitted.

All rivets in steel members shall be of steel, and, wherever possible, all rivets shall be driven by power.

All pin-holes shall be truly bored, so as to be equally distinct, parallel to each other and at right angles to the axis of the member. All bearing-surfaces shall be truly faced.

Power-riveters shall be direct-acting machines, worked by steam, hydraulic pressure or compressed air, and capable of holding on to the rivet when upsetting is completed.

Surfaces in contact shall be painted before being put together.

All eye-bars shall be die-forged. No welding in the body of the bar will be allowed. All screw ends shall be enlarged, so that the diameter of the bottom of the thread shall be equal to the diameter of the bar.

All pins shall be truly turned to a gauge, and shall be of full size throughout. The pin-holes shall be bored to fit the pins, with a play not exceeding  $\frac{1}{32}$  of an inch. Pilot-nuts shall be used in the erection of the work.

All workmanship, whether particularly specified or not, must be of the best kind now in use in first-class bridge work. Flaws, incorrect lengths, surface imperfections or irregular shapes will be sufficient ground for rejection of material.

Constant and strict inspection of the whole of the workmanship and materials will be made during the progress of manufacture, and at any time before or after manufacture the engineer shall have power to reject any part of the whole on account of defects.



## PAINTING.

The surfaces of all members connected by rivets shall be well painted before they are put together, and all the work shall have one coat of approved metallic paint before it leaves the shop, except machine-finished bearing surfaces, which shall be coated with white-lead and tallow. The entire iron work shall, after erection, receive two good coats of white lead mixed with boiled linseed oil.

## TIMBER WORK.

Cross-ties will be of best quality white oak, 26 feet in length, 9"×9" square, and will be boxed down 1 inch on the track stringers.

There will be four lines of guard-timbers of white oak 8"×8", and boxed down 1 inch on the ties and bolted to every third tie by  $\frac{3}{4}$ -inch bolt.

Foot-walks of 2-inch pine plank and about 3 feet 3 inches wide will be laid on the ends of the ties, between the outer guard-timbers and the hand-rails.

## HAND-RAIL.

A hand-rail will be placed on each side of the bridge, consisting of cast-iron posts, placed 4½ feet apart and bolted to the ties, and supporting four lines of 1½-inch gas-pipe railing.

The railing to be painted in the same manner as the other iron-work.

## APPENDIX No. 2.

## REPORTS BY EXPERTS ON FOUNDATIONS.

J. TILLINGHAST, Esq.,

*Vice-President Niagara River Bridge Company.*

DEAR SIR,—We have this day visited the site of the new bridge across the Niagara River, which is now being built by the Central Bridge Works, of Buffalo, a short distance above the Railroad Suspension Bridge.

We have visited the foundation pits and examined the foundations as thoroughly as circumstances would admit of. In some respects, this examination would have been more satisfactory if made before any of the beton filling had been put in the pits; but the main question, the stability of the irregular masses of rock on which the piers rest, can be determined now as well as at an earlier date.

We find that the piers which are to support the iron towers which carry the cantilevers, are founded on layers of beton nowhere less than five feet thick, in pits excavated in the broken rock which forms the slope from the foot of the vertical bluff to the water's edge, the bottom of the excavation being about level with the water. This rock consists of fragments of various sizes, some of them over thirty feet long, which have broken off of the hard limestone ledges, which are found higher up on the sides of the chasm. Although the situation is such that no soundings can be taken in the water, except at great expense, and then only near the shore, the character of the slope shows that it must extend under water with a stable angle, and the growth of large trees immedi-

ately at the edge of the water shows that the mass has remained undisturbed for a very long period. A short distance below the bridge the natural formation is found in position on the east side of the river. It consists of a soft red stone, which is affected by the water, with harder layers of limestone above; it is these harder layers which have fallen and formed the *debris* when the soft stone had been worn from below by the water.

We consider that this mass of hard boulders, which owe their present stability to gravity, forms a foundation whose stability is second only to rock in position, and extending at an approximately uniform level across the river; and we consider such a foundation very much safer and more stable than a foundation on a stratified rock rising abruptly from the water, and exposed to the wear of the channel below the surface.

We find that the weight which your bridge will throw upon this foundation is within the limits which safe practice allows for foundations on loose material; and we consider the method adopted, of distributing the weight on a mass of concrete which locks into the interstices of the mass of rocks, the best method of starting a foundation in such a situation.

We consider that the plans for these foundations which have been adopted are to all intents and purposes the same that we should have selected if the work had been placed in our charge and the plans prepared by ourselves, and believe that the use of piers located as these are, near the water's edge, is a wise economy in reducing the length of the span and simplifying the construction of the bridge.

We should recommend you, without hesitation, to proceed with the construction of your bridge on the plans now in use.

GEORGE S. MORISON.

CHARLES MACDONALD.

NIAGARA FALLS, ONT., July 6th, 1883.

BUFFALO, July 12th, 1883.

CHARLES H. FISHER, Esq.,

*Chief Engineer New York Central and Hudson River Railroad.*

DEAR SIR,—Having been requested by you to examine the location of the piers for the proposed new bridge of the Niagara Bridge Company, we have visited the site, inspected the work done, and gathered such information as we could obtain during our visit.

At the place chosen for the bridge the slopes are not perpendicular, as they are in many places above and below this point, but are at an angle of about 45 degrees, and the excavations made show that the material forming the face of the hill is not *in place*, but has been broken off from higher ground, and has fallen and packed together, forming a mass of boulders, mixed with clay, shale and other material from the original strata. Many of the boulders showing in the face of the slope are of immense size and appear to be firmly seated in their places.

The indications are that the present slope, showing above the water, has not been disturbed for a long time, as large trees are growing at various points on the slope referred to, and the condition of its surface indicates that there has been no serious undercutting by the river currents.

We know by inspection of other points further down the river that soft strata do exist in the river cliffs; but at this place they would appear to be covered up and protected by the slope composed of a gigantic mass of rip-rap placed by nature in the same position that engineers would desire to place it artificially if it did not now exist there.

The excavations for the foundations of the piers have been made in the mass of rip-rap referred to; the bottom of the excavations being at or near the water's level. The excavations are simply niches cut into the slopes.

Beton some 6 to 10 feet in thickness has been placed over the surface of the excavation, filling up the whole width of the cutting and forming apparently a solid mass. Upon this the masonry for the footings of the towers of the bridge have been started.

The question submitted to us is as follows: Is this a safe and reliable foundation for the proposed structure?

The stability of the proposed foundations depends entirely upon the stability of the mass of rip-rap which we have described, and under ordinary circumstances the foundations would be considered ample.

The stability of the mass of rip-rap depends upon the permanency of the footings of the slopes under the river at points inaccessible to examination; the only danger to be apprehended is an undercutting of the foot of the slopes, and there are no present indications that such action is taking place, nor is it likely to take place at this point in the future, unless there should be radical changes in the river.

Taking all things into consideration, we are of the opinion that the foundations as now projected are as secure as can be had at this place; as any future changes in the river which would affect the integrity of the proposed foundations, would in all probability equally affect any other foundation which could be constructed in this mass of natural rip-rap.

After considering the subject in all its bearings, our judgment is that there is no good reason for changing the foundations as now projected.

JOHN A. WILSON,  
*Civil Engineer, Philadelphia.*

A. W. STEDMAN,  
*Chief Engineer Lehigh Valley Railroad.*

THEODORE COOPER,  
*Civil Engineer, 35 Broadway, New York.*

### APPENDIX No. 3.

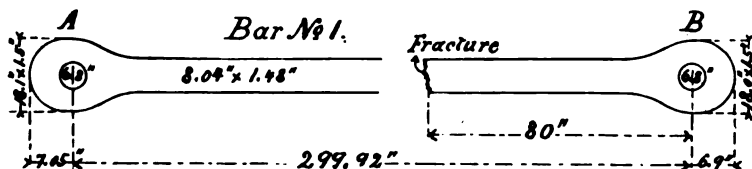
#### TESTS OF FULL-SIZED EYE-BARS.

WATERTOWN ARSENAL, MASS., October 16th, 1883.

REPORT OF MECHANICAL TESTS WITH THE 400-TON U. S. TESTING MACHINE, FOR THE CENTRAL BRIDGE WORKS, BUFFALO, N. Y.

TESTS MADE BY JAMES E. HOWARD.

*Tensile Tests of 2 Wrought Iron Eye-bars.*



Sectional area 11.9 square inches; gauged length 200 inches; twenty-eight 10" sections laid off on stem.

APPLIED LOADS.		IN GAUGED LENGTH.		ELONGATION C. TO C. OF PINS. Inches	APPLIED LOADS.		IN GAUGED LENGTH.		ELONGATION C. TO C. OF PINS. Inches	REMARKS.
Total lbs.	Lbs. per □	Elongation inches	Set. Inches		Total lbs.	Lbs. per □	Elongation. Inches	Set Inches		
11 900	1 000	0	.....	0	278 000	.....	..2100	.....	.....	
59 500	5 000	..0310	.....	..14	280 000	.....	..2140	.....	.....	
119 000	10 000	..0693	.....	..24	282 000	.....	..2170	.....	.....	
.....	1 000	.....	..0021	..32	284 000	.....	..2220	.....	.....	
178 500	15 000	..1085	.....	..04	286 000	.....	..2265	.....	.....	
214 200	18 000	..1362	.....	.....	288 000	24 200	..2295	.....	.....	Elastic limit.
226 000	19 000	..1478	.....	.....	290 000	.....	..2333	.....	.....	
238 000	20 000	..1595	.....	..32	292 000	.....	..2372	.....	.....	
.....	1 000	.....	..0204	..04	294 000	.....	..2417	.....	.....	
240 000	.....	..1646	.....	.....	296 000	.....	..2500	.....	.....	
242 000	.....	..1672	.....	.....	297 500	25 000	..2500	.....	..52	
244 000	.....	..1688	.....	.....	.....	1 000	.....	..0804	..18	
246 000	.....	..1708	.....	.....	303 450	25 000	..2670	.....	.....	
248 000	.....	..1725	.....	.....	309 400	26 000	..2740	.....	.....	
250 000	.....	..1747	.....	.....	321 300	27 000	..2700	.....	1.18	
252 000	.....	..1767	.....	.....	333 200	28 000	.....	.....	1.92	
254 000	.....	..1786	.....	.....	345 100	29 000	.....	.....	2.65	
256 000	.....	..1813	.....	.....	357 000	30 000	.....	.....	3.40	
258 000	.....	..1833	.....	.....	368 900	31 000	.....	.....	4.15	
260 000	.....	..1861	.....	.....	380 800	32 000	.....	.....	5.08	
262 000	.....	..1882	.....	.....	.....	.....	.....	.....	.....	{ Snapping sounds.
264 000	.....	..1905	.....	.....	416 500	35 000	.....	.....	8.05	
266 000	.....	..1934	.....	.....	476 000	40 000	.....	.....	15.35	
268 000	.....	..1974	.....	.....	.....	.....	.....	.....	.....	{ Ultimate strength.
270 000	.....	..1997	.....	.....	491 200	41 280	.....	.....	.....	
272 000	.....	..2025	.....	.....	0	0	12.15	.....	.....	
274 000	.....	..2048	.....	.....	.....	.....	.....	.....	17.39	
276 000	.....	..2070	.....	.....	.....	.....	.....	.....	.....	6.1%
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	5.8%

Elongation of pin-holes, A ".23, B ".32.

Elongation of 10" sections, ".33, ".66 ".57, ".56, ".58, ".57, ".58, ".57, ".59, ".57, ".59, ".59, ".59, ".60, ".56, ".60, ".61, ".62, ".61, ".61, ".64 (.80 fractured section), ".61, ".66, ".66, ".69, ".69, ".45.

Area at fracture, 7.73 x 1.39 = 10.74 square inches.

Contraction of area 9.7 per cent.

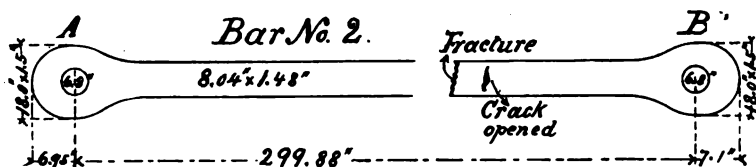
Broke stem 80" from center of hole B.

Appearance of fracture: dull, fibrous, laminated, with 10 per cent. granular streaks.

Started crack at outside of head A, front of eye; also opened seam in neck.

Cracks barely started in head B, front of pin.

Correct.—J. E. HOWARD.



Sectional area 11.9 square inches; gauge length 200 inches; twenty-eight 10" sections laid off on stem.

APPLIED LOADS		IN GAUGED LENGTH.		ELONGATION C. TO C. OF PINS. Inches	APPLIED LOADS.		IN GAUGED LENGTH.		ELONGATION C. TO C. OF PINS. Inches	REMARKS.
Total lbs.	Lbs. per sq. in.	Elongation. Inches	Set. Inches		Total lbs.	Lbs. per sq. in.	Elongation. Inches	Set. Inches		
11 900	1 000	0	.....	0	286 000	24 030	" .2085	.....	.....	Elastic lmt.
59 500	5 000	" .0326	.....	" .07	288 000	.....	" .2122	.....	.....	
119 000	10 000	" .0722	.....	" 15	290 000	.....	" .2174	.....	.....	
.....	1 000	.....	" .0015	" .02	292 000	.....	" .2220	.....	.....	
178 500	15 000	" .1110	.....	" .23	294 000	.....	" .2260	.....	.....	
214 200	18 000	" .1360	.....	.....	296 000	.....	" .2305	.....	.....	
226 100	19 000	" .14 8	.....	.....	297 500	25 000	" .2350	.....	" .45	
238 000	20 000	" .1546	.....	" .31	.....	1 000	.....	" .0537	" .12	
.....	1 000	.....	" .0108	" .04	300 000	.....	" .2440	.....	.....	
240 000	.....	" .1584	.....	.....	302 000	.....	" .2496	.....	.....	
242 000	.....	" .1612	.....	.....	304 000	.....	" .2555	.....	.....	
244 000	.....	" .1630	.....	.....	306 000	.....	" .2600	.....	.....	
246 000	.....	" .1645	.....	.....	308 000	.....	" .2670	.....	.....	
248 000	.....	" .1663	.....	.....	309 400	26 000	" .2740	.....	" .55	
250 000	.....	" .1680	.....	.....	312 000	.....	" .2882	.....	.....	
252 000	.....	" .1696	.....	.....	314 000	.....	" .3050	.....	.....	
254 000	.....	" .1716	.....	.....	316 000	.....	" .3240	.....	.....	
256 000	.....	" .1732	.....	.....	321 300	27 000	" .3750	.....	" .73	
258 000	.....	" .1750	.....	.....	333 200	28 000	" .9860	.....	1" .52	
260 000	.....	" .1770	.....	.....	345 100	29 000	.....	.....	2" .07	
262 000	.....	" .1791	.....	.....	357 000	30 000	.....	.....	2" .58	
264 000	.....	" .1811	.....	.....	368 900	31 000	.....	.....	3" .30	
266 000	.....	" .1832	.....	.....	380 800	32 000	.....	.....	4" .12	
268 000	.....	" .1850	.....	.....	416 500	35 000	.....	.....	6" .85	
270 000	.....	" .1872	.....	.....	443 000	.....	.....	.....	.....	{ Snapping sounds.
272 000	.....	" .1900	.....	.....	476 000	40 000	.....	.....	13" .23	
274 000	.....	" .1925	.....	.....	498 000	41 860	.....	.....	.....	{ Ultimate strength.
276 000	.....	" .1950	.....	.....	0	0	11" .29	.....	.....	5.6%
278 000	.....	" .1974	.....	.....	.....	.....	.....	.....	17" .24	= 5.7%
280 000	.....	" .1998	.....	.....	.....	.....	.....	.....	.....	
282 000	.....	" .2025	.....	.....	.....	.....	.....	.....	.....	
284 000	.....	" .2055	.....	.....	.....	.....	.....	.....	.....	

Elongation of pin-holes, A " .16, B " .20.

Elongation of 10" sections, " .31, " .51, " .55, " .52, " .53, " .53, " .54, " .53, " .55, " .54, " .55, " .55, " .57, " .56, " .59, " .61, " .54, " .64, " .56, " .56, " .59, " .57, " .58, " .60, " .60, " .64, " .64, " .37.

Fractured stem 19" from center of bar.

Appearance fibrous, lamellar 60 per cent., granular 40 per cent.

Area at fracture 7" .77 x 1" .38 = 10.72 square inches.

Contraction of area 9.9 per cent.

Opened crack 21" from fracture, 3" long, having fibrous appearance.

H. H. PARKER,

Major of Ordnance Commanding.

Correct.—J. E. HOWARD.

## APPENDIX No. 4.

## SAMPLE TESTS OF IRON AND STEEL.

STEEL MADE BY SPANG STEEL AND IRON COMPANY, PITTSBURGH, PA.

Steel made in Stationary Furnace. Accepted.

Number.	CHEMICAL ANALYSIS.			MECHANICAL TESTS.			
	Carbon, per cent.	Manga- nese, per cent.	Phos- phorus, per cent.	Elastic Limit, Pounds per square inch	Ultimate Strength, Pounds per square inch	Elongation in 8 inches, per cent.	Reduction of Area, per cent.
1	0.34	0.400	.....	60 780	83 970	18.75	46.52
2	0.35	0.510	.....	64 100	89 920	18.25	41.20
3	0.32	0.530	.....	65 780	87 030	15.62	41.45
4	0.36	0.622	0.096	54 440	83 620	20.37	39.24
5	0.34	0.508	0.099	53 540	79 780	19.25	43.62
6	0.38	0.610	0.099	53 720	83 230	20.87	39.43
7	0.34	0.450	.....	56 660	87 600	19.37	36.53
8	0.32	0.381	.....	58 120	79 790	21.00	45.36
9	0.35	0.580	.....	58 230	88 000	18.50	34.55
10	0.31	0.607	.....	52 340	81 860	17.75	36.80
11	0.33	0.618	.....	55 450	79 810	21.25	42.60
12	0.32	0.726	.....	53 050	80 880	20.00	37.60
13	0.36	0.650	.....	53 900	83 810	18.75	35.40
14	0.34	0.454	.....	55 030	82 070	19.50	43.60
15	0.32	0.509	.....	54 320	80 510	21.25	40.80
16	0.35	0.520	.....	50 998	81 350	18.75	37.60
17	0.35	0.505	0.105	51 530	81 690	18.12	34.90
18	0.31	0.540	.....	54 160	84 520	21.87	35.60
19	0.32	0.639	0.090	50 840	81 010	18.12	38.90
20	0.40	0.643	0.099	56 070	84 540	17.50	33.20
21	0.35	0.462	.....	50 730	79 900	18.37	32.70
22	0.36	0.412	0.098	49 080	80 630	20.50	33.10
23	0.37	0.441	.....	51 130	79 550	19.75	35.30
24	0.34	0.480	.....	50 090	80 010	17.50	32.90
25	0.31	0.621	.....	61 850	84 110	19.00	36.80
26	0.38	0.588	0.080	54 870	85 620	17.50	35.00
27	0.35	0.510	0.082	49 450	79 430	19.37	45.50
28	0.31	0.353	.....	55 630	85 730	16.75	38.00
29	0.30	0.470	.....	52 070	82 650	18.75	34.80
30	0.35	0.570	.....	51 510	83 650	15.75	31.30
31	0.36	0.610	.....	51 370	84 720	19.50	33.40
32	0.15	0.470	0.099	47 530	68 880	23.12	48.70
33	0.16	0.428	.....	39 920	61 010	25.50	56.80
34	0.16	0.449	.....	44 730	66 000	24.25	45.50
35	0.35	0.540	.....	51 490	83 640	15.62	30.70
36	0.37	0.530	.....	50 070	79 300	18.00	32.40
37	0.32	0.460	.....	54 760	85 330	17.12	30.10
38	0.31	0.560	.....	62 000	83 900	20.00	40.90
39	0.30	0.620	.....	57 500	82 400	16.80	31.60
40	0.30	0.570	.....	60 140	81 640	22.25	40.70
41	0.37	0.580	.....	57 460	84 410	15.87	32.20
42	0.33	0.590	0.098	62 200	89 980	20.75	38.40
43	0.39	0.610	.....	52 480	79 520	22.75	37.70
44	0.16	0.410	.....	46 650	66 210	28.12	56.40
45	0.16	0.400	0.080	47 030	66 920	24.75	56.40
46	0.36	0.520	.....	47 850	85 300	20.75	36.30
47	0.34	0.590	.....	52 400	79 500	16.75	36.70
48	0.39	0.620	.....	58 290	86 350	19.25	36.30
49	0.33	0.577	.....	54 000	84 000	19.37	31.60
50	0.39	0.610	.....	52 480	79 520	22.75	37.70
51	0.40	0.620	.....	51 680	80 000	21.25	37.70
52	0.35	0.570	0.106	57 200	87 200	18.50	30.00
53	0.30	0.530	.....	53 100	79 300	18.40	30.30

Rivet  
Steel.Rivet  
Steel.

## Steel made in Stationary Furnace. Accepted.

Number.	CHEMICAL ANALYSIS.			MECHANICAL TESTS.			
	Carbon, per cent.	Manga- nese, per cent.	Phos- phorus, per cent.	Elastic Limit, Pounds per square inch	Ultimate Strength, Pounds per square inch	Elongation in 8 inches, per cent.	Reduction of Area, per cent.
54	0.36	0.600	0.102	54 000	83 100	18.75	33.00
55	0.38	0.580	.....	52 000	81 000	20.62	34.20
56	0.32	0.560	.....	56 300	90 200	17.37	31.50
57	0.32	0.790	.....	53 320	87 530	19.00	34.40
58	0.35	0.620	0.094	53 500	83 900	19.20	36.80
59	0.32	0.530	0.103	57 500	84 800	20.00	33.30
60	0.32	0.510	.....	56 600	83 300	19.00	37.30
61	0.32	0.530	0.077	58 700	85 300	18.62	31.50
62	0.38	0.590	.....	57 700	87 600	16.25	31.30
63	0.35	0.640	.....	50 900	79 600	19.10	36.90
64	0.30	0.530	.....	51 900	79 500	19.50	33.40
65	0.30	0.490	0.096	54 000	80 200	16.75	36.90
66	0.35	0.580	.....	53 400	84 040	16.50	31.30
67	0.32	0.530	.....	51 130	83 600	16.25	32.20
68	0.30	0.500	0.085	53 400	78 900	16.70	33.70
69	0.36	0.590	0.088	58 270	86 500	19.50	31.20
70	0.30	0.570	.....	52 210	89 350	21.50	37.20
71	0.36	0.620	.....	58 440	80 200	18.50	31.50
72	0.31	0.610	0.086	58 590	85 780	16.75	33.40

## Steel made in "Pernot" Furnace. Accepted.

73	0.31	0.567	0.099	55 170	80 610	18.75	32.02
74	0.35	0.554	.....	57 610	87 630	18.00	35.00
75	0.32	0.585	.....	51 790	82 370	18.00	34.80
76	0.34	0.579	.....	51 240	81 750	18.60	40.59
77	0.34	0.617	0.098	52 070	86 130	18.12	39.00
78	0.38	0.600	.....	51 260	85 610	20.25	34.10
79	0.40	0.620	.....	52 350	83 720	19.87	35.80
80	0.30	0.610	0.099	50 040	79 670	20.75	38.70
81	0.38	0.480	.....	50 450	81 590	18.75	33.70
82	0.35	0.400	0.099	50 020	82 160	21.25	33.20
83	0.31	0.540	.....	51 400	81 560	19.50	33.00
84	0.35	0.400	0.099	50 020	82 160	21.25	33.20
85	0.38	0.592	.....	54 250	83 660	20.00	34.30
86	0.38	0.573	.....	50 870	82 210	22.00	33.40
87	0.36	0.511	.....	52 350	81 920	24.25	42.50
88	0.37	0.492	.....	51 260	80 340	19.25	36.40
89	0.37	0.490	.....	52 080	81 400	21.62	42.40
90	0.35	0.550	0.098	60 600	84 130	18.25	41.00
91	0.36	0.450	.....	58 760	81 050	17.50	37.60
92	0.39	0.598	.....	57 260	86 760	16.62	31.30
93	0.31	0.450	0.101	51 790	79 690	22.87	41.10
94	0.36	0.600	.....	51 060	84 520	19.00	31.30
95	0.35	0.555	.....	51 600	85 780	18.12	31.10
96	0.35	0.640	.....	50 930	79 540	20.50	36.00
97	0.36	0.620	.....	52 310	83 700	21.50	32.50
98	0.36	0.620	.....	49 990	81 090	20.00	32.80
99	0.41	0.640	.....	55 840	90 010	16.75	32.30
100	0.36	0.550	.....	51 370	81 850	18.75	34.90
101	0.40	0.600	.....	52 630	86 380	18.12	36.20
102	0.35	0.570	0.096	53 610	79 740	17.75	32.30
103	0.35	0.540	.....	49 540	80 390	21.25	39.10
104	0.35	0.510	.....	52 630	81 490	17.25	32.70
105	0.37	0.479	.....	55 480	89 210	18.00	35.70
106	0.35	0.500	.....	51 870	84 100	20.50	37.20
107	0.38	0.490	.....	52 610	80 990	15.62	32.00
108	0.32	0.600	.....	49 380	78 850	18.00	39.30
109	0.30	0.460	.....	52 220	79 050	19.50	44.50
110	0.35	0.567	0.074	55 080	81 600	21.00	30.40
111	0.35	0.650	.....	51 400	85 880	17.25	30.50

STEEL USED FOR PINS, MANUFACTURED BY CAMBRIA COMPANY,  
JOHNSTOWN, PA. Made in "Pernot" Furnace.

Number.	CHEMICAL ANALYSIS.			MECHANICAL TESTS.			
	Carbon, per cent.	Manganese, per cent.	Phos- phorus, per cent.	Elastic Limit, Pounds per square inch	Ultimate Strength, Pounds per square inch	Elongation in 8 inches, per cent.	Reduction of Area, per cent.
112	0.40	.....	.....	49 795	79 940	20.87	40.80
113	0.36	.....	.....	54 039	87 140	19.00	33.10
114	0.40	.....	.....	50 072	80 744	19.12	31.90
115	0.32	.....	.....	51 210	83 985	20.37	34.40
116	0.34	.....	.....	53 050	84 760	19.12	35.30

STEEL MANUFACTURED BY SPANG STEEL AND IRON COMPANY, PITTS-  
BURGH, PA.

Steel Made in Stationary Furnace. Rejected.

Number.	Percentage of Carbon.	Elastic Limit, Pounds Per Square Inch.	Ultimate Strength, Pounds Per Square Inch.	Elongation in 8 inches, Per Cent.	Reduction of Area, Per Cent.
1	0.28	55 540	77 590	20.00	49.9
2	0.29	51 650	71 860	22.00	45.1
3	0.27	47 160	76 350	21.25	39.0
4	0.28	50 930	75 670	22.50	49.1
5	0.29	49 000	73 400	22.12	48.4
6	0.30	49 405	76 980	21.00	40.5
7	0.31	57 790	78 360	21.25	40.1
8	0.30	52 920	76 760	19.50	33.3
9	0.30	53 750	79 130	19.12	42.8
10	0.30	51 650	78 910	19.38	44.3
11	0.29	48 530	74 310	20.25	44.6
12	0.29	49 790	74 290	19.38	36.0
13	0.42	53 480	94 490	12.00	11.6
14	0.30	52 910	78 360	19.37	41.9
15	0.33	44 430	70 830	23.12	40.7
16	0.26	45 670	76 360	19.00	40.1
17	0.34	46 780	77 810	17.75	32.7
18	0.30	50 440	76 840	22.00	36.9
19	0.34	48 630	81 190	16.87	32.8
20	0.30	46 660	73 180	20.87	42.3
21	0.34	47 640	74 870	19.00	38.2
22	0.33	49 010	77 340	21.87	49.7
23	0.27	57 610	97 490	14.62	23.7
24	0.31	49 140	80 160	16.87	36.2
25	0.35	48 630	79 310	18.75	35.2
26	0.35	48 150	80 630	21.50	46.4
27	0.43	54 260	87 100	18.12	30.2
28	0.33	48 020	75 940	22.25	41.2
29	0.35	57 920	91 440	19.50	36.3
30	0.29	50 260	78 060	18.87	33.9
31	0.42	47 640	74 630	19.50	40.0
32	0.39	56 630	86 870	16.62	30.6
33	0.42	56 980	91 920	16.00	21.7
34	0.43	60 590	95 960	16.25	21.1
35	0.40	56 920	91 340	17.12	26.9
36	0.51	55 940	95 130	6.87	7.8
37	0.43	50 110	88 380	11.25	16.0
38	0.35	48 240	76 860	18.50	32.5
39	0.30	47 460	77 540	21.37	39.9
40	0.28	46 880	76 330	21.50	35.0



## Steel Made in Stationary Furnace. Rejected.

Number.	Percentage of Carbon.	Elastic Limit, Pounds Per Square Inch.	Ultimate Strength, Pounds Per Square Inch.	Elongation in 8 inches, Per Cent.	Reduction of Area, Per Cent.
41	0.30	44 640	79 020	19.50	38.9
42	0.33	49 260	77 080	19.75	39.0
43	0.33	48 440	77 460	19.62	39.1
44	0.41	51 110	85 940	16.00	23.0
45	0.30	50 320	76 610	21.87	38.4
46	0.33	56 070	85 180	16.25	27.0
47	0.36	53 190	86 780	13.00	25.3
48	0.33	50 330	75 980	20.50	34.9
49	0.36	65 710	94 390	17.87	28.3
50	0.35	64 260	95 720	13.75	18.3
51	0.33	51 370	77 880	21.62	40.4
52	0.36	59 030	91 330	14.75	26.2
53	0.35	66 040	92 570	16.50	30.2
54	0.30	52 770	76 350	19.00	34.4
55	0.39	66 140	98 390	16.87	25.5
56	0.45	56 500	93 190	11.25	17.7
57	0.43	53 800	87 600	14.25	18.7
58	0.33	52 900	79 700	13.75	18.7
59	0.27	50 000	78 200	23.25	40.4
60	0.37	59 800	95 100	14.60	23.0
61	0.34	57 000	91 200	13.50	18.7
62	0.35	54 200	80 800	16.25	26.8
63	0.32	52 610	82 240	13.75	26.5
64	0.35	55 100	86 000	13.80	23.7
65	0.32	52 590	76 770	21.75	42.1

## Steel Made in "Pernot" Furnace. Rejected.

Number.	Percentage of Carbon.	Elastic Limit, Pounds Per Square Inch.	Ultimate Strength, Pounds Per Square Inch.	Elongation in 8 inches, Per Cent.	Reduction of Area, Per Cent.
67	0.31	46 400	75 820	15.00	32.8
68	0.28	48 660	74 150	21.25	46.2
69	0.32	48 530	76 750	21.26	43.6
70	0.31	44 910	70 420	19.50	45.7
71	0.28	44 790	73 570	21.87	43.4
72	0.31	50 260	78 800	17.50	36.2
73	0.29	50 330	76 960	21.87	46.6
74	0.42	55 990	90 570	23.50	16.5
75	0.35	56 290	89 080	8.75	8.8
76	0.36	45 520	74 990	20.62	36.9
77	0.30	47 160	73 140	20.62	44.3
78	0.36	49 540	77 420	20.62	55.3
79	0.38	47 160	76 350	21.87	36.5
80	0.34	56 950	92 250	15.62	24.9
81	0.43	51 510	85 420	15.50	26.8
82	0.32	51 370	82 950	12.50	25.4
83	0.25	57 320	85 870	16.62	23.8
84	0.32	50 000	82 750	12.50	28.5
85	0.30	46 920	74 810	21.62	38.8
86	0.25	41 230	64 950	22.50	49.8
87	0.31	45 690	74 970	22.37	43.5
88	0.31	53 680	78 310	14.87	34.3
89	0.33	50 998	78 108	18.00	36.4
90	0.34	44 840	74 650	21.62	41.2
91	0.30	48 700	74 620	22.75	36.9
92	0.31	51 400	75 550	22.87	46.7
93	0.35	46 900	72 880	22.25	36.2
94	0.33	47 990	73 102	20.75	38.5
95	0.45	53 310	92 620	12.75	14.5
96	0.33	46 580	79 700	16.25	28.7
97	0.34	48 750	88 140	15.00	22.5
98	0.36	56 420	91 660	15.12	29.5
99	0.35	48 830	77 950	20.25	37.6
100	0.32	50 040	76 780	23.00	38.6
101	0.30	53 190	86 780	13.60	25.3
102	0.43	50 930	82 390	13.12	24.2
103	0.45	50 930	85 990	12.25	18.8

## Steel Made in "Pernot" Furnace. Rejected.

Number.	Percentage of Carbon.	Elastic Limit, Pounds Per Square Inch.	Ultimate Strength, Pounds Per Square Inch.	Elongation in 8 inches, Per Cent.	Reduction of Area, Per Cent.
104	0.30	50 070	86 160	9.25	9.4
105	0.32	48 701	76 020	22.50	39.4
106	0.37	40 900	77 600	21.75	37.3
107	0.35	53 070	40 520	6.50	9.0
109	0.36	58 750	79 610	5.75	5.1
109	0.35	51 530	78 460	15.25	21.0
110	0.34	48 040	77 266	21.87	36.4
111	0.40	62 270	91 090	9.00	22.8
112	0.53	55 480	98 420	10.25	22.8
113	0.37	50 040	78 330	8.00	20.0
114	0.27	43 400	69 330	25.00	37.2

## IRON MANUFACTURED BY GRAFF, BENNET &amp; CO., PITTSBURGH, PA.

Number.	Material.	Area of Test Piece in square inches.	Elastic Limit, pounds per square inch.	Ultimate Strength pounds per square inch.	Elongation in 8 inches, per cent.	Reduction of area, per cent.
1	12" X 6" bar.....	0.7775	30 540	52 410	21.62	28.9
2	12" X 6" ".....	1.152	30 380	51 740	20.25	25.9
3	12" X 6" ".....	1.2773	31 710	51 440	17.50	21.1
4	12" X 7" ".....	0.8090	30 280	51 300	25.87	28.6
5	12" X 7" ".....	0.8043	29 220	47 250	16.75	32.8
6	12" X 7" ".....	0.6263	34 320	52 850	15.02	19.3
7	12" X 7" ".....	0.6277	34 250	53 210	21.62	23.9
8	12" X 7" ".....	0.7995	28 890	50 660	27.50	40.1
9	12" X 7" ".....	0.7885	27 250	47 560	18.75	20.4
10	12" X 6" plate.....	0.4990	30 460	47 500	13.12	20.5
11	12" X 6" ".....	0.6232	33 050	50 220	14.50	20.3
12	30" X 3" ".....	0.3705	31 850	48 300	14.00	19.1
13	30" X 3" ".....	0.3735	30 120	48 210	14.37	19.6
14	7" X 12" bar.....	1.396	29 730	49 050	21.50	27.3
15	7" X 12" ".....	1.4068	30 920	50 820	25.00	29.7
16	7" X 11" ".....	1.2670	30 380	49 330	23.00	25.0
17	7" X 11" ".....	1.278	30 520	49 090	22.75	25.2
18	8" X 11" ".....	1.250	28 000	49 200	29.00	38.4
19	8" X 11" ".....	1.255	28 680	48 840	24.00	27.5
20	8" X 12" ".....	1.0207	28 410	49 770	18.50	27.8
21	6" X 11" ".....	1.285	27 240	46 770	17.00	21.5
22	6" X 11" ".....	1.176	27 210	45 400	11.00	16.0
23	15" X 3" plate.....	0.7476	29 420	50 490	17.25	20.4
24	16" X 3" ".....	0.3761	30 580	50 410	10.50	8.5
25	16" X 3" ".....	0.3991	32 830	49 730	11.50	14.9
26	16" X 3" ".....	0.5030	30 810	50 790	13.00	15.4

## IRON MANUFACTURED BY ATKINS BROTHERS, POTTSVILLE, PA.

27	3 1/2" X 5" X 1/8" L.....	0.5575	26 200	48 200	17.12	19.0
28	" " " " ".....	0.6734	28 070	48 860	16.40	18.0
29	8" [42 lbs. per yard.....	0.231	29 000	55 410	13.25	28.0
30	" " " " ".....	0.221	29 410	53 400	12.50	17.0
31	10" [50 lbs. " ".....	0.265	29 260	54 740	14.00	17.0
32	9" X 1/2" plate.....	0.368	27 000	53 670	13.25	17.0

## APPENDIX No. 5.

## REPORT OF TESTING COMMITTEE.

NEW YORK, July 15th, 1884.

CHARLES C. SCHNEIDER, Esq.,

*Chief Engineer Niagara River Bridge Company.*

DEAR SIR,—On the occasion of the formal opening on December 20th, 1883, of the great Cantilever Bridge, built under your direction across the Niagara River, we were requested by you to act as a committee of engineers, to observe the tests made of the bridge. Various circumstances, among which may be enumerated the very unfavorable weather and the large crowd of visitors, disturbed the accuracy of the observations to such an extent that we did not feel prepared to make more than a very indefinite report, and requested an opportunity to make another examination on some later occasion. The observations made on the 20th of December indicated a maximum deflection of 6½ inches at the ends of the intermediate span under a maximum load.

On Monday, June 9th, 1884, the desired opportunity was afforded to us for a second examination, the bridge being placed at our disposal from 9 A. M. till noon, and the tests being conducted in a quiet manner, without the attendance of visitors. The test load consisted of two trains of cars loaded with gravel, each drawn by two locomotives. The train on the north track was drawn by Michigan Central locomotives Nos. 400 and 417, and that on the south track by locomotives Nos. 421 and 424; the weights of these locomotives were reported as follows :

	No. of Engine.	Weight of Engine.	Weight of Tank with Coal and Water.	Total Weight of Engine and Tank.
North track... }	400	88 800 lbs.	70 100 lbs.	158 900 lbs.
	417	88 000 "	77 700 "	165 700 "
Total .....	.....	176 800 lbs.	147 800 lbs.	324 600 lbs.
South track... }		84 000 lbs.	63 000 lbs.	147 000 lbs..
		85 000 "	53 000 "	138 000 "
Total .....	.....	169 000 lbs.	116 000 lbs.	285 000 lbs.

Each locomotive occupied about 55 feet.

The gravel cars had been loaded so as to make the weight upon them as nearly as possible a gross ton per foot ; the trains were long enough to cover the entire structure.

Rods had been set up at the six points designated on Plate LXIV accompanying this report; these rods being of wood, painted white, and with iron bases, which rested directly on the iron-work of the structure. The observations on the south truss were made with an astronomical

telescope with a  $2\frac{1}{8}$ -inch object glass of 30-inch focal length, fitted with cross hairs especially for this work, and which was kept fixed on a foresight. The observations on the north truss were taken with an ordinary engineer's level. The observers sighted on pencil points held by the markers, who marked the reading directly on the rods, from which, on the completion of the observations, the record was taken; the rods themselves remaining as a permanent record. Seven sets of observations were taken as follows:

Observation A: Head of train at west anchorage.

Observation B: Head of train at point 5 (west tower).

Observation C: Head of train at point 4.

Observation D: Head of train at point 3.

Observation E: Head of train at point 2 (east tower).

Observation F: Head of train at east anchorage (whole bridge covered).

Observation G: Cars removed west of point 2, leaving two engines and three cars on east cantilever arm.

The train on the north track was then allowed to go, and a set of observations was taken with a train on the south track only; these observations were as follows:

Observation H: Head of train at point 5 (west tower).

Observation I: Head of train at point 4.

Observation J: Head of train at point 3.

Observation K: Head of train at point 2 (east tower).

The train was then allowed to depart, and observations were taken with the bridge free from load.

The results of these observations are shown graphically on Plates Nos. LXXV and LXXVI accompanying this report.

These diagrams show slight changes at different points, generally not exceeding  $\frac{1}{4}$  inch; they are small in amount and irregular in character; they may represent the measure of the inaccuracy in the observations taken.

Neglecting these small irregularities, the results of the test may be summarized as follows:

The greatest observed deflection in the shore arms of the cantilevers was in the south truss, at the east end, under the conditions of observation G, being  $1\frac{1}{2}$  inches, and the greatest observed elevation of the same point, under the conditions of observation E, being  $2\frac{1}{2}$  inches, showing a total vertical motion in this point of 2 inches. The greatest observed deflections at points 3 and 4, being the river arms of the cantilevers, occurred in the north truss at point 3 (west end of river arm of American cantilever), under the conditions of observation E, being  $7\frac{1}{2}$  inches; the greatest elevation occurred under the conditions of observation G, being  $1\frac{1}{2}$  inches, showing a total vertical motion at this point of  $8\frac{1}{2}$  inches. The changes at points 3 and 4, that is, the river ends of the cantilevers, occurred under conditions which followed each other so rapidly that the change is readily noticed by a careful observer; these changes, however, are strictly in accordance with the laws of the structure, and as they should be.

The observations with a load on one track only were made for the purpose of determining how far the weight on a single track is carried by the two trusses. The distance between the centers of the trusses is 28 feet; the distance between centers of tracks is 13 feet, so that if no weight were transferred by the vibration rods and floor connections, 73

per cent. of the load on one track would be carried by the adjoining truss, and 27 per cent. by the opposite truss. Under the conditions of observation J, the deflection of the south truss was 4 inches and that of the north truss  $2\frac{1}{4}$  inches, showing that 66 per cent. of the load only was carried by the adjoining truss. Under the conditions of observation K, the deflection at point 3 was  $4\frac{1}{2}$  inches in the south truss, and  $2\frac{3}{4}$  inches in the north truss, which indicates that just 40 per cent. of the total load was carried by the north truss and 60 per cent. by the south truss. This indicates that one-eighth of the total load (73 p. c. — 60 p. c.) is transferred by the diagonals and floor connections, and taking the maximum load at half the weight of a locomotive, or 80 000 pounds, the amount of the weight which one set of diagonals may be called on to transfer is 10 000 pounds, which may produce a strain in the vibration rods of the central span of 17 400 pounds, which is equivalent to 11 200 pounds per square inch; this strain is within the limits of good practice. Rough observations showed that this transfer of weight from one truss to another was accompanied by a corresponding side movement in the trusses, and by a slackening of one of the diagonals; this is as it should be.

Observations taken at the first panel point from the west anchorage, at the connection between the inclined end-post and the bottom chord, showed a maximum depression of  $\frac{1}{8}$  inches under the conditions of observation B, and a maximum elevation of  $\frac{1}{8}$  inches under a special condition when the train covered the entire bridge east of the western tower, this observation being taken between observations F and G, thus showing a total vertical motion of  $1\frac{7}{8}$  inches at this point; the results of the observations at these points are given in detail on sheet marked Appendix A.

During observations B to G, an examination was made by Mr. Arthur V. Abbott of the elongations under strain of the two diagonals which unite on a single connection at the shore arm of the eastern cantilever; the result of these observations and the equivalent strains on an assumed modulus of elasticity of 26 000 000 are given in Appendix B. It will be observed that under those conditions in which the strains were anywhere approaching to a maximum the two bars worked very closely together, the differences not exceeding those noted by the gauges before observation D, which was the first condition under which any strain was to be expected from weight of moving load.

So far as could be ascertained, the bridge returned to precisely its original position when the loads were removed.

There are one or two matters to which we think your attention should be called.

The first is the condition of the anchorages. The anchor bolts at the south end of the eastern anchorage are a little slack, allowing a vertical motion of  $\frac{1}{8}$  inch in the pin at the head of the anchorage. The wall-plate castings on the west anchorage have never been properly packed, but are set up on a series of small shim plates, a slight disturbance of which would leave a play in the anchor rods. The anchor rods of all the anchorages should be carefully adjusted so as to make sure that they are all uniformly tight and there is no motion in the anchor pins. The bearings of the castings at the west end ought to be thoroughly packed so as to prevent any future disturbance; before this is done, an observation ought to be made of the play of the pins in the west anchorage, which could easily be done by removing the nuts from the pins and watching them when any ordinary train crosses.

*Second.*—The bridge has been standing long enough to make it important that a competent man should examine the condition of all the adjustable parts in detail, and, if they are out of adjustment, correct them; it is better that this should not be done at all than that it should be done by any one not familiar with this class of work.

*Third.*—The condition of the slopes of the chasm under the shore arms of the cantilevers is such that more or less small stones roll down from time to time, some of which have struck against the iron-work; they have done no harm, but heavy stones might do serious injury, and it would be expedient to clean the slopes and even them off in good shape around the masonry.

In conclusion, we wish to say that the behavior of the bridge under the test made on June 9th (and also under the test of December 20th, so far as the latter could be observed) was a satisfactory one, and we consider the Niagara Cantilever Bridge a successful example of what may be termed a novel system of construction. While recognizing the fact that cantilever bridges have been built before, and have been advocated by able engineers for many years, we think it fair to say that we know of no other structure in which the distinguishing features of the cantilever have been so fully carried out in the details which exemplify American principles of construction. We congratulate you upon the successful completion of the structure.

Respectfully yours,

GEO. S. MORISON,  
THOMAS RIDOUT,  
THEODORE COOPER,  
CHARLES MACDONALD.

## APPENDIX A.

Observations on west end of Cantilever Bridge taken at bottom of end-posts.

## LOAD ON BOTH TRACKS.

*South Side.*

Observation A.—Bridge unloaded.

"	B.—Depression,	$\frac{13}{8}$ "	below unloaded bridge.		
"	C.—	0"	"	"	"
"	D.—Elevation,	$\frac{3}{16}$ "	above	"	"
"	E.—	$\frac{9}{32}$ "	"	"	"
"	F.—	$\frac{7}{16}$ "	"	"	"
"	Special.—	$\frac{5}{8}$ "	"	"	"
"	G.—	$\frac{1}{16}$ "	"	"	"

*North Side.*

Observation B.—Depression,  $\frac{13}{8}$ " below unloaded bridge.

"	C.—	$\frac{13}{16}$ "	"	"	"
"	D.—Elevation,	$\frac{1}{4}$ "	above	"	"
"	E.—	$\frac{1}{8}$ "	"	"	"
"	F.—	$\frac{1}{16}$ "	"	"	"
"	Special.—	$\frac{9}{16}$ "	"	"	"
"	G.—	0"	"	"	"

## LOAD ON ONE TRACK.

*South Side.*

Observation A.—Bridge unloaded.

"	H.—Depression,	$\frac{11}{16}$ "	below unloaded bridge.		
"	I.—	$\frac{1}{16}$ "	"	"	"
"	J.—Elevation,	$\frac{1}{8}$ "	above	"	"
"	K.—	$\frac{1}{8}$ "	"	"	"

*North Side.*

Observation H.—Depression,  $\frac{7}{16}$ " below unloaded bridge.

"	I.—Elevation,	$\frac{1}{16}$ "	above	"	"
"	J.—	$\frac{1}{8}$ "	"	"	"
"	K.—	$\frac{1}{4}$ "	"	"	"

## APPENDIX B.

Results of observations on the tension members of the Niagara Cantilever Bridge, made at the test of June 9th, 1884.

Two verniers were used, each extending over 5 feet of the bar under examination, and reading to .00001 foot. The following readings are reduced to millionths of a foot for 1 foot of bar, the observations being lettered to correspond with those on Diagram No. 2.

	Vernier A. (On Long Bar.)	Strain. Pounds per Square Inch.	Vernier B. (On Short Bar.)	Strain. Pounds per Square Inch.
Observation <i>B</i> .....	12	312	6	156
" <i>C</i> .....	14	364	16	416
" <i>D</i> .....	112	2 912	130	3 380
" <i>E</i> .....	236	6 136	256	6 656
" <i>F</i> .....	56	1 456	128	3 328
" <i>G</i> .....	-40	-1 040	-160	-4 160
Bridge unloaded .....	0	0	0	0

The modulus of elasticity is taken at 26 000 000.

## APPENDIX C.

Results of observations on movements of the anchorage connecting links at the south side of the American anchorage. Length of links 36 inches.

Observation.	Horizontal Movement Towards the River.	Upward Movement.
<i>B</i> .....	0	0
<i>C</i> .....	0	0
<i>D</i> .....	$\frac{3}{16}$	$\frac{3}{16}$
<i>E</i> .....	$\frac{3}{16}$	$\frac{3}{16}$
<i>F</i> .....	$\frac{1}{16}$	$\frac{1}{16}$
<i>G</i> .....	0	0



# Arrangement of Targets.

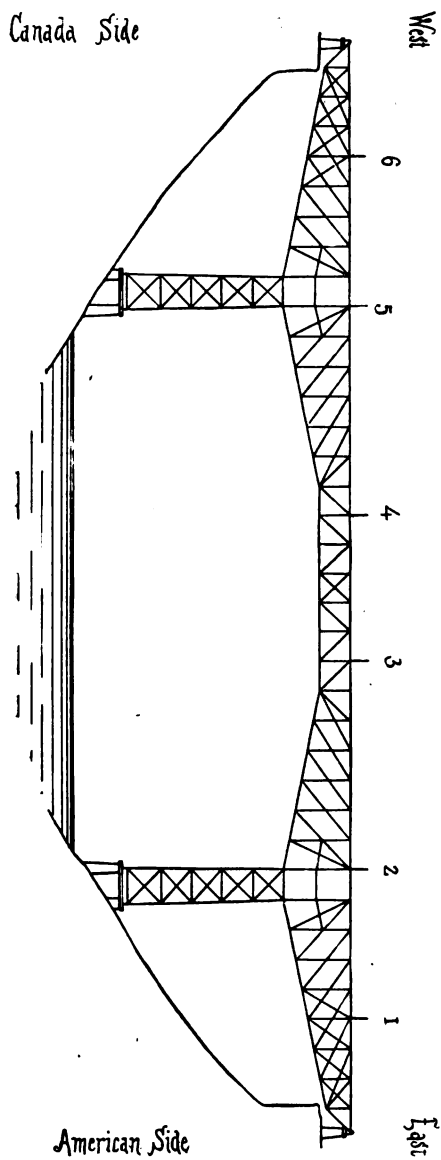
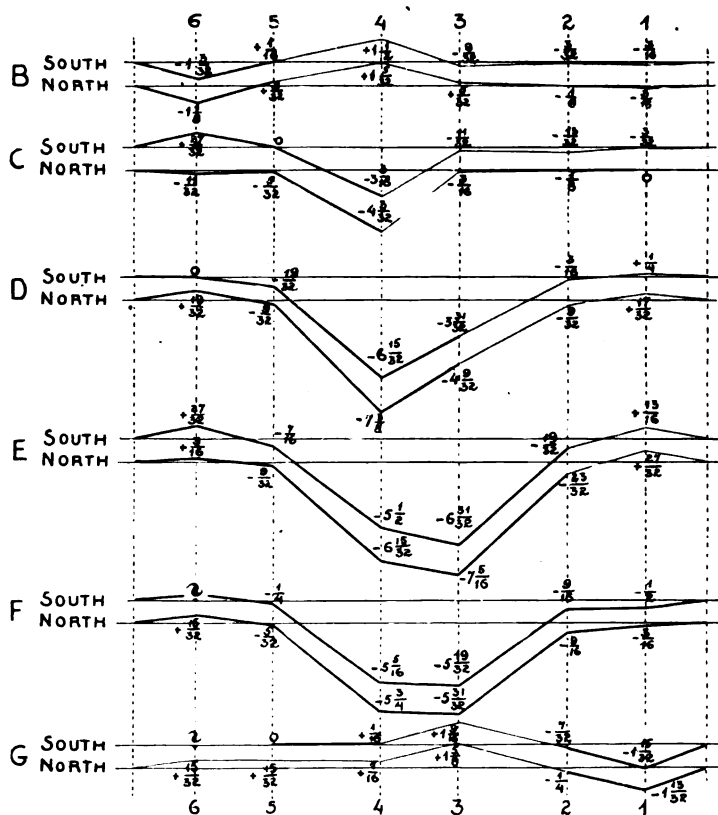


PLATE LXIV.  
 TRANS. AM. SOC. CIV. ENG'RS.  
 VOL. XIV, No. 317.  
 SCHNEIDER ON  
 CANTILEVER BRIDGE  
 AT NIAGARA FALLS.

DIAGRAM No2.

DEFLECTIONS OF NIAGARA CANTILEVER  
BOTH TRUSSES LOADED WITH TRAINS EXTENDING OVER SPACES  
INDICATED BY THE HEAVY LINES.



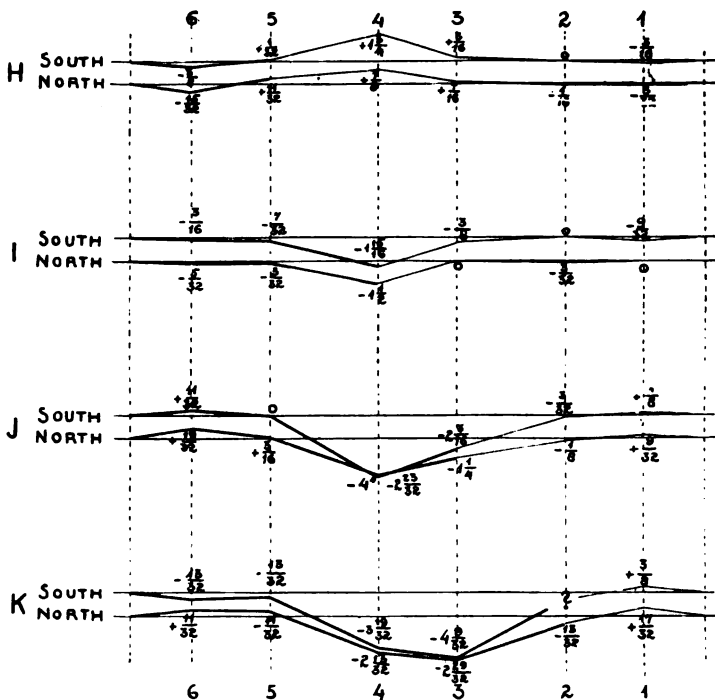
HORIZONTAL SCALE, 1 INCH = 150 FEET. VERTICAL SCALE, 1/6 FULL SIZE.

NOTE. THE LINES MARKED 1, 2, 3, 4, 5 & 6 CORRESPOND WITH THE TARGET NUMBERS IN DIAGRAM No. 1.

DIAGRAM N<sup>o</sup> 3.

# DEFLECTIONS OF NIAGARA CANTILEVER

SOUTH TRUSS ONLY LOADED, TRAIN COVERING SPACES  
INDICATED BY THE HEAVY LINES.



HORIZONTAL SCALE, 1 INCH = 150 FEET. VERTICAL SCALE, 1/6 FULL SIZE.

NOTE THE LINES MARKED 1, 2, 3, 4, 5 & 6 CORRESPOND WITH THE TARGET NUMBERS IN DIAGRAM N<sup>o</sup> 1.



SCH



## DISCUSSION ON THE CANTILEVER BRIDGE AT NIAGARA FALLS.

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DISCUSSIONS PRESENTED AT MEETING OF MAY 20TH, 1885.

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JAMES CHRISTIE, M. Am. Soc. C. E.—The propriety of using such hard steel (.34 to .42 carbon) in bridge members is at least questionable, and recent experience with this metal does not dispel the doubt. There is always a probability of portions of the steel becoming partially tempered or unequally annealed, and thus, in the mass, possessing very different physical properties from that exhibited by small specimens.

The tensile tests, as recorded by Mr. Schneider, show strength at the elastic limit varying from 48 000 to 66 000 pounds per square inch, or an elevation of nearly 40 per cent. of the strongest over the weakest specimens, and this in steel so carefully selected that nearly 60 per cent. of the heats were rejected. As steel is known to be equally variable in compressive elasticity, we cannot expect harmonious resistance from a single member, possibly composed of a number of such unequal parts, even if the material is capable of surviving manipulation in the workshop without deterioration. In the case of the Glasgow Bridge steel column, tested at Watertown, the member failed under a pressure one-third less in amount than would be expected, judging by tests of its constituent parts, and 20 per cent. less than the compressive resistance of columns made of iron channels and having the same proportion of length to section.

The soft steels seem to be of more uniform tenacity than the hard steels; the records of a number of tests of .10 to .12 carbon steel show a tensile resistance at the elastic limit varying from 35 000 to 40 000 pounds per square inch, or only 14 per cent. increase of the highest over the lowest.

The soft steel bears manipulation in the workshop with less probability of injury than hard steel, and I know of no evidence that it has

exhibited inferior resistance in built-up sections, as compared with the individual parts.

The specifications allow an increase of load of 37½ per cent. for steel over iron compression members, which is wrong in principle, as the ratio of section to length must be considered; and if short steel columns should be proved trustworthy under greater loads than similar members of iron, then, in applying Gordon's formula to steel, all the numerical co-efficients used for iron must be changed, just as they have to be changed when substituting wrought for cast-iron.

Mr. Schneider is to be commended for the pains he has taken to present to the Society in such clear and abundant detail all the facts connected with this bridge, which will stand on record as the pioneer of its class.

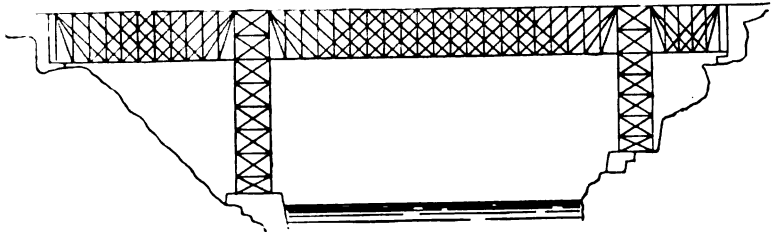
It appears to me that the omission of diagonals in the tower panel, while fixing definitely the reactions at the tower, might yet be open to objection under certain circumstances. If, for example, a heavy train should be suddenly stopped at mid-structure by the brakes, the resulting strain would be transferred in an irregular way to the anchorages of the cantilevers, already under strain from direct action of the load.

T. C. CLARKE, M. Am. Soc. C. E.—It may be interesting to the Society to hear how the problem of bridging the Niagara River was treated some ten years since.

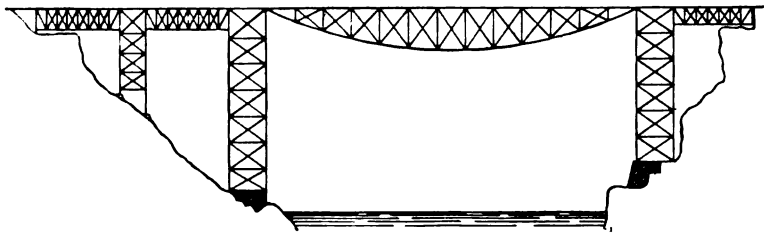
In the winter of 1874-75, I was requested by the Manager of the Great Western Railway of Canada to report upon the best mode of construction, necessary time required, and cost of a double-track iron bridge across the Niagara River, near the present suspension bridge. The site had been selected by John Kennedy, M. Am. Soc. C. E., Chief Engineer of the Great Western Railway, and I was furnished by him with a plan and section of the Niagara River. The site selected was 300 feet below the present suspension bridge, instead of above, as the present cantilever is built. The distances across the water and at the top of the cliffs were such that it was deemed prudent to locate the masonry piers below 430 feet apart, and the total length of the bridge was 825 feet, and the height above water 235 feet. Borings on the Canada side showed rock at a depth of 12 feet under the proposed dwarf piers at the water edge.

I reported in May, 1875, that I had investigated four different plans of construction, and advised the adoption of No. 4.

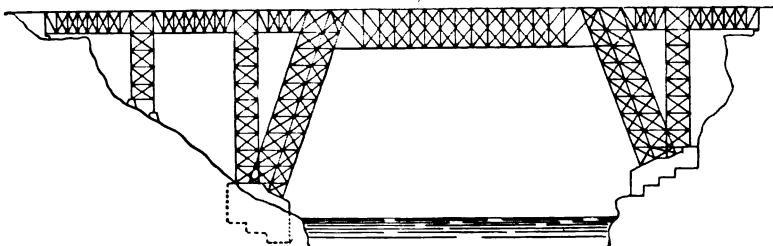
1. The first plan discussed was a continuous girder, to be built by corbelling out from each end, until the parts met in the center, with rollers on each pier. The estimated weight of this was 3 200 tons.



2. The second plan was three discontinuous girders. The loose central span was to have been erected on temporary cables. Reducing the weight of these cables to their equivalent value in tons—the weight of this design was 2 840 tons.



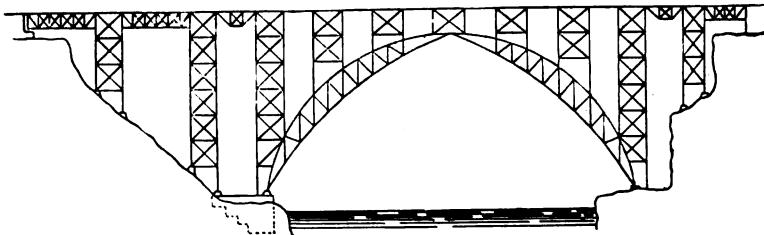
3. The third design was similar to No. 2, except that the length of the central span was reduced from 430 feet to 300 feet by leaning towers, somewhat like the false works of the Attock Bridge, in India, lately illustrated in *London Engineering*. The estimated weight of this design was 2 590 tons. This design is only eclipsed in ugliness by Max Am Ende's bridge in South Africa.



4. The recommended design was a braced arch, hinged in the center



and at the springing. The clear span was 430 feet, and the height of versed sine 175 feet. The arches were to have been erected by corbelling out as was done at St. Louis. The whole design was very similar to that since erected over the Tagus, in Portugal. The estimated weight of this design was 2 320 tons, and the time required fifteen months. All these designs were intended to carry two trains of coal cars, headed by one locomotive of the heaviest type.



In comparing this design with the bridge as actually built, it will be seen that the latter is simpler in general design and in its details; that this allowed of its being constructed and erected in half of the time estimated for the arch; and that the changes of lengths of parts due to changes of temperature are better provided for. The weights of material are about the same, but the arch piers are 430 feet apart, while those of the cantilever are 480 feet apart. In short, the cantilever design is better in every respect than the arch as designed by me.

Without entering into the delicate question of priority of design, I will say that the first bridge involving the cantilever principle (in its erection, certainly) was built by C. Shaler Smith, M. Am. Soc. C. E., over the Kentucky River in 1876-77, and that its successful erection without scaffolding solved all the difficulties which were subsequently met and overcome at the Niagara River.

The Niagara River Bridge is of better design than that at the Kentucky River, inasmuch as it makes its piers fixed points, and allows its trusses to expand and contract in each direction from them as centers, while the Kentucky River Bridge requires a movement of its piers, and the necessity of rollers under them, to provide for the differences of lengths due to changes of temperature of its central span.

CARL GAYLER, M. Am. Soc. C. E.—The results of the tests of the Niagara Cantilever Bridge, observed June 9th, 1884, and published as

an appendix to Mr. Schneider's paper, read March 4th, 1885, present some interesting features.

The full load on both tracks of the middle span of 470 feet length (see Plate LXV) bends the ends of the river arms of the cantilevers down about seven inches (*E*), and a full load on the shore spans alone raises the same ends on an average one inch and a half (*B* and *G*)—a total vertical movement of these points of eight and one-half inches thus being produced.

The side spans of 195 feet length deflect under their load over one inch (*B* and *G*), and are bent upward, if the middle span is loaded, about three-quarters of an inch.

As the test loads have not been heavier than those generally applied to important railroad bridges and viaducts after their completion, an inquiry into the causes of these great deflections appears natural.

It has been known that the cantilever bridges built on the system called from their inventor "Ordish Cantilever Bridges," have shown considerable deflections. For instance, a roadway bridge built by Ordish in Prague, across the Moldau, with a center span of 400 feet length, deflected seven inches under a test load of about forty pounds per square foot of roadway, and the explanation has generally been that this is due to the extension of the exceedingly long tensile members under strain, together with the circumstance that on account of the effects of temperature the same cannot be made continuous. But this explanation does not apply to the new system of cantilever bridges introduced by Mr. Schneider, first on his bridge over the Fraser River in Canada, and then on the Niagara Bridge. The new system has the advantage over the Ordish system that, instead of long cables over the top of a high tower, a straight truss is introduced, thus reducing the length of the tensile members to a minimum. But whatever is gained in this manner is lost again by the fact that the deflection of the center span depends on the stiffness of the side spans, whilst Ordish transfers the strains directly to the anchorage.

An illustration of the effect produced by supporting a truss in such a manner that its weight and its loads are transferred on and resisted by the strength of the truss of the adjoining span, is presented in the tests of the Kentucky River Bridge.

The trusses of the middle span of the Kentucky River Bridge (as will be hardly necessary to explain) extend beyond each pier, thus

forming cantilevers. The trusses of the side spans rest with one end on the abutments, their other end being supported by the projecting ends of the trusses of the middle span. It would naturally be supposed that a full load on the side spans would show the greatest deflection near the middle of their length, as the length of the projecting arms of the middle trusses is only 75 feet, and the length of the side trusses 300 feet, yet the tests show the maximum deflection at the ends of the cantilevers, this being caused by the bending upward of the trusses of the middle span.

The test loads of the Niagara Bridge, as graphically represented on Plate LXV, though not heavier than test loads generally are, are certainly far in excess of the ordinary duty of the bridge, but it has to be borne in mind that the diagram gives the deflections for the different positions of the trains whilst the latter stand still; they do not represent the vibrations and sudden changes which take place under a moving train.

A train passing over the bridge, even at a moderate speed, has in less than one minute bent each of the cantilever arms downward and upward; it has strained and released again each set of anchor bolts separately; it has thrown the intermediate span of only 120 feet length out of its horizontal position, first raising its nearest end one inch (*H*, Plate LXVI), then pressing the same end down three or four inches (*J*, Plate LXVI), and repeating the same motions at the other end. That the time in which the strains change and are partly being reversed, and the rapidity with which the movements of the different members take place, are important factors in the consideration of the effects of the loads, cannot be doubted.

I have looked over these tests with the greatest interest, as similar effects of moving loads have come under my daily observation on a viaduct which was completed last fall in St. Louis, and for which I had designed the main spans as cantilever trusses.

There is one point for which I could find no explanation in the test or details.

The cantilevers have full play to expand and contract under changing temperatures from each pier towards the river and towards the abutments, but as there is no diagonal bracing between the posts above the piers, I am at a loss to see what should prevent a horizontal movement of the superstructure. The tests (see Appendix C) show that a mere load-

ing of the bridge causes horizontal movements of the shore anchorage links. What would have been the effect of such a test as the one applied to the Kentucky River Bridge, *i. e.*, a train entering the bridge at full speed, and suddenly brought to a halt by means of the brakes?

The Niagara Bridge is undoubtedly the boldest and best cantilever bridge in existence, both in design and execution, and it has, in this respect, well deserved the world-wide reputation which it has gained. Its shortcomings are simply those of the system.

It is true that this system has made the erection possible without the use of any additional supports for the middle span whatever, but the success of the erection of the St. Louis Bridge, and particularly the completion of the Kentucky River and Minnehaha Bridges by C. Shaler Smith, M. Am. Soc. C. E., under analogous circumstances, prove that obstacles as great as those encountered in crossing the Niagara River near the Falls can be overcome without resorting to a system which, even under the hand of the most skillful engineer, is not perfect.

J. D. HAWKS, M. Am. Soc. C. E.—I do not care now to discuss this subject in detail. I send, however, a copy of a report made to me last winter by the builders of the bridge, showing its condition at that time. I am having a plank walk laid on each lower chord to facilitate examination of the structure. I propose to have a daily inspection made by one of the gatemen who worked on the bridge during construction. Our bridge engineer, Mr. Benjamin Douglas, spends one day each two weeks at the bridge. In addition to this, I propose to ask some prominent bridge company to make an examination, say every six months. I should be pleased to furnish any Member of the Society facilities for looking over the bridge at any time on application.

BUFFALO, N. Y., January 2d, 1885.

J. D. HAWKS, Esq.,

*Chief Engineer Michigan Central Railroad,*  
Detroit, Mich.

We have just finished a very careful inspection of the Cantilever Bridge. Inclosed please find report made by our Superintendent of Erection.

Very truly yours,

UNION BRIDGE COMPANY,

By GEO. S. FIELD.

GENTLEMEN,—I have spent five days with one bridge man inspecting the Cantilever Bridge over the Niagara River.

We examined every rod, section, joint, anchorage and expansion point, and found nothing that needed adjustment or repairs. We tried the line over the top, using the old original center tacks and found the line perfect, showing that foundations and piers are as when first finished, and the lateral adjustment has not changed. When the length was taken for the center span the thermometer stood at 60 degrees; when I (one year later) took the measurement, it stood at zero.

I find the suspension pins one inch from center of expansion slots, showing there has not been the slightest movement of any part of the structure except that due to change of temperature.

S. V. RYLAND.

C. A. MARSHALL, M. Am. Soc. C. E.—I notice that no mention is made in the paper of reaming or otherwise removing the injury done to steel by punching. The specifications simply say the holes shall not be punched more than  $\frac{1}{16}$  inch larger than diameter of rivet to be used, and presumably no more reaming was done than was necessary to pass the rivet through where the punched holes did not fit. I figure that the unit stresses used to enter the column formula vary from about 14 000 to 14 700 pounds per square inch, so that apparently no reduction of the usual allowed stresses has been made on account of this departure from the prevailing treatment of steel for this kind of work and grade of steel.

Now I do not wish to be understood as saying that this constitutes an error of judgment in proportioning the structure, but on a work of this magnitude it is much to be regretted that such a deviation from common practice in a point often insisted upon as important, should be made without tests to substantiate the correctness of the position assumed. The writer would be inclined to believe that the matter of reaming has little influence on the strength of heavy struts such as here used, yet such struts usually fail by buckling somewhere. The stress determining failure is partially bending, whether local or general, and there is certainly room for doubt on the question of whether the full value of the material can be developed without the reaming. A few hundred dollars spent in testing sample columns of the same material (like that used in the bridge) differing only in having holes punched full size in one set and punched and reamed in the other, would certainly have thrown much light on the subject, and have largely enhanced the worth of this bridge considered as an engineering achievement or example of enlightened construction. If we can get full value out of steel of 80 000 pounds tensile strength without reaming, then, as a matter of economy,

let us save the reaming. It is safe to say, from published tests, that the net tensile value of steel of the mildest description may be developed in a riveted joint equal to that of a plain specimen, whether the holes be punched or drilled. Hence, almost certainly, in reckoning for compression, the difference between reaming and not reaming need not be taken into account for that kind of steel. Can Mr. Schneider assure us that the same holds true for the steel he used?

Jos. M. Wilson, M. Am. Soc. C. E.—Mr. Schneider's paper is exceedingly interesting, and it would be well if just such papers on all the important works of our country could be presented to the Society in so complete a shape. The matter of foundations seems to have been unique in its way, and great credit is due for the admirable disposition of the peculiar conditions of the case. A suggestion might be made as to whether it would not be well to remove some of the stone now on the slope above and behind the pier foundations so as to avoid any possible risk, even in the far future, of a shoving or pushing forward of the whole mass by a glacier-like action, and thereby a disturbance of these foundations *en masse*.

In my own practice I am using considerably heavier loading than has been taken in this case, taking a typical consolidation engine with 12 000 pounds on the front pair of wheels and 24 000 pounds on each of four pairs of drivers; also a typical passenger engine with 16 000 pounds on each of the two front pairs of wheels and 40 000 pounds on each of two pairs of drivers, the tender having 32 tons on four pairs of wheels in either case, and the train load 3 000 pounds per foot lineal of track. I think my experience justifies such loading. The tendency is towards still heavier loading in the future, and there has never yet been a standard established that has not been exceeded. There are now cars in use on the Pennsylvania Railroad that give a weight of 3 000 pounds per lineal foot of track with standard loading, and they are often overloaded considerably in excess of this. The formulæ for limiting maximum strains as deduced from Weyrauch, are somewhat different from my own practice and give somewhat higher results. I hope to be able to explain myself further in this connection at a future time.

My preference would have been for wrought-iron shoes under the tower posts. My experience with castings in such locations is unfortunate, although in a structure of size like the present, the effects of impact are not nearly so great as in smaller bridges.

The requirements in the specifications for steel seem rather high, and my preference would have been for a little lower quality and a lower limiting resistance. I do not pretend to have very much experience in this direction, and what I have had has not been very favorable, but I acknowledge the great value of steel in long-span structures when its quality can be depended upon.

FREDERICK H. SMITH, M. Am. Soc. C. E.—In the Testing Committees, Report, appended to Mr. Schneider's valuable paper, the observations made with single-track loading show different deflections at the four corners of the intermediate span, and as the train advances these deflection differences vary among themselves, and thereby impose a twisting moment upon that span.

The same report notes a side movement of the trusses, accompanied by the slackening of one of the diagonals. This side movement must develop in opposite directions at the two ends of the intermediate span when trains are entering that span at diagonally opposite corners.

The twisting moment imparted to this span by one or both of these movements may or may not be serious, and as Mr. Schneider has given us the first double-track cantilever bridge, I should like to ask him whether this twisting action has been made a subject of expert observation, and if so, with what result.

GEO. S. MORISON, M. Am. Soc. C. E.—There is one point which I think the author did not bring out in his paper as distinctly as he might have done, and that point relates to the foundations. The method of the foundations was somewhat criticised while the work was in progress. What I wish to bring out especially, is that these foundations are built on masses of very large rocks which have fallen from the strata above, and which owe their stability entirely to gravity. These large rocks have been termed boulders, which is an unfortunate word, and does not indicate the real character of the rocks. They are very large, some of them as large as a car; they have not been moved, but have simply fallen from the higher strata of rock above, when the softer strata below were undermined. The general opinion of those who examined these foundations was that these large stones, whose position was perfectly stable—that stability being due to gravity alone, with no aid from the cohesion which stiffens a material in position—were much safer for the founda-

tions of the bridge than a stratified rock would have been, unless that stratified rock extended continuously across the river. Their stability was well established, and no new disturbing element would change the conditions. On the other hand, a stratified rock which did not extend continuously across the river might become unsafe by the washing out of softer strata below it, and the rock might continue to support the weight of a bridge until the undermining had progressed to such an extent that the breaking of the rock brought about a sudden catastrophe. A continuous layer of hard rock, extending in its natural position across the river, would be preferable to the foundations which were found, but next to it, this loose mass of enormous stones is the best thing that could be had.

HENRY W. WILSON, M. Am. Soc. C. E.—The situation at Niagara which was selected for the new railroad bridge appears peculiarly adapted to a cantilever design, and Mr. Schneider seems to have worked out his problem with great ability. I have been considerably interested in this type of structure lately, and have looked over the present paper with great interest. I have taken the opportunity of examining pretty carefully into the details of the design, so far as the small scale drawings submitted will allow, and they appear to have been worked up very thoroughly, and with full consideration of the latest recognized theory of the action of forces, and the adaptation of details to resist them. Such matters as the question of pin moments, etc., appear to have been all well considered. While the assumed loading is not as heavy as I have been adopting, yet this is a question which was regulated by the specification, and, of course, the work has been carried out in accordance therewith.

In the designing of the floor system I should have preferred having the longitudinal track girders, designated stringers in the paper, directly under the rails, and in the cross girders or floor beams I would rather have had the vertical stiffeners closer together. I am aware of the wide difference of opinion among engineers concerning the precise duty which these stiffeners exercise in a plate-girder, and I merely give this, therefore, as an opinion.

THEODORE COOPER, M. Am. Soc. C. E.—The pioneer cantilever



bridge of large dimensions was the Kentucky River Bridge, designed and built by C. Shaler Smith, M. Am. Soc. C. E.

While less bold in conception and in untried principles than the above, the Niagara Cantilever Bridge has excited a far greater public interest, from its location over the world-renowned chasm of Niagara River and at a center of public travel.

For engineers, however, the interest in any new structure centers upon the adaptation of the design to the purpose and location, and upon the details of its execution and final operation. It is a matter of congratulation for the Society that Mr. Schneider has so fully and clearly given us an account of his design and its execution.

He has clearly stated the principles governing him in the selection of this kind and form of bridge; facility of erection, and a positive determination of the strains developed in each member. The avoidance of ambiguous strains in the trusses, by omitting the diagonal members in the panel over the towers, is worthy of approval. The concentrating the two systems of diagonals into a single link at the ends is in the same direction of positiveness of strains. I do not approve of double intersection trusses where it is possible to avoid them. It is very doubtful in my mind if the strains in the usual double intersection truss can be determined within 20 per cent. Mr. Schneider, by the use of the coupling link, has reduced the doubt to a minimum. In connecting his floor beams rigidly to the vertical posts, I think he has violated somewhat his desire for exactness of strains. In a double-track bridge, with one track only loaded, the trusses will deflect differently, producing in a rigidly attached floor beam indefinite and often excessive strains. This is partly prevented in this structure by the diagonal sway rods, which tend to hold the trusses parallel.

The crudity of the specifications, for which Mr. Schneider disclaims the responsibility, would be more noticeable if it were not of such frequent recurrence.

Why Launhardt's formulæ should be used for steel members and not for iron, would appear more mysterious if we did not know, from everyday experience, that the average specification is a compilation, by non-experts, of glittering generalities picked up here and there without any intelligent conception as to their intent or meaning; frequently contradictory, and always subject to various interpretations.

The rolling load consists of two Mogul engines (called consolidation in specifications) with a train of one ton per lineal foot.

This is far less than is adopted by the roads that have profited by past experience, and are looking ahead to increased locomotives and rolling stock.

Mr. Schneider has not, however, been confined to these specifications, for he has designed his floor system, the most important part, for a heavier rolling load; and has made variations in other parts of the specification.

The Central Bridge Company deserve credit for their part of this work—the execution of the contract, and for giving even a better structure than might have been made under the specifications.

The modern development of the cantilever, in my opinion, takes its departure from the completion of the St. Louis Bridge.

The necessities of erection at the site of this bridge over the Mississippi River at St. Louis, required the work to be supported from above by the use of suspension cables formed of ordinary bridge links. As the work neared completion, each river pier supported two enormous cantilevers 245 and 254 feet long; which, with the thickness of the piers, made a total length from end to end of 533 feet.

The success of this method of erection, by building out from each support a self-sustaining structure, established the security and facility of thus erecting large spans over chasms and rivers, where false works below were impossible.

The next step was to make the bridge self-supporting without the use of cables.

Mr. C. Shaler Smith, M. Am. Soc. C. E., soon thereafter accepted the results of this lesson, and constructed his Kentucky River Bridge by making it upon the cantilever principle, self-sustaining beyond the shore arms. For boldness of design and execution, it will always stand unequalled. Later structures have this additional experience to aid their development.

Cantilever bridges of moderate spans, about 500 to 600 feet, have no special merit aside from the facility of erection at certain localities. The same spans can be built as separate structures with the same or less material. The absolute necessity of erecting them, as here at Niagara, without false-works, or to save the expense of the false-works at deep gorges or over deep rivers, makes their use justifiable at such

points. I assisted John C. Goodridge, Jr., in measuring the current velocity of the Niagara River at the site of this bridge. A taffrail log which had been tested over a standard course of one nautical mile was used. We obtained the following readings for the central surface current:

First mile run out at	4	minutes	20	seconds,	
Second " " " "	8	"	0	"	
Third " " " "	12	"	30	"	

which reduced to statute miles, gives a velocity of 16.3 miles per hour.

In endeavoring to take the velocity at other points across the river, the log vane was cut off and lost, thus discontinuing the tests.

CHARLES MACDONALD, M. Am. Soc. C. E.—It is, perhaps, a little unfortunate that the word boulder has been used. There are really no such things as boulders in the Cañon of the Niagara. The masses of rock which have been precipitated to the bottom of the gorge by reason of the undermining of a soft stratum, have not been moved since the original denudation, and the monolithic foundation which has been judiciously distributed over them, is an evidence of engineering boldness and skill, equal, if not superior to the conception involved in the novel and elegant superstructure itself.

It would not be strange were there found to be some evidences of crudeness in detail. When it is remembered that the whole structure from foundation to completing-rail was executed in such an extraordinarily short time, the wonder is that there remains so little upon which adverse criticism can be made. If it had been possible to utilize steel to a greater extent, a more compact arrangement might have been made in the top chords over the piers, but the shortness of the time in which the work was required to be completed, doubtless prevented the manufacture of steel eye-bars for that purpose.

It is questionable whether it would not have been better to have increased the length of suspended span from 120, to say 140 or 160 feet, with a view to a reduction of the deflection under a central load. In the Louisville and New Albany Bridge, now in process of construction, Mr. Hemberle and myself have suggested 160 feet suspended spans in clear openings of 480 feet, but until this arrangement is tested it will be best to defer judgment.

THEODORE COOPER, M. Am. Soc. C. E.—When the commission, of

which I was a member, made an examination of these foundations, we found but little knowledge of the slope of the river bed beyond a short distance from the water's edge. I then suggested that, owing to the swiftness and strength of the current, an approximate idea could be had by securing a heavy boulder to a line by a ring bolt and then rolling it into the stream. By trying this and also using a heavy sledge tied to a line, the conclusion was drawn that the slope was continuous for about 50 to 60 feet beyond the water's edge. At about that point there appeared a sudden drop as if there was a break in the strata.

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DISCUSSIONS PRESENTED AT THE ANNUAL CONVENTION, JUNE 24TH, 1885.

F. COLLINGWOOD, M. Am. Soc. C. E.—The general discussion upon Mr. Schneider's paper on the Cantilever Bridge at Niagara has dwelt upon the points involved in location, foundations, form, strains and general details, but nothing has been said about the specifications. As these bring up the subject of the testing of steel for structures, it seems worth while to see how they compare in this with present English and European practice. Mr. Dorsey, in papers published in the Transactions for February, 1884, and May last, has given considerable information in this direction, and is entirely in accord with the general statement that the use of steel is rapidly on the increase for bridge work, and that many engineers now give it the preference as being more uniform in character and a safer material than iron. They insist, however, on a rigid inspection, and the effort there, as it should be here, is to arrive at uniformity in requirements, and to reduce the testing to the lowest amount consistent with securing soundness of material.

Mr. Schneider has specified a maximum percentage of carbon and of phosphorus, which must not be exceeded in the steel to be furnished.

Now it is questionable whether the state of our knowledge respecting the behavior of steel is yet sufficiently exact to warrant us in prescribing to the steel-maker what it shall contain. As an example of this uncertainty I would refer to a table of tests given by Mr. Salom in a paper read before the Society of Mining Engineers. (See vol. XII, p. 661 of Proceedings.) He restates the fact (previously known), that silicon prevents carbon from combining with the iron, and suggests that phos-

phorus acts in the same manner. He gives an analysis of a sample of steel containing 0.16 per cent. of combined carbon, and 0.15 of graphitic carbon (or total of 0.31 carbon), 0.32 of manganese, 0.14 of silicon, and 0.065 of phosphorus. The tensile strength was about 63 000 pounds, with an elongation of 23½ per cent. and reduction of area of 49 per cent. Had all the carbon been in the combined state, the strength should have been some 80 000 to 90 000 pounds; or, with the strength as it was, the elongation should have been about 28 per cent. We learn then that silicon is also to be proscribed, if we are to specify the chemical constituents of steel. Mr. Salom gives, on the contrary, tests of samples containing .075 per cent. phosphorus, which gave good results. In a paper on the Clapp-Griffiths process by Mr. R. W. Hunt, published in Vol. XIII of Transactions of the American Institute of Mining Engineers, he gives tests of steel very low in silicon, but containing 0.11 per cent. carbon and 0.346 phosphorus (or over one-third of 1 per cent). The metal bent double cold, worked beautifully hot, was rolled into rail-plate and made good rails, had the valuable merit of stiffness, welded readily, and made button-head bolts which bent double in the thread. Samples from the screw-heads rolled into ½-inch round bars gave, when tested, a strength of 75 000 to 81 000 pounds per square inch, elastic limits of 55 000 to 60 000 pounds, stretches of 14 to 25 per cent. on 8 inches, and reduction of area of 14 to 49 per cent.\* As Mr. Salom says: "it would seem that phosphorus is not the *bête noir* it has been considered; it has had to bear the burden of many sins wrongfully ascribed to it."

In view of such facts as these, it is evident that the safe course for the engineer to pursue is to specify the physical characteristics of the steel he requires, and leave the method by which they are reached to the manufacturer. By so doing he saves himself an immense amount of labor; he widens greatly the sphere of competition; and is almost certain to get his material at lower rates.

This seems to be the uniform practice of English engineers. Mr. Parker, Chief Engineer of Lloyds' gave the following as the full detail of their method of testing steel for ships. A small ingot is cast from

\* In a later paper Mr. Hunt gives still further experiments, which all show that with low carbon, and silicon practically absent, phosphorus may run up 0.85 per cent., and still furnish a steel with a stretch of 9½ per cent., and a reduction of 9½ per cent. in area. The tensile strength was 101 540 pounds, although a test on a larger specimen gave less. The elastic limit of these high phosphorus steels seems to be about 75 per cent. of their ultimate strength.

each charge of steel. This is reduced to a standard size of plate or rod, which must have a tensile strength of not less than 27 tons (60 480 pounds) or more than 31 tons (69 440 pounds) per square inch, and give an elongation of at least 16 per cent. (not 20 per cent., as in Mr. Dorsey's paper)\* in 8 inches. Quite recently the papers state that this has been changed to 28 and 32 tons respectively. After heating to a cherry red, and being cooled in water of 82 degrees F. temperature, it must stand bending round a curve, the diameter of which is not more than three times the thickness of the plate used. This is called a "bending" or "temper" test. For certain parts of the ship's framing a strength of 33 tons (73 920 pounds) is allowed, provided the material will bear the other tests, and can be efficiently welded.

The testing is all done under the eye of an inspector, stationed at the mill, who sees the plates, etc., rolled, and a shearing taken from each. Each piece and its shearing is stamped with two numbers, as for example,  $\frac{1498}{491 B}$ . The lower number indicates the particular charge from which the plate, or beam, etc., is rolled, and the upper number the specific running number of the individual piece, by which it may be identified. A temper test is made from every plate, angle, or beam, if the inspector desires it.

The specifications require that not less than one plate, etc., in fifty shall be tested by both tensile and temper tests, and should it fail to come up to the requirements after a second test, the whole charge from which it came would be rejected.

For boilers, the tensile strength must range between 26 tons (58 240 pounds) and 30 tons (67 200 pounds), with 20 per cent. stretch on 8 inches, and with the same bending test.

Plates against which flames impinge directly in the furnaces and combustion chambers, must every one be tested by the temper test.

Mr. Parker says that punching affects only the under side of the punched plate, and that the plate can be safely bent without annealing if the concave side be the side that was down when punched. He also says, that to leave the metal in the best state after punching to resist strain, the bolster should be as large as it can be, and still leave a smooth hole. This prevents the punched metal from flowing into the

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\* Structural Steel. By Edward B. Dorsey, M. Am. Soc. C. E. No. 275, Vol. XIII, February, 1884, p. 43.

mass surrounding the hole, and in great measure the resulting state of tension which is such an element of weakness in steel plates.

He also lays particular stress on the necessity in rolling steel, of the use of high-speed rolls, so as to prevent cold rolling. When beginning to roll, the first two or three plates rolled are usually poor, in consequence of chilling, but they are perfectly restored by annealing.

He does not allow annealing, however, in most cases, but requires holes either to be drilled, or, if punched, assembled afterward and reamed  $\frac{1}{8}$ -inch larger.

Mr. Ewing Matheson, M. Inst. C. E. (of the firm of Matheson & Grant), showed me specifications for a bridge across the Hoogly on the East Indian Railway. The strains prescribed for tensile test were "for steel in plates, either with or across the grain, angles, T's, or flat bars, to be not less than 27 tons (60 480 pounds) nor more than 31 tons (69 440 pounds) per square inch, and to reduce 30 per cent. at point of fracture," with extension as in Lloyds'. Similar samples to bear the temper test as in Lloyds'. Every steel plate is to be tested for tensile strength, and to guard against brittle or dangerous material, an end and side shearing, properly marked for identification, is to be taken from each flat bar, angle bar and T bar by the manufacturer; also at least one angle or flat bar from every charge of steel. The shearings are to be tested by cold bending, in the presence of the engineer or his deputy. The angle and flat bars reserved are to be tested by what is called the "ram's horn" test. This consists in splitting them for some distance and bending the split portions back so as to touch the main part of the bar.

The rivet steel must stand a tensile test of 25 tons (56 000 pounds) to 28 tons (62 720 pounds), and reduce 40 per cent. at point of fracture. It must bend double both hot and cold, and bear flattening down from the head without cracking or other defects.

Straightening by hammering is not allowed for the angle and T bars; and the edges of all plates and the ends of all bars must, where practicable, be planed, or otherwise be dressed with chisel and file.

This firm gives the preference to steel over iron for bridges, and does not hesitate to recommend its use everywhere.

Mr. Matheson in a paper, Vol. LXIX, Minutes of Proceedings of the Institution of Civil Engineers, among other things states as follows:

"Rolled plates and bars (among which he includes deep I beams, channels, bulb beams, etc.), of the various forms required for structures,

can be manufactured in steel with as much certainty in regard to quality as iron of the first class.

"Steel can be manipulated in the factory—bent, straightened, cut, planed, drilled and punched—with the same tools and processes as are used for iron, and, for the most part, without extra force.

"The employment of steel may be encouraged and extended by a fuller knowledge among users of its qualities; by facilities for verifying those qualities; by exercising a wider choice of the kind of steel suited to the purpose in view; and by such liberal alteration of the present official rules as will allow fuller advantage to be taken of steel than is usual or permitted at present."

The Board of Trade rules allow but  $6\frac{1}{2}$  tons (14 560 pounds) working strain on steel; and the author urged that a 35-ton steel and 8-ton (17 920 pounds) strain might be used in many cases. In the discussion on the paper, Prof. Kennedy stated that mild steel could now be obtained of much greater uniformity than wrought-iron; that it was not more affected by scratches or corrosion; and was altogether a more reliable material. Dr. Siemens and others claimed that steel of 35 to 40 tons (78 400 to 89 600) and 20 per cent., could be produced of sufficient uniformity to warrant its use. (The average strength of steel in the superstructure of the East River Bridge was 80 000 pounds, with 20 per cent. stretch and 30 per cent. reduction of area at point of rupture. Mr. Schneider's requirements are less as to stretch.) Mr. White said the steel makers stated that they could produce Z bar sections, deep angle bulbs or heavy T bulbs more readily than iron makers could.

To show the wide divergence of the views held by different engineers on the matter of testing, it will not be amiss, in passing, to refer to the specifications framed by the United States Naval Advisory Board for the steel in the new United States cruisers. These do not allow the edges of sheared specimens to be planed, but only rounded off with a smooth file; while, according to Mr. Baker, M. Inst. C. E. (in a discussion mentioned further on), the results from specimens prepared in this way cannot be relied upon; and in his specifications for the Forth Bridge it is provided that test pieces shall be planed out.

The further anomaly is pointed out by Mr. Salom, that a cast of steel—perfectly uniform in quality and giving the full stretch required, but falling 100 pounds short in the strength required by the specifications—was rejected; while another cast—of which one plate tested 2 000 pounds short, and another 2 000 pounds over the requirements—was accepted, and rightly according to the letter of the specifications. In other words, the preference seems to be given to the steel which lacked most in uniformity.



In this connection it is desirable to refer to a recent paper by Mr. Hackney, Assoc. M. Inst. C. E., in Vol. LXXVI of Minutes of Proceedings, on Forms of Test Pieces. He calls attention to the fact that, to obtain uniformity in results, it is necessary to have a constant ratio of length to diameter of test piece, and that it is not sufficient to specify length only. Variations of 50 per cent. in percentage of stretch on the same material can be obtained by quadrupling the diameter and keeping a constant length.

For test pieces of a constant length, from plates of the same thickness, by increasing the width by doubling, trebling, etc., up to six times, the stretch was increased 33 per cent. in a particular case.

In the discussion on this paper, Mr. Baker, M. Inst. C. E., says that in actual work the bending test is the one most used, and in the hands of an experienced man it is as sound a test of ductility as can be obtained. (A crop end from every beam was tested by bending under a hammer, but had no previous heating, the object being to test the state in which the metal was left after rolling.) He mentions a test made in this way on a specimen, the edges of which had been filed to  $\frac{1}{4}$ -inch radius, which broke under the first tap of the hammer. A companion piece, however, with  $\frac{1}{4}$  inch planed off each edge, bore a succession of four or five light taps, followed by a heavy one, for an hour, and was then bent round through an angle of 120 degrees to a radius of 3 or 4 inches without breaking.

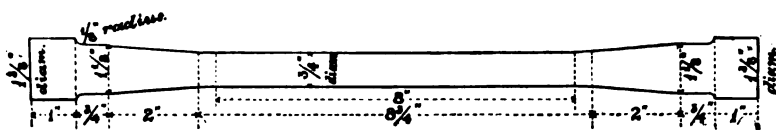
He had made all the tests on a 38-ton steel, of heating red and plunging in cold water, and bending to a radius of 1 inch; also the full stretch in tension, etc., required by the Admiralty, and considered it an admirable material for engineering works.

He says the form of a specimen to be bent "is quite as important as its form if tested by direct tension. It was much easier to bend a narrow piece of metal to a sharp radius than a wide one;" the reason given being that the narrow piece gives greater freedom for the flow of the metal, consequently the resulting strains of tension and compression are less.

Mr. C. P. Sandberg, Assoc. M. Inst. C. E., laid great stress upon the drop or concussion test as being "of more practical use than a tensile test, and should be given the preference wherever possible."

Dr. Wedding gave the standard form for test pieces adopted by the Prussian Experimental Institute as given below, and said if the pieces

broke outside of the 8-inch points the test was rejected. He considered it important to have the straight portion of the specimen continue beyond the limits taken for test, as is shown.



An example of recent French practice in the use of steel was furnished me by Mr. Lavoinne, engineer of the Ponts et Chaussées (since deceased), in a specification for the new arch bridge at Rouen. "Bars 8 inches long and  $1\frac{1}{4}$  inches wide, cut from any part whatever of a piece of steel, whether before or after work of any kind—of shearing, forging, drilling or riveting—must stand bending cold at the middle, so that the two ends touch, and so that the longest distance between the two exterior faces shall be reduced to four times the thickness, without developing a trace of cracks. Tests to be made at the mill are :

"*First.*—They must be able to resist a tensile strain of 31 000 pounds per square inch within the limit of elasticity.

"*Second.*—To stand a breaking strain of 71 000 pounds on the same section.

"*Third.*—At the moment of rupture, the stretch on the specimen shall be 18 per cent. of its length, and the stretch must progress uniformly with the load, without sudden jumps."

The French Admiralty require for steel plates up to  $\frac{1}{4}$ -inch thickness a minimum average tensile strength of 67 000 pounds, and a mean stretch of 10 to 14 per cent. on 8 inches ; for plates of  $\frac{1}{4}$ -inch thickness and upward, 63 000 pounds and 20 per cent. ; and between these thicknesses intermediate values.

The initial load in testing must be  $\frac{1}{10}$  of the breaking strength mentioned, and must be applied for 5 minutes. Then successive additions of 700 pounds per square inch are to be made at about every half minute, and the stretch for each noted. A plate with less than  $\frac{1}{10}$  of the mean final stretch allowed to be rejected. At least 5 tests (from one or more plates) with 5 across the plate to be made from plates of each delivery, and the means of each set taken. That mean which is the least must be equal to the minimum average allowed.

A temper test is made similar to the English, the inner radius of curve after bending to equal the thickness of plate.

A forging test as follows: From a plate there is struck up a hemispherical cup, whose inner diameter must equal 40 times the thickness of plate, and whose border must be 10 times the thickness in width, and join the sphere exteriorly by a curve of radius equal to the thickness.

For angles, T, I and bulb bars, etc., one is taken from each delivery. Samples cut from the webs of angles and T's must show a tensile strength of 68 000 pounds, with stretches of 18 to 20 per cent., and I and bulb beams 65 500 pounds, and 16 to 18 per cent., with the same rules as to time, etc.

In the temper test the radius of the curve to be  $1\frac{1}{2}$  times the thickness.

The forging test for angles consists in bending one web (while the other remains flat in its own plane) to the form of a cylinder having an interior diameter equal to  $3\frac{1}{2}$  times the width of the flat web. To close the two webs of another piece until they are in close contact. To expand the webs of a third piece until the inner faces lie in the same plane.

T bars have a piece forged so that the flange shall be in the form of a semi-cylinder, with interior radius 4 times the height, while the web remains in the same plane. To split the web of another piece in the middle for a length 3 times the height of the bar, drill a hole to keep it from splitting farther, and then bend the detached part by a sharp bend in its own plane to an angle of 45 degrees with the other branch.

I and bulb beams shall have similar tests. One flange of the I beam or the bulb of the bulb beam being bent by one or several heats with a sharp bend to an angle of 45 degrees with the other flange, the web being flat.

Referring to the Appendix to Mr. Schneider's paper, it is interesting to note, in the tests on full-sized bars, that up to 2 000 pounds beyond the limit of elasticity, the stretch, as measured on two-thirds of the body of the bar, is almost exactly one-half of that between the centers of the pins; the greater proportional amount on the other third being due to the flexure and compression in pin-holes.

One other point in these specifications which may be criticised, is the direction to "grout" each course of masonry. Nothing is better established than that grouting is an exceedingly uncertain method of filling masonry joints. Good mortar can be worked in by "swords" or bars until it is positively certain that the joint is full.

PERCIVAL ROBERTS, JR., M. Am. Soc. C. E.—In examining the tables of tests of steel in Appendix No. 4 of Mr. Schneider's paper, the first impression obtained is that although in steel we have a homogeneous material, it is yet a far from uniform one, no matter how carefully we may watch the chemical composition and make rejections for variations of but .01 of 1 per cent. in carbon. I have rearranged the tables and grouped all tests showing same carbon together, and obtained the average results for each set so grouped. Considering first the table of accepted tests, we have the lowest carbon .30, and the highest .41; the greatest number of tests however contain .35 per cent. of carbon, and are 24 in number. From this group take for example:

Test 2	0.35	0.510	....	64 100	89 920	18.25
" 27	0.35	0.510	0.082	49 450	79 430	19.37

A difference of 12 per cent. in the ultimate, and 23 per cent. in the elastic limit. Again;

Test 90	0.35	0.550	60 600	84 130	18.25
" 95	0.35	0.555	51 600	85 780	18.12

A difference of nearly 15 per cent. in elastic limit, while, in other respects, both physically and chemically, the two pieces were practically the same.

Compare the stretch and reduction of area and we find no uniformity, for example:

Test 27	49 450	79 430	19.37	45.50
" 111	51 400	85 880	17.25	30.50

A difference of 15 per cent. in reduction of area, and in other respects nearly identical. This matter of non-uniformity is not confined to a few isolated cases which would lead us to look for errors in testing machines or recorded results, but are scattered throughout the entire tables, the only uniform point in the whole matter being its want of uniformity.

Leaving the tables of accepted tests, take up that of rejected specimens. In this carbon ranges from .25 per cent. to .53 per cent. No other element is given in analysis.

Take, for instance, tests 62, 89, 106, and many others. Carbon we find within specified limits. Physical tests meet requirements. They are probably rejected for phosphorus, that poor little suffering element so important to the engineering brain, and yet trampled on so basely

in the above rejections. But I need not consume any further time in analyzing these tables, as the points I have made are plain to the most casual observer. The question I wish to raise, in view of the above-mentioned points, is the advisability or justice of the following clause taken from Mr. Schneider's specification. "All steel shall be manufactured by the open-hearth process. Bessemer steel will not be accepted." No reason being given for the decision, I presume I am right in considering that such a clause was inserted with the belief that open-hearth steel is more uniform than Bessemer, a statement which, I think, rests more upon fancy than fact. Equally good steel can be made by either process, if handled properly, and equally good materials used. In working very heavy masses the open hearth has, I think, the preference, as it admits of slower working than the pneumatic, but for all weights of ingots covering requirements for structural work, I can see no reason for barring out the Bessemer converter. Our prejudice has, I think, been strengthened, also, by the fact that it has been the plant employed by the rail-makers. Rail-steel I consider no more fitted for structural purposes than rail-iron as compared with structural iron. Their organizations have been planned and arranged entirely for this single line of manufacture, and to turn aside for a short time upon another class of products has produced an uncertainty in results in nowise due to the process, but to the improper handling of the same. I feel confident a Bessemer plant, organized with a view to the making of special steel, will produce results which, in practice, cannot be distinguished from those of the open hearth, and I think it a matter of injustice to exclude so important a method of manufacture without more accurate proofs than we have. An engineer's province should be to make as rigid an inspection and tests as he chooses, but leave the method of working to the manufacturer. In support of what I have said above, the following table gives tests of Bessemer steel:

Tests 59 to 67 were made to conform to United States Government Specification.

Tests 69 to 207 were made to conform to Lloyds' Ship Specification.

They were, as you may see, upon greatly varying sections, whereas the cantilever tests were all upon test specimens of same area, with same reduction in rolling, and should on this account have been much more uniform than had they varied in these respects.

## TESTS OF BESSEMER STEEL.

No. of Test.	Original Section.	Length.	Area in Square Inches.	PER SQUARE INCH.		Stretch, per Cent.	REMARKS.
				Elastic Limit.	Breaking Strain.		
59	$3\frac{1}{2} \times \frac{1}{8}$	8"	.4214	33 900	53 300	31.2	Open Hearth, Otis.
60	"	"	.4088	37 900	61 700	28.9	
61	"	"	.4214	38 000	57 600	27.3	Open Hearth, Otis.
62	"	"	.4214	37 800	56 400	28.1	
63	"	"	.4137	35 400	53 400	30.4	
64	"	"	.4056	38 400	61 800	25.7	
65	"	"	.4062	37 600	56 100	28.5	
66	"	"	.4056	37 100	56 800	28.9	} A Rail Bloom. Shows unsuitability of this grade.
67	$6'' \text{ I}$	"	.3869	56 220	107 500	9.4	
68	"	"	.4418	53 600	96 600	18.4	
69	$3\frac{1}{2} \times \frac{1}{8} <$	"	.4052	38 300	61 500	29.0	
70	"	"	.4170	39 300	58 200	30.0	
72	$3 \times \frac{3}{8} <$	"	.4750	37 470	61 050	28.1	
73	"	"	.4855	36 560	58 080	27.3	
103	$4 \times 3 <$	"	.4453	39 800	63 160	28.5	
104	"	"	.4532	39 490	63 050	26.6	
106	$3 \times \frac{1}{2} <$	"	.4000	41 870	66 060	25.8	
107	"	"	.3920	44 770	65 560	23.4	
116	$5 \times 3\frac{1}{2} <$	"	.5020	39 340	61 800	26.2	
153	$7'' \text{ I}$	"	.4185	35 840	65 710	23.0	
154	"	"	.484	35 840	62 910	24.6	
155	"	"	.448	38 230	63 390	21.9	
156	$3 \times 2\frac{1}{2} <$	"	.3782	44 550	66 100	22.7	
157	$10'' \times \frac{1}{2} <$	"	.3812	45 840	68 640	24.6	
161	$3 \times 2\frac{1}{2} <$	"	.5200	39 900	60 530	23.0	
163	"	"	.3937	43 120	63 240	26.8	
207	$3 \times 3 <$	"	.4233	38 090	59 290	28.1	
...	$7'' \text{ I}$	....	....	37 000	65 700	23.0	
...	"	....	....	37 500	62 910	24.4	
...	"	....	....	36 000	63 390	21.8	
...	$8'' \text{ I}$	....	....	39 000	61 440	....	
...	$9'' \text{ I}$	....	....	41 000	67 400	21.9	
...	$12'' \text{ I}$	....	....	42 000	70 200	21.5	

The method now in vogue of testing steel from sample billets rolled into  $\frac{1}{2}$  rounds, I think open to criticism. It tells us about as much concerning our steel, as used in the shops, as a chemical analysis of the ingot would do. Test pieces should by all means be taken from the finished sections, for what guard have we against bad workmanship in the rolling mill unless we do. The heating furnace, to which so many failures in iron can be traced, plays an even more important part in the manipulation of steel. I think there would be about as much wisdom in making all our tests of wrought-iron upon  $\frac{1}{2}$  rounds, and the iron mills would never know what a condemnation was. It is said in answer to this that the manufacturer of steel would be put to too great a loss if condemnations were made upon finished material. I confess his position is not a pleasant one, as, in the present case, 56 per cent. of all steel made was

rejected. But I feel confident that the test as now made should not be final. Another point to which sufficient attention has not been given, is the very great injury done to steel in the operation of straightening. The gag press should be unknown in the steel-mill. A number of experiments we have made at Pencoyd, fully confirm my strong language in this matter.

It is claimed that testing, as now done, gives us a uniform grade of material. Take, for instance, a built-up section of component parts differing considerably in sectional area, but all of this uniform grade of metal. Owing to difference in reduction of area from bloom in rolling, these will all give different physical results. Would it not be better to have uniform physical results in all, and a material differing somewhat chemically, which would be necessary to produce this result?

In all our testing, I think we have given too great a prominence to the behavior of the material at the point of rupture, an event which we trust will never occur during the life of our structures, and have not considered sufficiently the action of strains below and up to the elastic limit. I am well aware that these belong more properly to laboratory work, and yet they are of the greatest importance in every day work. The fact first described by Mr. James Christie, M. Am. Soc. C. E., that per unit of load steel and iron deflect equally within their elastic limits, is of the utmost importance in every day practice. The recent failure of the Dolphin, I think, may partially be traced to want of rigidity, due to this very circumstance, steel being considered, in all respects, 25 to 20 per cent. stronger than iron. In conclusion, I hesitate to say one word more in reference to Government aid in the matter of testing, which I believe is the only solution of our present difficulties. As a manufacturer I can speak for ourselves alone, but I do not hesitate to say that if the Government would make tests, all the materials required would be furnished free of cost.

E. B. DORSY, M. Am. Soc. C. E.—The thanks of every Member of this Society are due to Mr. Schneider for his very elaborate and lucid description and plans of the largest cantilever bridge that has been built; and, as engineers, we should also thank him for showing how quick engineering work can be done. During my residence abroad I have heard many prominent foreign engineers express their astonishment at the speed with which this bridge was built.

In looking over the tables of the tests of steel made by Mr. Schneider, one is struck with the great number of rejected heats. Out of two hundred and forty-five heats made by the Spang Iron and Steel Company, one hundred and thirty-six, or 55 per cent., were rejected. What was the cause of this? Was it owing to too high requirements in the specifications or bad workmanship on the part of the steel makers?

In my judgment, engineers make a great mistake in making the specifications of steel too high, for two reasons:

*First.*—The cost of manufacturing the rejected portion will be added by the manufacturer to the cost of the part that is accepted. In this case over one-half of the steel was rejected, which probably added 50 to 100 per cent. to the cost of that which was accepted; while, if the specifications had been lower, say the same as Lloyds' Register of England requires for steam-boiler plates, viz., tensile strength, 26 to 30 long tons (58 240 to 67 200 pounds) per square inch, and elongation, 20 per cent. in 8 inches, the rejected heats and wasters would not have amounted to 3 per cent. in a well-managed steel-works.

Thus, in order to get one-third more tensile strength, the cost was probably increased from 50 to 100 per cent.

*Second.*—In my opinion, formed after consulting freely with those who have worked a long time and largely in mild steel for structural purposes, I think the best steel for this use, and the one that will work the easiest, and be the most reliable under all circumstances, is that with a tensile strength of between 60 000 and 65 000 pounds per square inch, an elongation of over 20 per cent. in 8 inches, and which, heated to a low cherry red heat and cooled in water of 82 degrees F., will then bend, without fracture, to a curve the inner radius of which is one and one-half times the thickness of the piece.

Steel of this quality can now be bought for about the same price as good wrought-iron suitable for bridge work. In other words, for the same money, a material one-third stronger, and much more reliable and uniform in texture, can be obtained. With this advantage the engineer should be contented.

Above 70 000 pounds tensile strength per square inch, steel becomes uncertain and unreliable, in fact very capricious, cracking and breaking without any apparent cause; while at and below 65 000 pounds it is entirely reliable, being very ductile, and bending cold to almost any desired shape.



In using steel of high tensile strength, nominally the structure may appear stronger, but in reality it is probably weaker than if built of material of less tensile strength, but of more uniform and reliable character.

In my judgment our engineers are requiring too much from steel. It is undoubtedly a very superior metal, and one that will, in my opinion, soon supersede wrought-iron almost entirely, but at the same time they should not impose too much upon it.

It can be made much stronger than wrought-iron—say four times—but until the reasons for the capricious actions of the stronger kinds can be discovered and remedied, I would not advise the use of any over 65 000 pounds tensile strength per square inch.

Since reading my last paper on steel to this Society (at the Convention at Buffalo), I have seen and consulted with many parties who are largely engaged in manufacturing steel for structural purposes, but I have heard nothing to induce me to change or modify the opinions expressed in that paper.

CHARLES DOUGLAS FOX, Cor. M. Am. Soc. C. E.—I have read with much interest the ably written communication by Mr. Schneider, descriptive of a Cantilever Bridge at Niagara Falls.

It appears to me that this bridge is one in every respect adapted for its purpose; admirably designed, both generally and in detail; and carried out with remarkable rapidity. It would have been interesting, for purposes of comparison, if the cost had been given, and perhaps Mr. Schneider may consent to add this.

I observe that the strains on this bridge are simple and well defined, and thus easily ascertained, which is unfortunately too often not the case.

I observe that steel was only used in compression and in pins, owing to the uncertainty of its quality and delivery. Engineers in this country are using open-hearth steel increasingly in various parts of structures with great advantage, and tests are adopted which secure fairly uniform results.

The results of the testing of the tie-bars appear to me very low, viz., 18½ tons per square inch, with 10 per cent. contraction and 6 per cent. elongation. These are no higher than are yielded by inferior plate iron in this country.

The report on the testing of the bridge proves the great strength and stability of the structure.

I observe that this bridge was designed by the manufacturers, which is not usual with us here.

B. BAKER, M. Inst. C. E.—Equal interest has been taken by engineers on both sides of the Atlantic in the Niagara Cantilever Bridge, not only on account of its intrinsic merits, but by reason of its being to many engineers the first known example of a system of construction now attracting much attention. In illustration of the latter fact, the writer may state that he recently received a letter from Palermo, Sicily, announcing that an investigation of the cantilever bridge system was the thesis appointed this year for the consideration of engineers proposing to take the degree of "doctor." There is little doubt, therefore, that cantilever bridges will soon be found in many parts of the world.

As regards the foundations of the Niagara Bridge, it may be safely concluded that no apprehension of insecurity need be entertained. A natural rip-rap mound or slope, which has had ages to consolidate, must necessarily be more stable than an artificial mound of rubble stones, having imperfectly filled interstices; and yet, in cases such as the Holyhead Breakwater, close-jointed masonry walls, 40 feet in height, are supported without any indication of settlement, though the weight per square foot upon the 40-feet high rubble-mound foundation is considerably in excess of that obtaining in the instance of the Niagara Bridge.

The author observes that if the web-systems were continuous, the stresses would be ambiguous, but that by omitting the diagonals in the panel over the braced piers, the stresses become well defined and can be determined with precision. There is, of course, ambiguity of stress in every bridge. If a number of eye-bars are strung on to one pin, the stress upon each will be affected by imperfections in workmanship, by variations in the modulus of elasticity, and by the amount of cold straightening the bars may have respectively undergone. Within the same limits, and no further, the introduction of bracing in the panel over the pier would lead to ambiguity of stress; but from experimental results, the writer believes this consideration need not affect the expediency of putting on or omitting such bracing in a cantilever bridge. Much useful information concerning the actual stresses on existing structures can be obtained by a little apparatus of almost ridiculous simplicity, arranged by Mr. Stromeyer, of Lloyds' Registry of Shipping. It consists of two short strips of polished hoop-iron lightly pressed to-

gether, and having a little roller of steel wire about  $\frac{1}{16}$  of an inch diameter clasped between. One end of each piece of iron is clamped to the bar under test, and a straw index-needle is fastened by sealing-wax to the end of the wire. As the specimen extends or compresses, the steel wire revolves, and by an easily constructed scale, the strain is read off. With this apparatus the straining of a ship's plates by a passing wave is plainly apparent, and the writer, with it, has determined the distribution of stress on a braced steel column before flexure with the most satisfactory certainty.

The experience as regards uniformity of quality in steel would appear to have been rather unfortunate at the Niagara Bridge. It has been otherwise at the Forth Bridge, where, although plates have necessarily been rejected for surface defects, and for occasionally passing the limits of required variation in tensile strength and elongation, not a single plate or bar has been rolled which might not with perfect safety have been used in the bridge. The writer has at times taken a rejected plate with what would look like an ugly surface crack running along it, and placed it, crack downwards, under a steam hammer, but he has never yet succeeded in picking out a plate which would not bend at least to an angle of 90 degrees before cracking. Out of 12 000 tons of steel-work now manufactured at the Forth Bridge, the number of plates and bars which have failed in forging and bending, or otherwise, has been under half a dozen, and the cause then was improper treatment, and not bad quality of metal. Steel of a strength suitable to bridge-work requires the holes to be drilled and the edges planed, and no work should be put on the plates at a blue heat without subsequent annealing. Apparently the usual temper test was not enforced in the case of the steel for the Niagara Bridge. This test is imperative, as steel which is affected more than 10 per cent. either in strength or ductility by any kind of heating and cooling, is hardly suitable to the requirements of the bridge-builder. Good steel, even of as high a tensile strength as 80 000 pounds per square inch, will stand a great deal in the way of heating and cooling, though punching or shearing is extraordinarily fatal to its ductility. Thus the writer has made red hot about one-third of the area of a 3-foot square by 1½-inch thick plate and cooled the plate by dashing water over the whole surface. Notwithstanding the heavy internal stresses which may be supposed to result from this treatment, and which have so often been cited as the cause of

"mysterious fractures" in steel plates, the specimen so treated has bent readily to an angle of 90 degrees before fracture. On the other hand a similar plate, not heated and cooled at all, but simply having the edges sheared, has snapped across with a hardly visible amount of bending.

The execution of the works of the Niagara Bridge in so short a time was a feat of which the engineers and contractors of any country in the world might well be proud. Probably in this as in almost every other case, the experience gained would suggest certain modifications in the details of construction. As it stands, however, the bridge is a very interesting example of the adaptability of essentially American details of construction to a novel type of bridge. At the Forth Bridge, owing to the magnitude of the parts, the details in many respects present a greater analogy to the building of an Atlantic steamship than to that of an American bridge. In erecting the work the unit will be a single plate, and not a complete strut or tie, as in the case of a smaller bridge. Nearly all the riveting will thus be done when the plates are in their final position, and plant of a novel kind has necessarily to be devised for the purpose.

C. A. MARSHALL, M. Am. Soc. C. E.—There are two or three points in the remarks of Mr. Roberts which I would like to reply to. There seems to be no one here to defend the steel of that particular contract, and I do not like to see the discussion become one-sided. I have no interest therein, but I can explain partly, I think, the apparent irregularities that Mr. Roberts drew attention to. You remember that the specification requires the test to be made upon a  $\frac{3}{4}$ -inch round rod made from a 4-inch test ingot. Our experience has been, at the Cambria Iron-works, that those 4 or 5-inch test ingots, cast as they are ordinarily, and as I have no doubt was done in this case, without any special precautions, will not be uniform—will not give reliable results. We can correlate very well the chemistry and the test of a  $\frac{3}{4}$ -inch round piece when we take our test billet out of the large ingot; but the small ingots are full of blow-holes and cold shuts, and make miserable bars. In many cases bars coming from them are not such as can be used—would not be merchantable steel; and I think that will explain, in a very considerable percentage of cases, the irregularity that Mr. Roberts drew attention to.

At any rate we have discarded the use of such ingots in every case we possibly can. Mr. Roberts mentions also that the objection to discarding those ingots is that it entails a loss on the manufacturer to work up a portion of his steel into its final shape. It is true there is such a loss if the steel is rejected; but there is also the advantage that a good deal more of the steel gets a fair test and goes through. Mr. Roberts also mentions the manufacturer's loss in testing the finished material. It is also to be remembered that the engineers, the contractors, or whoever is buying the steel, lose a great deal too by that method. They lose time. If any question arises as to the suitability of any heat or blow of steel after it has all been rolled, and after the rolls have been changed and the mills have gone on to other work, there is a loss to the manufacturer and the buyer in waiting until new steel can be made and rolled.

I would like to call attention also to the fact that the method of testing, which many advocate, of cutting test-pieces from the finished material, and accepting or rejecting material from those tests, has caused a great deal of dissatisfaction in England, where this method has been followed for ship plates, etc. I am sure any one who will read up back numbers of the Proceedings of the Institution of Naval Architects will discover that fact. They have been through the experience, and paid for it dearly. Another point bearing on the same subject is the question as to whether we should require a certain uniform ultimate strength or physical characteristics from finished material of different sizes and shapes. Different treatment, as is well known, causes, in the same steel, these physical characteristics to vary. I do not wish to undertake to discuss the subject thoroughly, which I am not prepared for, but would call your attention to a paper, which many of you have doubtless seen, relating to some boiler plates which Mr. W. Parker, Chief Engineer-Surveyor of Lloyds' Register, investigated. The boiler plates were made to such a specification; they were required to have a certain ultimate strength between definite limits. These particular plates were, I think,  $1\frac{1}{4}$  inches thick, and were required to have the same strength as a  $\frac{3}{4}$ -inch or a  $\frac{5}{8}$ -inch plate.

The result was that the manufacturers, in order to attain that specific result, put in the neighborhood of  $\frac{3.0}{100}$  per cent. carbon. They obtained the ultimate strength, but got plates which would not stand the shop work that the boilers had to undergo, particularly the heating, and the

plate cracked when the boiler was finally tested. In bridge work, it is true, the material has not to stand such extreme torture as in boiler work, but nevertheless there are other more important points to be looked after in material to be put into a bridge than simple ultimate strength. The cost of the bridge is made up of material, in which you can save by adding strength, and of workmanship, in which you lose by adding strength.

G. LINDENTHAL, M. Am. Soc. C. E.—The Cantilever Bridge over the Niagara River, by C. C. Schneider, M. Am. Soc. C. E., is typical of a class of continuous girders over four (4) supports, which has certain advantages in bridging streams where the erection of false-works for the middle span is impracticable. The girders can be calculated with precision on statical principles with as rigid economy in the use of material as in common truss bridges.

A comparison of weights with single-span girders of same spans and for double track, and with parallel chords and for same loads, will be instructive.

Thus, the following weights of Iron and Steel (steel in compression members only), for ordinary trusses, including 100 pounds per lineal foot for hand-railings, are deduced from experience as minimum weights:

Two 208-foot spans, 2 800 pounds per lineal foot, 582.4 tons.

One 495-foot span, 6 200 pounds per lineal foot, 1534.5 “

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Total..... 2216.9 “

The weights as given by Mr. Schneider are :

For cantilevers..... 3 211 134 pounds

“ intermediate span... 294 973 “

“ anchorage..... 181 032 “

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Total..... 3 687 139 “ equal to 1843.6 tons.

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The cantilever bridge is lighter by..... 273.3 “

The saving is nearly 13 per cent. as compared with single-span girders. In this connection could be mentioned the Kentucky River Bridge, by C. Shaler Smith, M. Am. Soc. C. E., as another type of pin-connected continuous girders with stringers of wrought-iron. It has three (3) spans of 375 feet each, single track.

A comparison of weights with a discontinuous girder bridge with the same floor construction and for same loads, is as follows:

Actual weight of continuous girders over three	
375-feet spans.....	1 427.7 tons.
Weight of iron in discontinuous girders per lineal	
foot 3 100 pounds, assumed from experience .	1 743.7 “
Difference.....	316.0 “

representing a saving in material of about 18 per cent. as compared with ordinary trusses. Continuous girders, hinged, one way or the other, show then, a saving in material, which, however, does not always imply saving in cost. The cost of manufacture and erection may sometimes wipe out, if not exceed the saving in material. For the Niagara Bridge, however, the cost of manufacture could not have been greater than for ordinary pin-connected bridges, the character of details and work being the same. The cost of erection is not given in Mr. Schneider's paper, but it could hardly have been more, if not less, than if the girders could have been erected on false-works resting on the bottom of a ravine 150 feet deep, a method which, of course, could not be thought of in this case. If this is correct, cantilever bridges are cheaper for long spans than ordinary truss bridges, even in cases where the erection on false-works is convenient.

While, in continuous girders without hinges (or with hinges arranged as in the Kentucky River Viaduct), the dead load may, without serious error, be assumed as evenly distributed, such can evidently not be done for an arrangement as in the Niagara Bridge, and Mr. Schneider justly calls attention to the importance of the question of how the distribution of dead weight in the cantilever shall be assumed.

An approximation of the distribution of material, and therefore weight in a girder, will be obtained from a solid beam of rectangular section in which the height changes with the moment of resistance at the respective sections. Thus we obtain for a uniform load on a beam, for its profile, a half-ellipse.

But from experience we know that the weight of framed single-span bridges is nearly uniform; the wind bracing is heavier towards the abutments, and is to be added to the weight of the trusses proper, so that the weight per lineal foot remains nearly the same; it is represented by a rectangular profile instead of an elliptical one. In a beam supported

at one end only (a cantilever or bracket), the conditions are different. For a uniform load, the form of the bracket would be a triangle, and for a load uniformly decreasing towards the end, the bracket would have the shape of a triangle, with the upper edge straight and the lower edge in a concave curve, a parabola of the  $\frac{1}{2}$  degree, if we neglect in each case the weight of the bracket itself.

The dead-load, or weight of cantilevers in longer spans forms such a large part of the total load, that on its more or less correct assumed distribution the design depends for its success.

Thus in the Niagara Bridge the dead-load represents about 45 per cent. of the total load, and in the Forth Bridge, now in process of construction in Scotland, the dead-load represents about 90 per cent. of the total load. A few trial calculations will always be necessary to narrow the results down to the nearest approximation. This is even a better way than a mathematical analysis of the problem, "to find the profile of a bracket in which the ordinates are proportionate to the moments of resistance at each section from the imposed loads, including the load of the bracket itself."

The analysis is difficult and intricate, involving, as it does, the calculus of variations. I am not aware of a solution of this problem, and I doubt whether it would furnish us with any practical results.

The conditions of maximum strains are: 1—A load suspended from the end. 2—A uniformly-distributed train load. 3—The load of the wind bracing, uniformly increasing towards the supports. 4—The load of the cantilever girder itself.

The limiting curve is probably of a changing character, concave towards the axis at its origin and for some distance away, and then becoming convex. The solution of the problem includes the finding of the area inclosed by this curve by the axis and by the ordinate at point of support; and then finding the minima of the sum of this area and one-half the area of suspended span, which may be assumed rectangular. Having found the expression for the minima, from it may then be found the relative lengths of the cantilever and suspended girders for the most economical arrangement as respects amount of material. But this may again vary greatly with the system of the cantilever, so that the general solution, as outlined above, is not as reliable as an empirical investigation in each particular case. There is reason to believe that designing a cantilever bridge merely so as to



obtain greatest economy of material, will not give the best design. It may become wanting in rigidity.

The deflections under moving loads are greater in a cantilever girder than in any other kind of girder.

The deflection of nearly 7 inches, of the Niagara Bridge under the test-load on both tracks, and 4 inches under the test-load on only one track, shows this to be the case by actual trial. This means that for *fast trains* the cantilever bridge is not as well suited as other designs. This is an important defect. A cantilever bridge is vertically and laterally the least rigid of bridge systems, other things being equal.

There are two ways to reduce the deflection. One is to make the cantilevers high at the point of support as compared with their length (as Mr. Baker has done in his Forth Bridge), and the other is to reduce the length of the cantilever as compared with length of suspended truss. This again may result in greatly increased weight of the bridge. To balance all these questions in the design is one of the intricate problems of modern bridge engineering. Mr. Schneider's Frazer River Bridge has some points of excellence which are not in the Niagara Bridge. They are the solid stone piers on which the cantilevers are supported at only one point, presumably having a roller-bearing on the pier; and the single diagonal system instead of the double diagonal as in the Niagara Bridge.

In the Niagara Bridge the cantilevers have movable bearings at the anchorages, therefore the resistance to a train braking on the bridge must all be from the towers.

Taking a train over one-half the length of the bridge = 455 feet, averaging 2 500 pounds per lineal foot, and the resistance between rail and wheels as 10 per cent. of the train load, as usually assumed, then the whole  $\frac{455 \times 2\,500}{10} = 57$  tons must be resisted by the tower. From this the strain on the diagonals would be about 9½ tons per square inch, and at the foot of the tower 245 tons, or about 122½ tons on each post, for which no special allowance seems to have been made. It is not likely that the maxima from train load on both tracks from wind and braking will occur simultaneously; but if they did, the resulting strain of about 9½ tons per square inch on the steel columns as designed, would still appear to be within safe limits.

However, as the braking of a train produces a bending of the tower,

it is liable to set up in the structure a slight lurching motion, and therefore the above construction is not favorable to rigidity.

All this could have been avoided by anchoring the ends of the cantilevers fast, and having roller-bearings on the towers. The arrangement in the Frazer River Bridge is, I believe, the better of the two.

If Mr. Schneider had adhered to his original intention of using the single diagonal system, as shown in his first sketch, it would have been an improvement over the double diagonal system, as it always is wherever practicable. In the single diagonal system the strains are massed into larger and more compact members, through which they reach the supports on a shorter way.

The detail arrangement at both ends of cantilevers, of uniting both diagonal systems on one pin, and connecting this pin with a short link to the chord pin, is theoretically correct, because thereby the strains in both systems of diagonals become statically determinate, though practically it does not work well. One reason is that the strains divided up at the link pin, enter into tension members of different length. The shorter diagonal transmits its component into the chord sooner than the longer one. Another reason is that the maxima in the diagonals occur consecutively and not simultaneously. The result is a slight pendulum motion of the link, as observed under the action of the rolling load.

The omission of the diagonals in the cantilevers above the towers shows Mr. Schneider's appreciation of clearly defined strains.

There are many continuous girder bridges having double bearings on, and a continuous diagonal system over the piers, without any provision against the mischief which such an arrangement is sure to cause.

A notable example is the celebrated Iglawa Viaduct of the Austrian State Railway in Austria, having 6 spans of 200 feet each, supported on cast-iron towers similarly to the Niagara Bridge, but with the trusses continuous over the supports. It was built in 1870. The rocking motion of the rigid girders over the towers caused excessive strains, and injured the towers so that they had to be rebuilt of wrought-iron recently.

The Cantilever Bridge over the Forth in Scotland, now building, is another such example with double supports for the cantilevers. The middle cantilever on the Island of Inchgarvie must have diagonal

bracing between supports to prevent a tilting of the structure under a moving train; but the shore cantilevers need no such bracing, as the shore end is anchored down. Still the diagonal bracing has been put in, and consists not only of ties, but of immense intersecting tubes, 8 feet in diameter, capable of taking compression and tension alternately. They are not only unnecessary, but, if they act at all, the effect will do mischief, and never any good. The system is no longer statically determinate.

The strains in the shore cantilevers become ambiguous, and for safety the upright columns ought to be made *much stronger* than would be necessary if the diagonals *were not* put in. Granted, that on account of the small proportion of live to dead load the ambiguity from the effect of the live-load cannot be very great, the mischief can be great in case of settlement of any of the foundations after erection. Though the great care with which the work is conducted reduces very much the probability of such an occurrence, it is still better to design a structure so that, if it does occur, it can do no harm. In this view, compare the Niagara Bridge with the Forth Bridge. The pin connections of the former and the rigid riveted connections of the latter.

Suppose the diagonals between supports left out in both bridges, then the hinged connections of the Niagara Bridge adjust themselves under the influence of moving load, changes of temperature or settlement of supports, without sensible change in the stresses. On the other hand, the rigid riveted and some of them eccentric connections of the Forth Bridge cannot adjust themselves, under similar conditions, without seriously overstraining the metal. The unavoidable secondary strains can be made a minimum with correctly-designed pin connections; they can never be less than a maximum with rigid riveted connections in which they may, from slight causes, strain the metal, even more than the primary strains due to the loads. Particularly in the Forth Bridge the cross strains at the connections will be greater from dead than from live-load, as panel after panel is added during erection.

Again, compare the Niagara Bridge with the Forth Bridge in the event of one girder settling more than the other. The usual system of cross-bracing here with adjustable members, allows of easy correction, against no adjustment in the rigid connections of the Forth Bridge.

In point of quick manufacture and erection, the Niagara Bridge is an

example of what can be done with the present improved methods of bridge construction.

If I should discuss in this connection the advantage of pin-connections to riveted-connections in bridges, I should only repeat what so often has been discussed before, and always with the same result, namely, in favor of pin-connections, which are now universally recognized as superior to all others in rational bridge construction.

The architectural appearance of the Niagara Bridge will, to many, not be pleasing. Having Roebling's graceful railway bridge for a near neighbor, the disappointment may be the result of contrast of designs.

WM. SELLERS, M. Am. Soc. C. E.—I would like to say a few words with reference to the specifications which have been read, and to which my attention was directed particularly by Mr. Collingwood's remarks. Not having read the specifications, I am not able to say whether or not the chemical tests referred to are required. For myself I have always held that an engineer oversteps his position when he requires such chemical tests, or in fact any other provision of manufacture. He ought to leave the manufacturer entirely free to avail himself of all the knowledge he can acquire to produce the quality of work which is desired, and that quality ought to be determined solely by physical tests. The reference to the eye-bars made by the Edge Moor Iron Company was simply a few experiments that were tried some months ago and which have been since extended, and I have with me some drawings of bars that have been pulled to the destruction of the bar, in which the excess in the eye was but  $12\frac{8}{10}$  per cent. above the bar. With proper-sized pins in proportion to the width of the bar, that excess need not be over 10 per cent. There was a matter referred to in Mr. Baker's discussion in reference to condemnations for non-conformity to the specifications; he spoke of heating a plate of steel over two-thirds of the width and cooling it off with water, as if the cooling was injurious to it, or that it might be supposed that such cooling would be injurious to it. My own impression is that the cooling with water was all that saved it. I do not know what the effect would have been if he had allowed it to cool off gradually, but my impression is that it would have been a worse plate. The cooling off was actually giving it work.

My experience is that it would break at the point where the heated portion merges into the cold; so that in all such plates and bars I have

thought necessary to anneal—but annealing is hardly the right word; we assume that in annealing the plate is to be cooled off very slowly—I think the only requisite is that the plate or bar should be cooled off uniformly. If the cooling is too much prolonged the ultimate strength will be reduced and its ductility will be increased, but if it is cooled off as rapidly as it is cooled after rolling, and as uniformly, the bar will be restored to its original characteristics.

F. COLLINGWOOD, M. Am. Soc. C. E.—One point has been mentioned by Mr. Marshall that I wish to speak of. He says a great deal of trouble has arisen from engineers requiring tests of finished work.

I do not see how an engineer is safe in relying upon anything else, since in finishing the work it is subjected to the various processes of hammering, rolling, cutting, cooling, and often of reheating and forging—all tending to change the character of the material if not carefully done.

If by finished work is meant angles and bars of various sections, or plates rolled and cut to dimension, I cannot see that the testing of crop-ends or shearings by a bending test (or any other) can cause any loss to the manufacturer, as these tests can all be made at the mill. Other tests I have spoken of as made from the blooms or casts, are made for the purpose of determining whether the material is of the right character, and then the tests on each piece to see whether that piece has been damaged in manipulation.

Simply to see that the bar has not been rolled at too low a heat is enough, however, for if the steel be at all of a high grade the subsequent operations of forging into eye-bars may leave it in a very unsatisfactory condition, and this is the really difficult part of the problem. Careful inspection to mitigate as far as possible the effects of unequal heating, with a certain number of check tests on full-sized bars are all that can be done. It would seem to be desirable that all work of this character be annealed throughout when in finished shape. The specifications should be explicit as to the amount of testing to be done, and then the manufacturer should consider it a part of his prime cost.

WM. SELLERS, M. Am. Soc. C. E.—There is one word that I omitted to say. I omitted to say anything upon that question of drilling. I find that the universal practice in Europe is to drill all plates, and

undoubtedly such plates are not injured in that way. But there is one thing in connection with them that is not ascertained. My experience on the East River Bridge leads me to believe that it is better policy to punch all work that can be punched, because with steel made in this country there is no absolute certainty that the same character of metal will be found all over the plate or bar. Now if it is punched and there is an excess of manganese it will break. Occasionally we came across bars that would not punch, but broke like glass, and invariably this was caused by a local excess of manganese. If drilled we would not have discovered that there was this excess of manganese, so that I prefer the punching and reaming to drilling. The drilling is cheaper, but not quite so safe as punching and reaming.

P. ROBERTS, JR., M. Am. Soc. C. E.—I wish to call attention to the point which is often made, that open-hearth steel is more uniform than that made in the Bessemer converter, and the former should be the only material used in our structural work. I have seen, as yet, no tests advanced as proofs of these assertions, and, in the absence of such, doubt their correctness. The method of testing steel which at present is often used I consider totally wrong, namely, to take an ingot, roll this say into a 4-inch billet, and then reduce to a  $\frac{1}{2}$ -inch round, from which the test piece is cut, and upon the result of this test accept or reject the steel belonging to this heat, rolled into various shapes upon different mills. There would be about as much wisdom in making all tests of structural iron upon  $\frac{1}{2}$ -inch round samples, and I doubt if any condemnations would ever occur. There is more danger from improper heating of steel after being cast than from any other cause, and upon no consideration should acceptance of material be made until tests of final product be had. This is opposed by makers of steel, on the grounds of large rejections which would be made of material in a form useless for other purposes. In the case of the Cantilever Bridge, 56 per cent. of the steel made was condemned; hence they prefer the rejections to be made upon blooms, that these may be used for other purposes. If we make the sample tests, they should be but preparatory, and under no consideration final.

C. L. STROBEL, M. Am. Soc. C. E.—I have read Mr. Schneider's paper with very much interest and satisfaction. So complete and faith-

ful an account of an important engineering work is always valuable, but it is particularly so when the principles of construction involved have not often been put into practice before. Only a limited number of cantilever bridges have thus far been built, and the Niagara Bridge is the most conspicuous example of this type.

A feature of the bridge is its large deflection under load, comparing this deflection with that usual for a discontinuous bridge of the same span-length between supports. There are three causes for this: First, the use of steel, and the greater unit strains allowed for this material; secondly, the use of metallic towers for the intermediate supports; and thirdly, the great length of the cantilever arms as compared with span-length between supports.

As regards the first cause, if we assume that the modulus of elasticity for steel in compression is the same as for wrought-iron, the steel compression members in the Niagara Bridge will shorten under load  $37\frac{1}{2}$  per cent. more than they would if these members were iron. The exact value of the compression modulus for steel has not yet been determined, but it is probably nearly the same as that of wrought-iron. This cause will therefore tend to increase the deflection materially.

The second and third causes go together. The shortening of the towers under load causes a drooping of the ends of the river arms of the cantilevers which support the middle 120-foot span, and the longer these arms, and consequently the shorter the middle span, the greater will be the deflection (or lowering) of the cantilever end from this cause. It becomes maximum if the two cantilever arms in the middle bay are each made to reach over half the bay, so that their ends meet and the middle span is eliminated. It is most convenient to treat of cantilever bridges as continuous-girder bridges having their points of contraflexure fixed in position in every alternate span by the cutting of one of the chords. The deflection of a continuous girder is less than that of a discontinuous girder of the same span length between supports, and the deflection of a cantilever girder will be the same as that of a continuous girder if the fixed points of contraflexure coincide with the points of contraflexure of the latter. In the Niagara Bridge the fixed points of contraflexure do not so coincide, but on the contrary are far removed from the supports and brought nearer together, whereby the length of the middle discontinuous girder is reduced.

In Mr. C. Shaler Smith's Kentucky River Bridge, Cincinnati South-

ern Railway, the fixed points of contraflexure are located in the shore spans, and the ratio of cantilever arm to distance between supports is only 1 to 5, while in Niagara it is 1 to 2.7. The bays are equal, and a lowering of the tower supports due to the shortening of the columns under load is increased at the cantilever end only 20 per cent., while it is increased 90 per cent. in the Niagara Bridge. So far as can be judged from observations made, which were not very reliable, the deflections of the Kentucky Bridge are very moderate and probably do not exceed the usual limits for discontinuous girders of the same span length, and this notwithstanding the towers are higher by 40 feet than those of the Niagara Bridge. A part of this difference in deflection is, of course, due to the material, which is all iron in the Kentucky River Bridge. But only a very small part can be accounted for in this manner.

I have had occasion to make an accurate calculation of the maximum deflection of the channel span in the Ohio River Bridge at Henderson, now building. The girders are single system Warren, with parallel chords, practically all steel, and discontinuous. The span length is 522 feet between supports, or 52 feet greater than in Niagara. The depth of truss is 56 feet. Assuming the modulus for steel in both tension and compression = 29 000 000, the maximum deflection was found to be =  $3\frac{1}{2}$  inches, as compared with  $8\frac{1}{8}$  inches in the Niagara Bridge if the upward and downward motions of the cantilever ends are added, and  $7\frac{1}{8}$  if only the downward motion is considered.

The position of the fixed points of contraflexure in the Niagara Bridge was, of course, determined by the mode of erection adopted, and while unfavorable both as regards deflection and economy of materials, it is very favorable as regards ease, security and economy of erection.

I do not consider that the introduction of the short links connecting the last two sets of ties with the pin at the ends of the cantilever arms tends to define the strains in these ties except to a very limited extent, and I regret Mr. Schneider did not follow his preference, and adopt a single system of diagonals for the cantilevers, thereby removing all ambiguity of strain. I do not think the bridge would thereby have suffered, either in economy or constructive advantages.

Mr. Schneider attaches due importance to the avoidance of eccentricity, and in his main connections in trusses, and approximately so in transverse bracing of columns, the center lines of strains intersect in the same point at each vertex. While it is not necessary (and for the



lower chords of pin-connected bridges always difficult) to strictly conform to this requirement as regards the connections for lateral bracing, yet I think the tendency should be to more nearly approach to a realization of this requirement for these connections also than has heretofore been generally customary, though this cannot be done without additional cost.

The specifications exclude Bessemer steel. I believe this clause was not intended as a reflection on this process of steel manufacture, but was inserted merely to prevent the delays inevitable upon obtaining structural steel from works not already familiar with its manufacture. Since the building of the Niagara Bridge, Bessemer steel has been used for structural purposes with great success. After an experience with several thousand tons of it in the Susquehanna Bridge, Baltimore and Ohio Railroad, and a previous experience with several makes of open-hearth steel, I can say for Bessemer that it has been exceptional in quality. Its ductility has been greater than that of the steels previously used, and it has shown great uniformity.

The method of testing adopted for this steel was the following:

From each blow or cast a small test ingot was made, which was rolled into a  $\frac{1}{2}$ -inch round bar. Bending and tensile tests were made on this, and, if satisfactory, the ingots constituting the blow were rolled into the plates, bars or angles for which they were intended. The steel was not finally accepted on the strength of these tests of  $\frac{1}{2}$ -inch round specimens. After the rolling, specimens were cut from the plates, bars or angles themselves and tested, one tensile and one bending test being made from each furnace heat. In case of doubtful results a second test was made. If these tests conformed to the requirements of the specifications, the steel was accepted finally. In order that the tests on the  $\frac{1}{2}$ -inch rounds be a gauge of the tests on the finished product, so that if the former tests are satisfactory the latter will be, the steel should be given the same, or nearly the same, amount of work in reducing from the test ingot to the  $\frac{1}{2}$ -inch round specimen as it receives in reducing from the ingot to the finished product. The test ingot must be of suitable size for the purpose, and cold rolling must be avoided. If proper attention is paid to this, there will be few or no rejections of finished material, provided the heating at the mill is carefully and skillfully done. In a well-regulated mill, familiar with the working of steel, there should be no case of improper heating, and there is no reason why the tests on the rolled material cannot

be restricted to a few specimens, if not omitted altogether, or confined to bending tests only. It is desirable that these tests be limited to a few, as they are both expensive and consume considerable time. They are expensive because the material must be rolled with long crop ends in order to obtain the test specimens, thereby largely increasing the waste; and the specimens must be cut out in the planer, and many of them turned in the lathe, which is both expensive and requires time.

The object of testing the steel in the ingot is, of course, to save the loss and avoid the delays resulting from rejections of the finished product. The ingots thrown out as unsuitable can be used for other purposes, while the rolled material rejected has only scrap value.

This manner of testing has been found to give the most satisfactory results. If the steel manufacturer makes a practice of testing every cast in a systematic and reliable manner, there is no reason why the engineer may not adopt these tests as his own, and confine himself to the testing of a few specimens cut from the crop ends of the rolled product, for the purpose of checking the heating and incidentally also the manufacturer's tests.

For the Susquehanna Bridge, the range allowed in ultimate strength was 6 000 pounds per square inch. Steel used in tension was required to have an ultimate strength of 69 000 pounds to 75 000 pounds per square inch, and steel used in compression an ultimate strength of 79 000 pounds to 85 000 pounds per square inch.

This range is unusually small, and, I think, unnecessarily so. Abroad it is customary to allow 9 000 pounds. So long as sufficient ductility is obtained, I can see no good reason for drawing the lines very close. The percentage of rejections is thereby largely increased, and hence the cost; and considerable delay will be experienced in obtaining the steel. The average elongations for the Susquehanna steel is 24 per cent. in 8 inches for tension, and 22 per cent. in 8 inches for compression steel; the specifications calling for not less than 18 per cent. and not less than 15 per cent. respectively.

Mention has been made that it is undesirable to specify the chemical composition. I desire to emphasize this. The carbon percentage in the compression steel (Bessemer) furnished for the Susquehanna Bridge is as much as  $\frac{1}{16}$  per cent. lower than steel of the same physical characteristics heretofore used, made by the open-hearth process; and steel made by the new Clapp-Griffith process, though this is essentially Bessemer

steel, shows marked peculiarities in chemical composition. The engineer can safely leave the chemical composition to the manufacturer, as he does the mechanical contrivances of manufacture, requiring only certain physical tests of the material by which to determine whether it is suitable or not.

C. A. MARSHALL, M. Am. Soc. C. E.—Mr. Strobel has already replied largely for me to the points made by Mr. Collingwood and other gentlemen with regard to the extra expense caused by testing finished material. The expense comes in largely by delay. If manufacturers are compelled to work to the tests of finished material, deliveries will be delayed a week or two weeks longer than if buyers were content with the first test which the steel-works usually makes for itself. The aim has been to seek for such a test as will give an expert knowledge enough for him to predicate all the rest about that steel. We think we have found that test in a three-quarter round taken from bloom rolled out of large ingot. Of course engineers need to know the strength which the three-quarter round shows in relation to the strength which the final material shows. This will all be found out in the course of a year or two at the rate at which engineers are working at it now. All we ask is that you do not compel us to undergo this unnecessary delay and this unnecessary amount of testing of finished material. Now Mr. Strobel called the test on the three-quarters round a check of the heating. I think he is mistaken. The first test is the test of the material; it describes to one who is familiar with the subject the character of the material. The second test, the test on a part that has been heated and rolled, is a test of the heating and rolling. Now, as we become more expert, I feel perfectly safe in predicting that we shall require fewer tensile tests proportionally, and that we shall not require any tensile tests whatever for this purpose. A simple bending test will check over-heating or bad rolling. A simple bending test was all that was required by the Government in their specifications for cruiser steel, so far as the scheme of testing was made to apply to finished material apart from "heat" or "melt" tests.

I have not seen any specifications that are based on tests of finished material exclusively, except the foreign specifications already referred to relating to ship-building steel.

They read something like this "At least one plate in fifty shall be tested by samples cut both lengthwise and crosswise, which shall be pulled to rupture in a testing-machine," etc., etc.

Now that is wrong, because the particular plate tested may have been heated too hot in the furnace or rolled too cold in the mill, and either would make the test vary so much as to throw that piece out properly, but not the whole charge.

Mr. Sellers asks me to make a statement with regard to the mode of testing which was pursued with the East River Bridge steel. There was some attention paid to determining how uniform that material was. As I understand it, a test-piece was taken out of each end of each ingot, and one out of the middle of each ingot. These constituted the "heat" tests; anything further than this which was pursued on the East River Bridge was on the basis of checking for the heating and rolling.

C. L. STROBEL, M. Am. Soc. C. E.—I rise only to say that Mr. Marshall misunderstood me as saying that the three-quarter round test is intended to test the heat. The test from round ingots tested the material. The tests made afterwards were tests on the furnace heat at the mill. The three-quarter round test is not a test on the heating.

JAMES G. DAGRON, C. E.—I will be pleased to lay before the Society a short *resumé* of our experience in testing steel for the Susquehanna River Bridge on the Philadelphia Branch of the Baltimore and Ohio Railroad. The specifications for the steel were as follows:

"Steel used in compression members and in bolsters, bearing-plates, pins, and rollers, shall be open-hearth, and shall not contain more than one-tenth ( $\frac{1}{10}$ ) of one per cent. of phosphorus. A sample-bar  $\frac{3}{4}$ -inch in diameter shall bend 180 degrees around its own diameter without showing crack or flaw, and when tested it shall have an ultimate strength of not less than 80 000 pounds per square inch, and an elastic limit of not less than 50 000 pounds; and shall elongate at least fifteen (15) per cent. in eight (8) inches, and show a reduction of area of at least thirty (30) per cent. at point of fracture.

"Steel used for eye-bars shall not contain more than one-tenth ( $\frac{1}{10}$ ) of one per cent. of phosphorus. A sample-bar  $\frac{3}{4}$ -inch diameter shall bend 180 degrees, and be set back upon itself without sign of crack or flaw; and, when tested, shall show an ultimate strength of not less than 70 000 pounds per square inch, and an elastic limit of not less than 40 000 pounds; it shall elongate at least eighteen (18) per cent. in eight (8) inches, and show a reduction of area of at least forty (40) per cent. at point of fracture."

By these specifications Bessemer steel was excluded, but it was afterwards decided to use steel made by this process. The steel was made by the Pittsburgh Bessemer Steel Company at their works at Homestead, near Pittsburgh, and was rolled by the Union Iron Mills, Pitts-

burgh. The amount of steel required in the construction of the bridge will be approximately 5 000 tons.

At first tests were made on specimens  $\frac{3}{4}$  inch in diameter, and no upper limits as to ultimate strength were imposed, but in a short time, finding such considerable variations in the different blows presented for test, it was thought necessary to impose superior as well as inferior limits of ultimate strength. Tension steel was accepted with an ultimate strength of 69 000 to 75 000 pounds, compression steel being accepted when its ultimate strength ranged from 79 000 to 85 000 pounds, if the other conditions were complied with as to elastic limit, elongation and reduction of area.

The method of testing adopted was as follows, each blow\* and each heat† being tested:

(a) The blow test was made on a  $\frac{3}{4}$ -inch round bar rolled from a 4-inch ingot, two pieces being cut from the bar, one to be subjected to the bending test, the other to the machine test. If these tests were satisfactory, all ingots cast from the blow were accepted and shipped to the rolling mill; if the tests proved unsatisfactory, all the ingots were thrown out.

(b) The heat test was made on specimens cut from the finished material, one or more specimens being prepared from the material contained in each furnace.

These tests were made to ascertain whether the steel had undergone any change in the manipulations which occur in its passage from the ingot to the finished shape. One test on the finished material, if satisfactory, accepted the heat; if unsatisfactory, a second test was made, which determined the acceptance or rejection of all material rolled from that heat.

Chemical analyses of each blow as to carbon, phosphorus and manganese were furnished by the steel-works. These show a remarkable uniformity of chemical composition in the steel furnished. The following table shows the variations of the four elements:

Carbon	Tension steel.....	.20 to .23
	(Compression steel.....	.26 to .29
Manganese.....		.54 to .99
Silicon.....		.023 to .058
Phosphorus.....		.066 to .090

\* By blow is meant the number of ingots cast by one "blowing off" of the converter.

† By heat is meant the number of blooms charged into the furnace at the rolling mill to be heated prior to rolling into finished material

I do not agree with Mr. Marshall as to the  $\frac{1}{4}$ -inch round test being sufficient to determine the quality of the material. The material put into the structure is not in the shape of  $\frac{1}{4}$ -inch rounds, but in that of plates, bars, angles and channels and therefore, it is in specimens cut from these shapes that it should be tested. The advantage of making the  $\frac{1}{4}$ -inch round test is that it prevents the shipping to the rolling mill of large quantities of steel which would have to be rejected after test in the finished sections, thus avoiding unnecessary delays.

Our experience in the present case confirms these views. At first we rejected forty (40) per cent. of the material, but after establishment of the upper limits the amount of rejections steadily decreased, and at present does not exceed four (4) per cent., showing that by a strict enforcement of the specifications, manufacturers can be brought to furnish any required grade of material.

I tested to destruction last winter, at the shops of the Keystone Bridge Company, several full size steel compression members, and during the progress of the work have made many tests of steel eye-bars. The results obtained and also the data furnished by the specimen tests I hope to lay before the Society at some future date.

THEODORE COOPER, M. Am. Soc. C. E.—Manufacturers of steel, and those who work it into bridges, naturally complain of any restrictions which put them to additional trouble and expense. But they must remember that, as engineers, we are responsible for the condition of the finished structures. Unless these are satisfactory, it matters little how the steel may have tested in the ingot form. In this form it may be the most perfect material that human ingenuity can accomplish. We cannot stop here, but must insist on following it into its final condition, and know its condition then. The physical tests of the finished material must be the criterion upon which we must place the final decision.

The responsibility of delivering us this finished material of a satisfactory quality must rest with the manufacturer. He must select the proper ingot material, and must determine the proper manipulations to produce the final result in the best manner. He should, therefore, be entirely unhampered by any requirements as to the chemical or physical processes. It will then be his fault and not ours if he fails. I think, however, the engineers and manufacturers can mutually aid each other by noting the chemical properties of the steel and its physical char-

acteristics during the several processes of its manufacture. When these are more fully understood, not only will the manufacturer be benefited by a greater certainty in obtaining the required product, but the engineer will be able and justified in relaxing the onerous tests now absolutely necessary. The desired quality of steel can then be determined at the ingot stage, and the effects of the physical manipulations determined by simpler tests on the final material. The relative amount of such tests being greatly reduced as the processes become more constant and reliable.

It is not enough, now, for the manufacturers to assure us that they can make a constant product; we also demand that they must furnish us with the evidence that it is so.

The mistake usual in steel specifications, in my opinion, is that only one kind of steel is asked for.

The steel that is best suited for a rivet-girder, eye-bar or column is very different. Ingots that may be too hard or soft for one, may be the best for some other. If the latitude covering these varieties was properly defined, the steel-maker would not be compelled to suffer such a large percentage of rejections as under the usual specifications. He could select his ingots according to their grades, and use them for the particular material for which they are adapted, from the softest roof bars to the harder compression forms.

At present the manufacturers have no just cause of complaint in regard to the expenses of testing. They desire to extend the use of this material as a business matter, and to make a profit from its sale. They are in the same position as other manufacturers who desire to obtain or extend the market for their products—bound to expend a portion of their investment to make known and establish the quality and merits of their products.

F. COLLINGWOOD, M. Am. Soc. C. E.—One point I would like to mention from the experience on the East River Bridge, which was much the same as that of the Baltimore and Ohio road. We had at first very large rejections, but afterwards very little was rejected, which shows that when we demand the highest tests and the best materials we can get them. In reference to the point why we should demand tests of finished material, I would like to cite one instance. There was a controversy in connection with some buckled steel plates for the Franklin

• Square Bridge, and it was a question whether we should accept them. On visiting the mill, by request of the Inspector, I went out to the place where a number of rejected plates were piled and asked the foreman to bring a man with a sledge. One of the plates was laid on the ground, and struck, and it went to pieces at the first blow. He struck another plate with all his force 6 or 8 blows, and it stood. Now it would not have done to have accepted that material from a simple inspection without test. I suspect that a series of plates had been turned out at once, and this happened to be the outside plate, and it had cooled too quickly. It had hardened, and was unfit to use.

WM. SELLERS, M. Am. Soc. C. E.—There seems to be some misapprehension about the use of steel. Mr. Cooper insists upon the most rigid inspection. I suppose every manufacturer who wants to produce good work desires the most rigid inspection, but testing is expensive, and is excessive without adequate results. The present practice in the Bessemer works is to cast the heat into three ingots, and these are rolled out into what is called the ingot bloom. Now, when specimens cut from the end and the middle of this bloom are tested, it is fair to presume that these would indicate the quality correctly. If these stand the test nothing can happen to injure the material afterward, except over-heating in the final working. Now over-heating can be tested by the bending test as well as any other way, and if the engineer is fully satisfied that the three tests of the ingot fulfill the requirements of the specifications, he ought to be satisfied that the bending test will test the quality of the heating that preceded the rolling, and all other tests might properly be dispensed with. The plate Mr. Collingwood cited as being too rapidly cooled must have been a buckle plate, and I do not think it was exposed too rapidly to the air. The trouble was that it had been over-heated preparatory to shaping, and had not received sufficient work to bring it back to a good condition. If over-heated, the mere shaping or bending would cause no reduction, and give the material no work. In that case the quality of the steel would be injured undoubtedly. The chemical quality of that plate was the same as the other plate that stood all the punishing that could be given.

F. COLLINGWOOD, M. Am. Soc. C. E.—I think that Mr. Baker takes the same view, that after the testing of the quality, the bending test is sufficient.



T. COOPER, M. Am. Soc. C. E.—My own faith in the bending test for our structural material is very strong, but at present we need something more for steel.

P. ROBERTS, Jr., M. Am. Soc. C. E.—I believe there is one other test that has not been mentioned in connection with steel, and which, I think, is as important as any other, and that is the drop test, which, in regard to our structural material, is not used. I think the drop and the bending tests will tell us some points which the laboratory and testing-machine will not. But there is one other thing. If we could have a standard test and not every engineer have his own hobby—if we had a standard specification to work to—we could make our steel conform to that much easier than to be continually changing our material to meet the requirements of each new set of specifications issued.

T. COOPER, M. Am. Soc. C. E.—This is very desirable, but impossible with our present limited knowledge. Some weeks ago a prominent steel-worker said to me, "Why do you not get up a general specification for steel and present it before the American Society of Civil Engineers for discussion?" I answered that I did not know enough yet to make a specification. Finally I did block out the general form of a specification for the steel in its finished shape, and sent a copy to the most prominent manufacturers of structural steel with a request for suggestions and criticisms, that it might be put in a desirable shape for presentation to this Convention.

Not a single one of the recipients has had even the courtesy to acknowledge its receipt. The only thing I have heard has been from one of their representatives, who tells me his company do not know themselves what to say in reference to it.

Now this is about the condition of the best posted ones, still seeking information. If the manufacturers, with their facilities for observation and experiment, are loath to express an opinion, can they blame engineers for being cautious and for demanding the most rigid tests and inspection.

O. CHANUTE, M. Am. Soc. C. E.—Before the discussion on Mr. Schneider's paper closes, I would like to call attention to one point concerning the foundation of the Cantilever Bridge at Niagara. That

bridge, as we all understand, is founded upon a mass of loose stones which have tumbled down from the cliffs, and extend, as is believed, to the bottom of the river, and yet under that heavy structure everybody agrees that the foundation is admirable. I quite agree with them myself, but I want to call attention to the fact that this is secured by the way in which the foundation was started on the mass of loose stone. Upon that was spread one single stone which was interlocked into every nook and cranny of the foundation, and distributed the weight of the superincumbent masonry over the entire mass of *debris*. That was accomplished by the use of beton. At the Montreal meeting you may remember that I presented a paper showing what had been done by the use of that material on the Erie Railway, and I then promised that, at some future time, I would give some particulars concerning work then in progress. This consisted of an application of beton to stop the wear of a rocky river bed. The waterfall in the Genessee River was wearing its way back, and threatening to undermine the Portage Bridge. It had cut away some hundred feet of the natural stone. We treated it to a course of beton, and I am happy to say that the wear has been entirely stopped. The artificial stone that was made from cement and sand has proved very much preferable to the natural stone that formed the bottom of the Genessee River, and we have no longer any apprehension of the undermining of the Portage Bridge. Since then the same process has been applied to the bridge at Kansas City. This was built of limestone and was injured by the frost, so that the stones composing the bridge were badly shattered. They have been wrapped up in a layer of beton, and I have no doubt that the structure is now permanently safe.

F. G. DARLINGTON, Jun. Am. Soc. C. E.—Perhaps it might be of some interest to the Members to know that the cost of the plants of the Clapp-Griffith and the Bessemer process of the same capacity is relatively as \$50 000 to \$250 000. With the Clapp-Griffith process of the same capacity as the Bessemer process the cost is \$50 000, whereas the Bessemer costs \$250 000; so that if the Clapp-Griffith process can be made to produce homogeneous results, the cost to produce the steel would be less.

Mr. STROBEL.—I would like to be informed upon what this is based. \$50 000 is the cost of what, and \$250 000 is the cost of what?

Mr. DARLINGTON.—Of the Clapp-Griffith plant compared with the plant of the Bessemer of the same relative capacity.

WM. H. BURR, Assoc. Am. Soc. C. E.—This admirable paper by Mr. Schneider, and the reports of the various experts employed to scrutinize or test the work at various stages of progress and at completion, give a very comprehensive view of a very remarkable example of modern constructive engineering.

It becomes a matter of great interest not only to examine the structure as a whole, but also to observe the detailed disposition of metal and the behavior of the latter under test.

One matter which forcibly arrests the attention, is the comparatively light load named in the specifications. In view of the rolling stock now in use on many of the railroads of this country, a 66-ton locomotive is certainly far from being a heavy one, while a train of 2 000 pounds per lineal foot is far below that of many daily passing over a number of American roads. In fact the test trains of gravel cars exceeded this weight by more than 10 per cent.

I am aware that the passage of the bridge by two trains abreast is not likely to be a common occurrence; but, as a matter of fact, over-strains by present traffic are not obviated.

The deflection records are of unusual interest and value, and afford many suggestions. The measured deflections are very considerable, and it does not seem to me that their origin is completely stated. In the first place, the average weights of the four test locomotives is more than 15 per cent. in excess of those named in the specifications, for which the structure was designed; while the test load of gravel exceeded the specification load by 11 per cent. Again, the unit-stresses in all steel members are certainly as high as our present knowledge of that material, containing 0.34 to 0.42 per cent. of carbon, will justify. The amount of strain, or yielding, within the elastic limit, of any piece of material, depends wholly on its co-efficient of elasticity, and not on its strength or ultimate resistance; and the experiments of Mr. Christie and others show that all structural steel possesses a co-efficient of elasticity not essentially different from that of wrought-iron. Now, as the compression lower chords of the cantilever and the posts of the towers are of steel, and as the permitted stresses per square inch are consequently relatively high, the correspondingly large compression strains must

necessarily induce more than ordinary deflection, particularly when it is remembered that the test load was considerably in excess of that required by the specifications.

While it is evident that the character of the structure gives more flexibility than an ordinary truss bridge, I believe that the preceding considerations account for no small part of the observed deflection.

It would have added much to the already great interest taken in this structure if some full-sized steel columns had been tested to failure. Although an advocate of the use of structural steel, and a firm believer in its inherent superiority to wrought-iron in bridges when properly used, the writer believes that the use of steel as high in carbon as that found in this instance is attended by difficulties and uncertainties that are not yet overcome. Tests of full-sized steel columns, containing about the same amount of carbon, have lately come to his knowledge, the results of which would seem to justify working stresses little, if any, in excess of those applicable to wrought-iron columns.

ERNEST PONTZEN, Cor. M. Am. Soc. C. E.—The wind pressure of 30 pounds per square foot equals nearly 146 kilograms per square meter. We know that the wind pressure can rise to about 250 kilograms per square meter. But in such cases trains will be stopped, and therefore it is quite sufficient to introduce the pressure on the train surface with the maximum of 146 pounds per square meter. But Mr. Schneider says that he calculated the wind pressure on both exposed surfaces of the trusses, and then, in a further calculation he says under the title, Loads, "upon the vertical surface of one side of the structure—" To my mind both surfaces—I mean the surfaces of both trusses—should be considered as exposed to the pressure of the wind. Perhaps I am mistaken in concluding from the second mention that he neglected the second truss surface.

I would point out also that the lattice-work above the pier, shown on Plate LXI., does not seem to be effective. If, instead of the great number of thin lattice bars, he had made a single or double system of crosses of stiff links, the purpose of counterbracing the structure at its greatest height would have been better attained.

As the Gordon formula is not used here, I cannot readily determine what co-efficient of safety is used for steel and for iron in the various parts.

Is there not danger that the punching of holes for the full size in steel plates will cause weakness? Would it not be safer to punch them of a smaller size than the rivets, and then ream them until they are one-sixteenth inch larger in diameter than the rivets? This is only a question.

C. C. SCHNEIDER, M. Am. Soc. C. E.—The propriety of using steel of 80 000 pounds ultimate strength has been questioned, as well as the propriety of allowing steel which varies 10 000 pounds per square inch in the elastic limit to be used for the component parts of the same member, for the reason that the different parts could not be depended on to act harmoniously together.

As regards the harmonious action of the pieces composing one member, I beg to differ from Mr. Christie, as it is the modulus of elasticity and not the elastic limit upon which the elongations per strain unit depend. As far as I have been able to learn from experiments, the moduli of elasticity of steels differing 10 000 pounds per square inch in the elastic limit, do not vary any more than in wrought-iron which is made of the same stock.

In regard to the propriety of using steel of 80 000 pounds ultimate strength for heavy compression members, I am of the opinion that it is proper to use that kind of steel for members of such proportions as those of the Niagara Bridge, their lengths being only  $12\frac{1}{2}$  diameters.

No difficulties were experienced in the working of the metal, and hence I see no practical reasons for not using that kind of steel for such compression members; neither do I think that the working strains allowed on steel in compression are so very wrong in proportion to those allowed on wrought-iron, it being 8 for iron and 11 for steel. If it is safe to strain iron up to one-third of its elastic limit, I cannot see why it should not be just as safe to strain steel to only one-fourth of its elastic limit.

However, I agree with Mr. Christie, that in applying Gordon's formula to steel, all the numerical co-efficients used for iron must be changed; but I can assure him that I have not been guilty of using Gordon's formula in its usual form, either for steel or iron, since I have studied his valuable experiments.

Mr. Christie's experiments prove, that for short compression members steel has a decided advantage over wrought-iron; and while in short members, such as are used on the Niagara Bridge, steel may develop its

full compressive strength, in very long members, after we reach a certain limit of  $\frac{l}{r}$ , steel columns have no greater strength than iron ones. This is as it should be, if we consider that the modulus of elasticity of steel (according to Mr. Christie's experiments) is a little lower for compression than that of iron.

Mr. Marshall requests me to state why the rivet holes in the steel compression members were not reamed, and also asks if I consider steel work like that of the Niagara Bridge as good as if the holes had been reamed.

The holes in the steel work were not reamed, because the specification did not require reamed holes. As I am not responsible for the specifications, I have to refer that question to those who made the specifications. However, I would state that I prefer reamed holes in steel work.

Even if we set aside the probable injury which may be done to the material by punching, well-fitting rivet holes are always preferable to those which do not fit so well. A rivet cannot be made to fill the hole perfectly and completely if many thicknesses of plates are riveted together, unless the holes are reamed or drilled after the parts are assembled. This applies also in a measure to wrought-iron work, and, in my opinion, it is of great importance that holes for field-rivets in iron-work should be reamed when the pieces are temporarily put together, or drilled after iron templates.

Mr. Marshall also points out the importance of comparative tests made with full-sized compression members, for the purpose of testing the merits of reaming holes. I am very glad that suggestion comes from one interested in the manufacture of steel. As engineers are generally not allowed to spend the money of their clients on experiments, I am of the opinion that it is the duty of the manufacturers who desire to introduce their material for structural purposes to give us such tests, and thus establish the properties of their products for which they are anxious to find a market.

Mr. Christie has given us very valuable experiments, made on small samples, and I hope Mr. Marshall will follow his example by interceding in behalf of science, as well as in the interest of steel, and induce the steel works with which he is connected to have a series of experiments made on large-sized steel compression members, which, I am sure, will receive the approbation of the whole profession.

In making these tests I would suggest that more attention be paid to the behavior of the material below and up to the elastic limit, than to finding the crippling or buckling strain, which, according to my opinion, is of very little practical value.

I am aware that the specifications which were used for steel leave much to be desired; but as this question has been discussed by so many able engineers, there is nothing left for me to say, except that I hope engineers and steel manufacturers will soon be able to agree upon a uniform specification and mode of testing which will be satisfactory to both parties.

Perhaps it would be well to leave it to the manufacturer to make his own sample tests, the engineer testing only the finished material. The manufacturer, guided by his experience, will be able to infer from the sample tests the properties the material will have when rolled into finished shapes. In order to test the general quality of a heat, a few samples cut from the finished material would be sufficient, while the bending test should be applied to every piece to prove that the material has not been injured in heating and rolling, for which purpose the crop ends may be used.

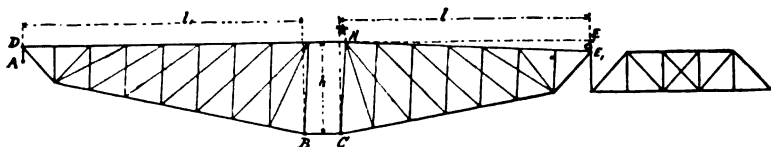
A great deal has been said about the deflections of the bridge under the test load, and various causes have been assigned for it. However, it will not be necessary to look very far for the causes of the deflections. They are natural and exactly as they should be, according to the well-known laws of elasticity.

A simple deflection or vertical motion, like the raising and lowering of one end of a fixed span, does not influence the stability of the system nor produce any additional strains, but only changes the grade of the track. The deflections, however, caused by the elastic deformation of the system, have an injurious influence, by producing secondary strains on its members.

As the cantilever system used on the Niagara Bridge consists simply of a series of ordinary trusses, the difference from other trusses being only in the way they are supported, the same law which applies to the ordinary truss supported at both ends will also apply to the trusses of the Niagara Bridge.

The strains in the river arms of the cantilevers are influenced only by the loads applied to it directly or to the intermediate span, and are not affected by the forces acting upon the shore arms, nor by the

manner the river arms are attached over the tower supports. The strains in the river arms of the cantilevers, therefore, would be the same if the cantilevers were rigidly fixed at these points. Hence it is evident that the deflection of the river arm of a cantilever, as far as produced by the elastic deformations of its members, is similar to a girder fixed rigidly at one end, the other end being unsupported. Besides the deflection resulting from the deformations of its own members, there are two other factors which add to the deflection of the river end of the cantilever.



The loads applied to the river arms and intermediate span will also produce strains in the shore arms; the upper chord of the shore arm receives tensile and the lower chord compressive strains. Point *B* is fixed, point *D* is (owing to the rocker attachment) free to move in a horizontal but not in a vertical direction. Hence for any length of the chord *BD*, the position of the point *D* is fixed, and consequently the system is stable against horizontal as well as vertical forces. If the lower chord or distance *BD* shortens, the point *D* will move towards the right. If the upper chord stretches, the point *H* will move towards the right. The distance  $\Delta$ , through which the point *H* moves, is the elongation of the upper chord *DH* plus the contraction of the lower chord *DB*. The vertical posts over the towers will rotate around their lower pins, which will produce a deflection in the river end of the cantilever. If the post *CH* rotates towards the right, the river arm will deflect downward. If the shore arm only is loaded, reversing the chord strains, the vertical post *CH* will rotate towards the left, and, consequently, raise the end of the river arm.

The vertical motion of point *E* will be:

$$EE_1 = \delta = \Delta \frac{l}{h}.$$

This deflection occurs, as is shown, independently of the strains in the river arm of the cantilever.



The compression of the tower will also produce a deflection in the river end of the cantilever. If  $\Delta_1$  denotes the reduction of length of the tower posts, then the deflection produced by it at the end of the river arm will be  $\Delta_1 \left( \frac{l+l_1}{l_1} \right)$

Investigation shows that the total deflection at the river ends of the cantilevers consists of three items:

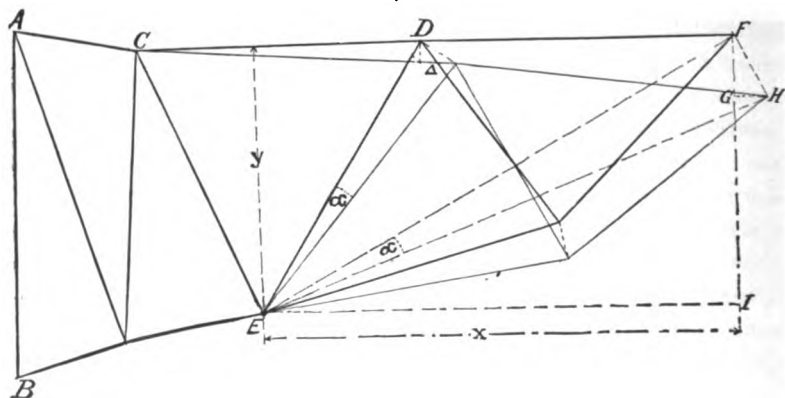
I.—The deflection produced by the elastic deformations of the river arms.

II.—The deflections produced by the elastic deformations of the shore arms.

III.—The deflection produced by the compression of the towers.

It may be interesting to compare the theoretical deflections with those observed under the test loads. I will therefore give my calculations, which are based on the same loads.

Allow me to make a few preliminary remarks in reference to the method\* adopted for calculating the elastic deformations of the river arms of the cantilevers.



Let the truss represented in the sketch above be fixed at *A* and *B*. If any member, for instance *CD*, elongates,  $\Delta$  denoting the increment, then that portion of the truss which is influenced by the elongation of that member will rotate around a point *E*, which point will be found by

\* *Civil Ingenieur*, Vol. XXVI, No. 3.

intersecting the truss by a line through the member  $CD$ , which line must cut only two other members of the truss; the intersection of the two other members determines the point of rotation. This point of rotation is the same point which is used in Ritter's method of calculating strains by static moments.

The angle  $\alpha$  around which the truss rotates is  $\alpha = \frac{\Delta}{y}$ ;  $y$  denoting the distance from the member to the point of rotation, measured at right angles from the line of that member.

The deflection of the point  $F$  in a vertical direction therefore (as will be seen by the similarity of the triangles  $EFI$  and  $F'GH$ ) is  $\overline{F'G} = \delta = x \alpha$ , where  $x$  denotes the horizontal distance from the point  $F$  to the point of rotation  $E$ .

By substituting  $\frac{\Delta}{y}$  for  $\alpha$  we have the vertical deflection of the point  $F = \delta = \frac{x \Delta}{y}$ .

These equations are general, and apply to chord members as well as to web members.

The total deflection will be the sum of the deflections produced by each member.

In the diagram on Plate LXVIII, which represents the river arm of the cantilever, are inscribed the unit strains per square inch of sectional area as produced by the test load, and which are denoted  $S$ ; the length of each member in inches denoted by  $L$ ; the increment of elongation or compression,  $\Delta$ , produced by the unit strain  $S$ , assuming a modulus of elasticity of 26 000 000 for iron as well as for steel, and the vertical distances in inches from each member to its respective point of rotation.

The deflection of the river arm due to the elongation of the upper chord will be

$$\frac{300 \times 0.1292}{312} + \frac{600 \times 0.0623}{372} + \frac{900 \times 0.0634}{432} + \frac{1200 \times 0.0658}{492} \\ + \frac{1500 \times 0.0623}{552} + \frac{1800 \times 0.0611}{612} = 0.863 \text{ inch.}$$

The deflection due to the compression of the lower chord:

$$\frac{600 \times 0.0623}{364} + \frac{900 \times 0.0965}{423} + \frac{1200 \times 0.0988}{482} + \frac{1500 \times 0.0988}{541}$$

$$+ \frac{1800 \times 0.1012}{600} + \frac{2100 \times 0.1035}{660} = 1.461 \text{ inches.}$$

The deflection due to elongation of diagonals:

$$\begin{aligned} & \frac{1260 \times 0.0849}{908} + \frac{1260 \times 0.0999}{1340} + \frac{1260 \times 0.1482}{996} + \frac{1260 \times 0.1738}{1296} \\ & + \frac{1260 \times 0.1591}{1612} + \frac{1260 \times 0.1722}{1940} + \frac{1260 \times 0.1724}{2274} + \frac{1260 \times 0.1390}{3017} \\ & = 0.958 \text{ inch.} \end{aligned}$$

The deflection produced by the compression of the vertical posts:

$$\begin{aligned} & \frac{1260 \times 0.0715}{1860} + \frac{1260 \times 0.0764}{2160} + \frac{1260 \times 0.0832}{2460} + \frac{1260 \times 0.0850}{2760} \\ & + \frac{1260 \times 0.0871}{3060} + \frac{1260 \times 0.181}{3360} = 0.278 \text{ inch.} \end{aligned}$$

The total deflection of the river end of the cantilever due to its own elastic deformation will be

$$0.863 + 1.461 + 0.958 + 0.278 = 3.560 \text{ inches.}$$

The deflection of the river end of the cantilever produced by the deformation of the shore arm is calculated as follows:

The average strain in the upper chord of the shore arm from the test load is 5 000 pounds per square inch, which makes an elongation in 220 feet of 0.423 inch, assuming a modulus of elasticity of 26 000 000.

The lower chord receives an average strain of 8 500 pounds per square inch, which reduces the length of 195 feet of the lower chord 0.637 inch. Hence the deflection of the river end of the cantilever produced by the elastic deformation of the shore arm will be

$$\left( 0.423 + 0.637 \right) \frac{175}{56} = 3.312 \text{ inches.}$$

The deflection caused by the compression of the tower:

The average strain on the tower posts produced by the test load is 5 500 pounds per square inch, which will reduce their length 0.33 inch, and hence will cause a deflection at the river end of the cantilever of

$$\frac{0.33 (195 + 175)}{195} = 0.626 \text{ inch.}$$

The total maximum deflection at the river end of the cantilever, which occurs when the bridge is loaded between the towers only, will be the sum of all the deflections, or

$$3.560 + 3.312 + 0.626 = 7.498 \text{ inches.}$$

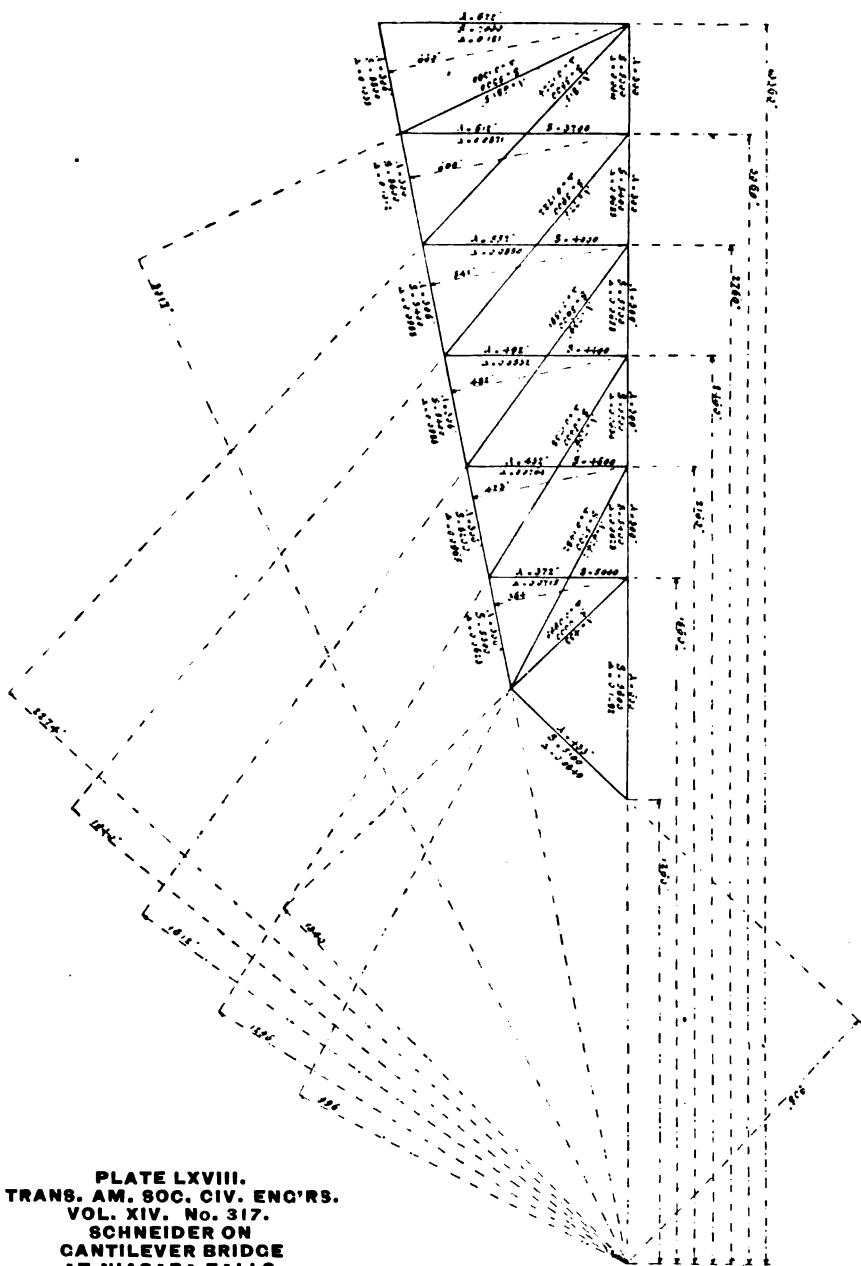


PLATE LXVIII.  
 TRANS. AM. SOC. CIV. ENG'RS.  
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 SCHNEIDER ON  
 CANTILEVER BRIDGE  
 AT NIAGARA FALLS.



Had the modulus of elasticity been assumed as 28 000 000 instead of 26 000 000, the calculation would have given us a deflection of

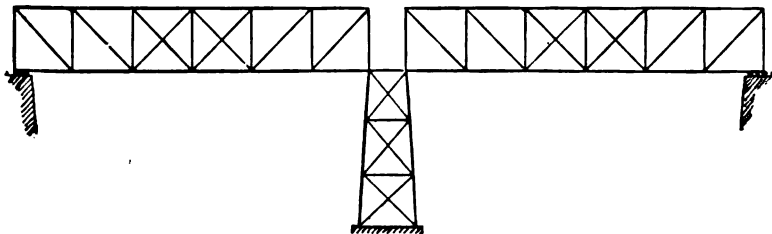
$$\frac{26}{28} \times 7.498 = 6.962 \text{ inches.}$$

These results, as will be seen, come very close to the deflections measured under the test load.

Attention has been called to the fact that the tests show that a mere loading of the bridge causes a motion of the short anchorage links, and the question is asked: What would be the effect of a test such as the one applied to the Kentucky River Bridge, viz., a train entering the bridge at full speed and suddenly brought to a stop by means of the brakes?

The stability of the bridge to resist horizontal force has also been questioned, on account of the omission of the diagonals in the panels over the towers.

I have already explained the causes of the horizontal motion of the anchorage links, it being the change of length of the lower chord of the shore arm and not any external horizontal force.



Each cantilever may be considered as consisting of two trusses (shore arm and river arm) which are fixed to the tower, and whose outer ends are free to move horizontally; therefore, each cantilever, as far as its stability to resist horizontal forces is concerned, is similar to two fixed spans, the inner ends of which rest on a tower to which they are rigidly attached, while the outer ends are supported on rollers, as shown in the accompanying sketch. In this case the tower would have to resist the horizontal force produced by the momentum of the train. The same idea applies to the cantilevers of the Niagara Bridge; the towers will have to resist the horizontal forces, and will consequently receive bending strains the same as the towers of the Kentucky River Bridge, for which ample provision has been made.

In reply to Mr. Fred. H. Smith's question regarding the twisting of the cantilever trusses under one-sided loading, I would say, that beyond measuring the deflections, no investigations were made at the time of testing. This twisting, however, can no more be prevented than the deflection. It may be reduced by increasing the depth of the cantilevers, or by reducing the length of the river arms and thus increasing the length of the intermediate span, which would have been of advantage in this case, and which has also been suggested by several other engineers in this discussion.

Mr. Fox is evidently laboring under a misapprehension in supposing that I was the manufacturer of the Niagara Bridge and not the engineer. Bridge engineers in this country not only make their own designs, as is done in England, but here they also furnish the details and working drawings for the same.

In reply to Mr. Pontzen's question, I would say that the wind pressure was calculated for the exposed surface of both trusses. The second mention of the wind strains to which Mr. Pontzen calls attention, occurs in the specifications, which were appended to my paper. For these specifications, which were not made by the bridge engineers, I disclaim all responsibility. I would also inform Mr. Pontzen that the lattice-work above the piers does not consist of thin lattice bars, but of 4" x 5" x  $\frac{1}{4}$ " angles, which are clearly shown as such on the drawing.

In conclusion, I would say that I have been much gratified by the lively interest which has been taken in my paper by engineers on both sides of the Atlantic, and more particularly by the Members of this Society. I take this opportunity to express my thanks to those who have taken part in this discussion for their friendly criticisms and valuable suggestions.