Whitney

TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XII.

JANUARY TO DECEMBER, 1883.

NEW YORK:

PUBLISHED BY THE SOCIETY.

1883.

NORTHEASTERN UNIVERSITY LIBRARY
BOSTON, MASSACHUSETTS

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

CCLXIII.
(Vol. XII.—September, 1883.)

REBUILDING OF THE MONONGAHELA BRIDGE, AT PITTSBURGH, PA.

By G. Lindenthal, M. Am. Soc. C. E.

Read at the Annual Convention, St. Paul, Minn., June 20th, 1883.

WITH DISCUSSION.

The above work having been completed under my direction, I have the honor of presenting the following account of the same, having prepared it in the belief that it will interest members engaged in bridge construction.

Smithfield Street bridge, across the Monongahela River, connects Central Pittsburgh with South Pittsburgh. The need of a bridge at this point existed early in the history of the city. A ferry accommodated the travel over the river for a long time. In 1810 a charter for a bridge was obtained, and in 1816 the first structure, a covered wooden bridge of eight spans, 188 feet each, was built. It consisted of wooden trusses, reinforced with wooden arches—for those days a remarkable engineering success.

The superstructure burnt down in 1845, and was replaced with a wire suspension bridge, the engineer of which was the late Mr. John A. Roebling, M. Am. Soc. C. E., who had then just finished a suspension acqueduct over the Allegheny River at Pittsburgh. The Smithfield Street bridge was Mr. Roebling's first road bridge.

On the old abutments and piers, after having been repaired, were erected cast-iron towers, which supported the cradled cables. The anchorages were put into the old abutments. Under each iron tower had been built up stone pedestals, 9 feet square and 3 feet thick.

The towers had the appearance of square truncated pyramids, 7½ feet square at the base and 4 feet at the top, and 16 feet high. The four-angle pedestals or columns of each tower were connected with cast-iron lattice frames, which filled the entire space on the road and river sides ; on the two other sides openings were left for the sidewalks, which led through the towers. The columns were surmounted by massive cast-iron top-pieces, in which the pendulums for the cables were hung with a slight inclination corresponding to the cradling of the cables. Each pair of towers was connected at the tops with a horizontal 12 × 16 wooden brace, which prevented the towers from being pulled towards each other by the cradled cables. The roadway was 20 feet wide, having two (2) car tracks.

On the outside of the cables were the sidewalks, 5 feet wide. The hand-railing was formed of a wooden continuous Howe truss, 5 feet high, bolted to the ends of the wooden floor-beams. These were 4×16, white pine, in pairs, 4 feet apart. The planking for the roadway consisted of a lower 2-inch longitudinal pine planking, and upper 2-inch white oak planking as wearing surface. There was no horizontal or wind bracing, save the one derived from the stiffness of the plank floor. Across each pier, under the floor, extended three wooden beams, 12×12 inches, 40 feet long, and diagonally braced from the piers below ; they were acting as cantilevers, giving some rigidity to the suspended platform near the towers. The cables, 4½ inches diameter, each contained 750 wires of No. 10 Birmingham gauge. The wires were laid parallel to axis of cable, and closely wrapped with No. 14 annealed wire.

The travel over the bridge, on which toll is collected, was of the heaviest kind, and is steadily increasing in bulk and weight. The suspension bridge had become very shaky and loose ; its continuous swaying and creaking had created anxiety in the public mind ; it had evidently outlived its usefulness. A new bridge was decided upon in the summer of 1880.

The Bridge Company at that time was in favor of another suspension bridge, but with larger spans on a higher grade, and adopted a plan proposing two (2) channel spans of 360 feet each, and two shore spans of 180 feet each. A contract for such a bridge was made, and the building of the same commenced the same summer, under the superintendence of Mr. Charles Davis, M. Am. Soc. C. E. The foundations for the channel piers were put in first, and the piers themselves built up to an average height of 10 feet. The winter season stopped any further work in that year (1880).

In February, 1881, the Bridge Company was reorganized, and the new managers desired a different bridge. It should not be subject to undulations, and should be capable of enduring the continually increasing traffic without limitation of load or speed.

At this time the writer was invited for consultation and to suggest suitable changes in the plans, which should provide for a widening of the bridge by adding another roadway or track, should this ever become necessary in the future. After having submitted such plans, they were accepted and the writer was engaged to carry them out. This plan proposed to utilize the foundations and piers which had been commenced. They were to be built upon, without any offsets, to a width on top of 56 feet.

As the width of the superstructure may ultimately reach 64 feet, or eight feet wider than the piers, it became necessary to let the sidewalks project over the masonry. The present width is 48 feet on the channel spans ; the room on the piers for widening the bridge was left on the up-stream side. For the channel spans Pauli trusses were proposed, 25 feet 8 inches apart, centre to centre, and the centre line of the new floor (of 48 feet width) was shifted down stream 8 feet 2 inches from the centre line of the old bridge. The sidewalk on the up-stream side was proposed to be detachable, so that the floor may be widened and the sidewalks again connected to it.

For the approaches to the channel spans, plate girder deck spans, on lighter masonry piers, were proposed. This arrangement allows of increasing the width of the bridge by simply adding more plate girders to each span on the piers which are long enough for that purpose. Being a deck bridge, it afforded an unobscured entrance view to the channel spans, the trusses of which were to rest on ornamental towers, giving to the superstructure an architectural appearance of strength and stability.

The shifting of centre line of new floor 8 feet 2 inches down stream from the centre line of old bridge allowed of erection of the new superstructure without stopping travel on the old bridge, in a manner described more in detail below.

The Pauli truss type commended itself for the channel spans in this instance, for several reasons :

1. The pleasing appearance (for a city bridge) in comparison with the ordinary parallel chord truss.

2. The fact that the trusses could be made high in the middle (without detriment to their stability in case of high winds), thereby reducing the chord strains and chord sections. In connection with the light and slender web-members, it permitted of an economy in the trusses of over 9 per cent., as compared with parallel chord trusses (with inclined end posts) of same height (50 feet). The deflection and vibration of high trusses is small and their rigidity great.

3. The bottom chord or cable is exposed to the sun's rays as much as any other truss-member; therefore unequal temperature effects in the trusses are avoided. The covered floor construction is independent of the trusses as to temperature effects.

4. The floor had to be cambered 18 inches in each 360-foot span to agree with the general grade of the new bridge. A straight bottom chord with a rise of 18 inches in 360 feet was undesirable.

At first it was proposed to build the new structure 15 feet higher at highest point than the old bridge. But the river men, in the interests of navigation, demanded the structure to be at least 20 feet higher, or 57 feet above low water mark, to which the Bridge Company objected, on the ground that the additional 5 feet height would injure travel over the bridge much more, by reason of a steep grade at the Pittsburgh end, than it would benefit navigation.

There is no statute prescribing the height of bridges over the Monongahela River. The case was taken to a court of equity, and argued there by lawyers *pro* and *contra*, resulting in a preliminary injunction against the Bridge Company building the bridge lower than 20 feet. To continue the litigation would have required much time. After a suspension of work at the bridge for 10 months the Bridge Company decided to accede to the demands of the river men.

The following is a description of the material and methods used in the construction of the bridge:

Masonry

consists of a gray, hard and durable sandstone, free from admixtures of clay or iron oxide particles. It was quarried near Homewood, Pa., on the Pittsburgh and Lake Erie Railroad, where it is found in large blocks of 100 to 500 cubic yards, without any stripping. The masonry is rock-faced, with drafts 1 inch wide all around the face of the stones, which are in courses of alternate headers and stretchers.

The dimensions of the stones are 24 inches to 16 inches in thickness, 7 feet to 4 feet in length, 3 feet to 1¾ feet in width, with beds and joints dressed regular and true. The backing for the abutments and wing walls consisted of regular-shaped stones, with dressed beds; for the heart of the piers concrete filling was used. It was applied in layers of 12 inches thick. It proved superior in every way to ordinary stone backing. Iron clamps bind the stones in the pier heads in every course.

The use of spalls was not permitted in any part of the masonry. All spaces between stones were filled with concrete, rammed with iron rammers, making every course absolutely water-tight. Great attention was given to the bond. The stone blocks were laid in alternate header and stretcher courses, which made the coincidence of stone joints in the heart of the pier impossible. In this way each stone is bonded in every direction. The concrete backing, after setting, was very hard and tough; it adhered to the stones with great tenacity, and made the piers monolithic in fact.

In the execution of the work care was taken to wet every stone immediately before setting. When laid in position the stone was settled by repeated blows of a heavy wooden ram. Any stone breaking under this operation was removed.

The face joints of the finished masonry were cleaned out to a depth of 1 inch, and thoroughly moistened, and caulked with Portland cement and sand mortar, mixed one to one.

For all face masonry exposed to the weather American Portland cement was used for the mortar; for concrete backing and foundations, Rosendale cement was ordinarily used.

All cements were required to be so finely ground that 90 per cent. of the whole would pass through a sieve of 50 meshes to the lineal inch. Tests as to its tensile strength were conducted on a Fairbanks testing machine with moulded briquettes of pure cement.

Rosendale cement made of a stiff paste, having been one day in water and one day in the air, at an even average temperature of 70 degrees Fahrenheit (in a room), were tested to show a tensile strength of at least 40 pounds per square inch.

American Portland cement briquettes, under same conditions, were tested to show a tensile strength of at least 80 pounds per square inch.

Similar briquettes, after having been four days in water and one day in the air, at the above average temperature, were tested for a tensile strength of 60 pounds per square inch for Rosendale cements, and 150 pounds for American Portland cement.

The concrete used throughout the work was composed of 2 parts of sound broken stone, passing through a 3-inch ring; 2 parts of clean gravel from the size of a pea to 2 inches diameter; 2 parts of washed river sand; 1 part of Rosendale cement of accepted quality.

For concrete under water 2 parts of cement were used to allow for waste by washing in depositing it under water. With a little care in the operation the loss, however, was insignificant. The stone, gravel and sand were first mixed on a board platform, then the cement added, and the whole mass thoroughly rehandled in a dry state. Water was then added in barely sufficient quantity to reduce the whole mass, by lively and severe shoveling, to a stiff mortar. This was put immediately in place in layers of not over 12 inches thick, and thoroughly rammed with iron rammers about 5 inches square and weighing 36 pounds, until the mass flushed uniformly over the whole surface.

For depositing the concrete under water for the pier foundations square wooden troughs were used, reaching down to almost the bottom, and the concrete dumped in and raked even with iron rakes having long handles. The running out of the concrete was prevented by sheet piling. When a change in the masonry of Pier No. 4 required the removal of a few stones they were found to form with the concrete backing one solid mass, which had to be rent asunder with steel wedges and sledge-hammer, and would sometimes break through the stone rather than through the concrete.

Openings or slots for one car track were left in the new abutments and piers to accommodate travel on the old bridge.

The pier posts of the channel spans on the down-stream side have their bearing near to the pier ends, and to prevent cracking of the channel piers or uneven settlement after the superstructure should be in place, riveted iron anchors were walled into the top of piers Nos. 2, 3 and 4.

The coping on the piers, consisting of two projecting courses of cut stone, was nearly all in place for a grade 15 feet higher than the old bridge, at the time of the dispute with the river men. When the height of the piers was increased to suit a grade 20 feet higher than the old bridge, the additional masonry was built on top of the coping in the form of pedestals of cut stone.

After the erection of the superstructure had so far progressed that travel could be turned on to one track on the new bridge, the old bridge was abandoned, and the gaps and openings in the masonry of the new abutments and piers successively walled in and closed. In this wise it was possible to complete the masonry work without stopping travel on the old or new bridge.

Superstructure.

The roadway is at present 22 feet 10 inches wide in the clear, and two sidewalks each 10 feet in the clear. The full width of the bridge on the deck spans of approaches is 43 feet 6 inches, and on the channel spans, which are through spans, 48 feet.

The bridge can be widened out, if ever required, to 64 feet. This made it necessary to erect the present superstructure nearer to the down-stream end of the piers. It detracts much from the appearance of the bridge, which is unsymmetrical at present.

It was important not to stop travel during the rebuilding of the bridge. Passengers and freights from and to the Pittsburgh and Lake Erie Railroad must pass over it. Besides, there is a heavy traffic in coke, iron and other mill material, which would have been compelled to take a long, roundabout way. The construction of the superstructure had to be arranged to allow of the erection first of one track and then of the other.

If the new bridge had really been built 15 feet instead of 20 feet higher than the old one there would not have been left height enough near the ends of the channel spans for teams to pass under on the old bridge. It was therefore intended to erect the channel spans about 5 feet higher than their proper grade, and to complete the floor and tracks of the same.

The pier posts would have temporarily rested on sand jacks, by means of which both spans, weighing about 1 600 tons when completed, could have been simultaneously lowered in a few hours to their proper grade. One track and sidewalk on the plate girder approaches on the down-stream side would have been meanwhile prepared for use. In this way travel would have been interrupted only for one day. But this operation became unnecessary when the new grade was raised 20 feet above the old bridge.

Channel Spans.

It was found that the use of steel, in the trusses at least, would prove economical as compared with wrought iron. The saving based on the prices at that time was over $21 600.

The Pauli trusses were designed with an uneven number of panels, namely, 13, in order to get two tangential points of attachment for each truss to the floor-construction, thereby securing greater longitudinal and transverse rigidity of the entire bridge frame. Roller bearings for pier posts were avoided ; the middle posts, supporting two truss ends each, have a fixed and square bearing on heavy pedestal castings on the pier. Each end post has a bearing on a 6-inch steel pin in a cast-iron pedestal on which it can rock. It is probable that very little movement takes place on account of friction on the pin, and that the posts would bend or spring. The resulting bending-moments on the end posts have been considered in proportioning them.

The projected length, 27 feet 7⅝ inches, of all panels being alike, it follows that the lengths of chords in a curved line are unlike, and if the curve were a circle or a parabola, then the angles formed by the straight chord sections would also all be unlike.

For practical reasons it is desirable to have these angles all alike, so as to have only one template for the beveled joints. This condition would prescribe the character of the curve, in this instance a sine-curve. The difference in curvature between a sine-curve and arc of a circle was found to be small ($2\frac{1}{4}$ inches). The difference in the bevel joints was inappreciable ($\frac{3}{64}$ inch). Therefore a true arc line was then assumed for the chords to facilitate other calculations.

The vertical web-members are in tension from the dead load or from a uniformly distributed live load. They will sustain compression strains only from an uneven distributed load. Near the centre of truss they are long and slender, requiring intermediate bracing, which was placed at half the truss height for the entire length of trusses.

The suspenders from trusses to floor, which were all stiffened to prevent vibration, were not made adjustable; their exact lengths were calculated to give the required camber of 18 inches to the floor construction. The truss camber was obtained by shortening the lower and lengthening the upper chord members $\frac{3}{16}$ inch, so that after erection it amounted to 2 inches.

All diagonal bars were made adjustable and single; they are strained from partial loads only. The trusses were adjusted to their proper shape by means of these ties, which received a slight initial strain.

The top and bottom chords, pier-posts, diagonal-ties and pins are of steel; all other parts are of wrought-iron with steel rivets. The calculated sections of the vertical web-members for steel were so light that for practical reasons they were all made of wrought-iron and of the same section.

Quality of Steel Used.

Every heat of steel was tested and its quality determined before any more work was done to it.

For the compression members and pins, the steel was required to stand the following tests on specimen bars $\frac{5}{8}$ inch diameter :

Elastic limit : 50 to 55 000 pounds per square inch.

Ultimate strength : 80 to 90 000 pounds per square inch.

Elongation in 8 inches : Minimum 12 per cent.

Reduction of area at fracture : Minimum 20 per cent.

Cold bending : 180 degrees around its own diameter without crack.

Cold punching of holes in flat 3 × $\frac{3}{8}$-inch bars: $\frac{3}{16}$ inch from the edge without crack or distention of metal.

All specimens and shapes were required to be finished at nearly the same heat, as it was observed that rods finished at a lower heat would give higer tension results than samples of same steel finished at a higher heat.

The Andrew Kloman firm in Pittsburgh had contracted to procure the steel and to furnish the steel shapes.

The intention was to use Bessemer steel for the compression members ; a large lot of Bessemer steel was tested, but few samples were found to stand the required tests. The difficulty seemed to consist in controlling the uniformity of the steel within close limits for quality and strength. After a while the attempt was given up and open hearth steel was substituted. No trouble was then experienced in getting a uniform grade of steel of prescribed quality.

The top chord sections consist of four leaves, which were originally designed to be each a 20-inch steel plate with $4 \times 4''$ angles for flanges. In ordering the steel it was discovered that enough plates of that width could not be procured in the required time. Therefore, the chord sections were changed to 10 inches and 12 inches steel plates, with 4×4 inch angles, composed as shown in the drawings.

Notwithstanding the great care used, the finished plates and angles were by no means a uniform product. According as they in rolling were finished at a higher or lower heat, they would have different degrees of hardness. Steel plates and angles finished at a lower heat had a smooth surface, and the noise of punching them resembled pistol-shots, while plates finished at a higher heat had a rougher surface, and there was hardly more resistance to punching than in wrought-iron.

The specifications for riveted steel work provided that the punched rivet-holes, $\frac{3}{4}$ inch diameter, should in the assembled parts be enlarged to 1 inch diameter by reaming. The time for the delivery of the steel work growing short, the question was considered whether the reaming of the holes could be avoided, to hasten the completion of the work at the shops. Messrs. Kellogg & Maurice, in Athens, Pa., had the contract for this part of the work.

To that end the following experiments were made :

Ten specimens were cut from the same steel plate $\frac{1}{4}$ inch thick ; one specimen was tested to ascertain the tensile strength of the steel in the specimen. The nine other specimens, all alike in form, were prepared as shown in sketch, for the purpose of ascertaining the effect of punching holes, of punching and reaming, and of drilling. The tests were expected to show the amount of reaming required, and whether any annealing effects from the hot rivet on the injured steel around the punched hole could be observed.

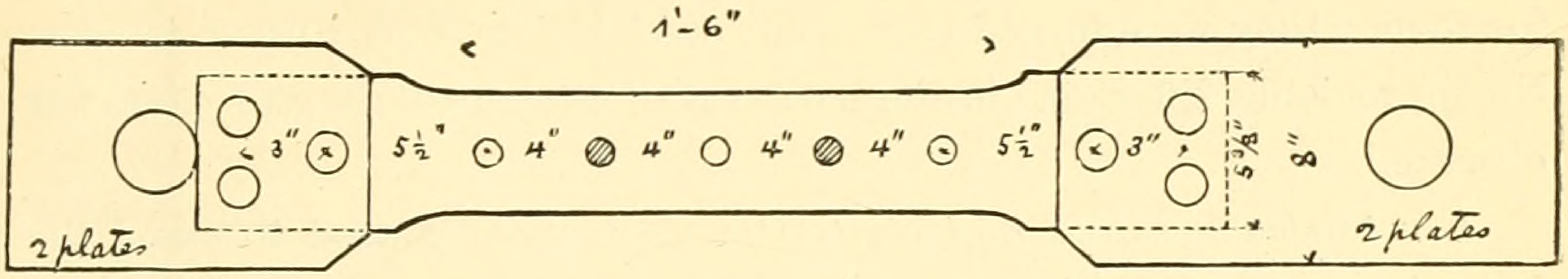

Fig. 1.

9 specimen bars $4'' \times \frac{1}{4}''$ like this.
Holes 1'' diameter, net section of bar 0.75 square inches.

			Strain per Square Inch.
			Pounds.
1. Plain specimen $3'' \times \frac{1}{4}''$..	Without holes	Broke with........., ...	89 730
2. 2 holes ◎ punched 1'' dia. 3 holes ○ punched $\frac{3}{4}''$, reamed to 1''............	No rivets in holes.	Broke in punched hole with.....	72 000
3. Prepared as No. 2....	No rivets in holes...	Broke in punched hole with........... . .	63 870
4. Prepared as Nos. 2 and 3...	Rivets in all holes...	Broke through punched hole with............	84 000
5. 2 holes punched ◎ 1'' dia. 3 holes ○ punched $\frac{15}{16}''$, reamed to 1''............	No rivets..........	Broke in punched hole with................	72 000
6. Prepared as No. 5..........	No rivets............	Broke in punched hole with................	55 200
7. Prepared as Nos. 5 and 6...	Rivets in all holes...	Broke in punched hole with..........	83 320
8. All holes drilled 1'' diam..	No rivets............	Broke in hole with.....	79 330
9. 2 holes ◎ punched $\frac{3}{4}''$, reamed to 1''............ 3 holes ○ punched $\frac{15}{16}''$, reamed to 1''............	No rivets..........	Broke in ○ hole with..	64 400
10. Prepared as No. 9..........	Rivets in all holes...	Broke in ◎ hole with...	83 320

The conclusion from these tests was that the injured steel (of the quality used in this instance) around the punched hole was in part restored by annealing in contact with the hot rivets, the size of which was large in proportion with thickness of steel plates and angles as used in the chords.

The reaming of the punched holes to a greater extent than to make the rivet holes smooth and straight was therefore dispensed with, and a reduction in the price for the finished work agreed upon.

The steel pins are 6, 5¼, 4 and 3⅜ inches diameter.

The same quality of steel as for the compression members was used for them ; they were forged from solid steel billets, and turned to size. No appreciable difference in the hardness of the metal in the pins was observed.

For tension members and rivets, the steel was required to stand the following tests on specimen bars ⅝ inch diameter :

Elastic limit: 45 to 50 000 pounds per square inch.

Ultimate strength: 70 to 80 000 pounds per square inch.

Elongation in 8 inches: Minimum 18 per cent.

Reduction of area at fracture: Minimum 30 per cent.

Cold bending : to a loop 360° around its own diameter, without crack.

Cold punching in 3 × ⅜-inch bars of 1-inch rivet holes : ½ inch from the edge without crack or distension of metal.

Open hearth steel of the above and uniform quality was obtained without trouble.

The eye-bars were made by the Kloman process, *i. e.*, the bars were rolled from billets between reversible and adjustable rolls, in such manner as to leave the ends thicker than the bar. The ends were then spread and forged to the proper shape of the eye, under a steam hammer. The heaviest steel bars for this bridge were 28 feet 6½ inches long, centre to centre of eyes, and $1\frac{1}{16}$ inches thick. All steel billets and all steel bars required very close inspection for flaws, the detection of which was sometimes difficult.

It has been stated that for the detection of flaws in steel or iron, a magnetic needle had been used with success, though the manner of its use the writer has not heard stated. A device for the certain discovery of flaws in steel bars is certainly needed. Where the solid metal sections are proportioned very economically to the work they have to do, flaws are a source of great danger, especially in attenuated steel structures ; flaws in wrought-iron are more likely to happen in the direction of the fibre, but in steel they can as well happen crosswise to the direction of the tension strain as any other way.

Three steel bars 9 feet long between centres of eyes, and 4 inches × $1\frac{1}{16}$ inches in section were tested to ascertain the effect, if any, of annealing the finished bars.

PLATE XXI
TRANS. AM. SOC. CIV. ENG'RS
VOL. XII NO. CCLXIII
LINDENTHAL ON
MONONGAHELA BRIDGE

BRIDGE over the MONONGAHELA
at
Smithfield St.
PITTSBURGH, Pa.

Scale 1/50 inch to 1 Foot

LA

Roadway 49.56
Pier No 4
Pier No 5
Pier No 6
South-abutment
Low-water line = 0
Commissioners Wharf Line
360'-0"
88'-3"
84'-9"
60'-1¼"
57'-6"
Floor construction and bracing of iron
Six iron plate girders of Equal Section in each Span

The results were as follows :

	Bar A. Annealed.		Bar B. Not Annealed.		Bar C. Not Annealed.	
	Eye.	Eye.	Eye.	Eye.	Eye.	Eye.
Diameter of eye	9''	9''	$9\frac{1}{3}$''	$9\frac{1}{8}$''	10''	9''
Least cross-section of eye	5.72□''	5.82□''	5.83□''	5.72□''	6.83□''	5.77□''
Excess of metal in eye over bar	29.4%	31.7%	38.1%	35.5%	57.7%	33.2%
Reduction of area in eye at pin-hole	7.9%	3.6%	3%	3.2%	3%	3%
Elongation of pin-hole	0.4''	0.72''	0.45''	0.44''	0.38''	0.4''
Average section of bar	4.42□''		4.22□''		4.33□''	
Average reduced area after test	3.96□''		3.97□''		3.89□''	
Reduction in percents	10%		5.1%		10%	
Reduction at fracture	43.87%		37.5%		37.55%	
Elongation of whole bar	10.5 %		10.3%		11.1%	
Elongation for 12 inches near fracture	24.6 %		23.2%		22.1%	
Elastic limit per square inch	43 140 pounds		45 360 pounds		40 940 pounds	
Ultimate strength	74 310 "		78 180 "		73 760 "	

All pin-holes were $3\frac{15}{16}$ inches diameter.

Pin-hole in one eye of bar C was bored $\frac{1}{4}$ inch out of centre line of bar, and accounts for its lower ultimate and elastic limit.

A specimen from the same heat of steel, of which the above bars were made, showed on a $\frac{3}{4}$ inch round—

Elastic limit	46 389 pounds per square inch.	
Ultimate limit	78 898 "	"
Elongation in 8 inches	18.0 per cent.	
Reduction	30.2 "	

The net section of the heads through the pin-holes for all eye-bars being at least 50 per cent. more than the bars, and the good effects from annealing being doubtful in the above tests, it was not thought necessary to anneal the steel bars.

For steel rivets the above quality of tension steel proved very suitable. The rivets were tough and tenacious.

It was, however, observed that the manufactured rivet-heads would easily break off with few blows, the fracture in each instance showing a fine granulated appearance.

Rivet heads, however, made by hand or riveting machine were very tough, and could not be broken off; they had to be cut off.

The cause for the brittle rivet-heads was supposed to be the upsetting by blows, in forming the head at a high heat in dies, producing sharp corners under the rivet-head and around the rivet-stem.

Plate Girder Spans.

There are six plate girders in each span beneath the flooring, namely, one girder under each rail and one girder under each sidewalk, which is detachable on the up-stream side.

This arrangement was chosen to admit of the erection first of the new down-stream street track, which came to lie sideways of, but on a higher grade than, the down-stream track of the old bridge. To this track travel was confined during erection. Plate girders were chosen for the reason that for the limited depth of floor (for a grade 15 feet higher than old bridge, as at first contemplated), it gave a more rigid construction than open girders of low depth. It was also more convenient to work into them, and get rid of a lot of wrought-iron which was on hand, and was left over from orders for the suspension bridge originally intended to be built.

Could the writer have foreseen that the new grade would be 20 feet higher than the old bridge, the deck spans of the approaches would have been made of two open girders of greater depth, in a manner that would have admitted of finishing both tracks on them at the same time. This would have been also more economical. As it was, the plate-girders were nearly finished when the change in grade was made.

For all wrought-iron work in the bridge the quality of iron was required to be equal to that of standard bridge iron.

Steel rivets were used for all wrought-iron bridge-members and girders.

Removal of Old and Erection of New Bridge.

The new north abutment wall was located 40 feet back of the old one. In preparing the foundation for the same it was necessary to remove the anchorage of the old cables, and to construct temporairly two anchor chains attached to the second pair of old towers. Previous to this wrought-iron anchors had been imbedded into the foundation of new pier No. 1, which had been built up to obtain the requisite weight for the temporary anchorage.

These anchor chains were composed of steel eye-bars, which were on hand from the intended suspension bridge. Each chain was made adjustable in length by means of a transverse screw rod, and four sets of eye-bars, forming a funicular machine. The chain could thereby be shortened with comparatively little power. To the cable the chain bars were attached by means of two wrought-iron plates. Between these plates were cast-iron friction clutches holding the cable, and pressed and held together with bolts passing through the plates. These were attached to the cable as near to the towers as possible. To prevent slipping of the clutches on the cable, the wire wrapping was removed, and spikes driven through the cable wires behind the clutches.

The transfer of the anchorage was done without mishap while travel as usual was going over the old bridge on both tracks. The pull per anchor chain was at times 160 tons.

Under the first north span of the old bridge, false works had been built, which, after the transfer of the anchorage, supported the old roadway, and at the same time served for the erection of the iron girders for the new bridge. No other part of the old bridge was removed till after the erection of the new channel spans.

In the false works for the latter an opening 100 feet wide near the Pittsburgh end was left for navigation, and temporarily bridged over with wooden Howe trusses. The false works were further so arranged as to clear one track on the old bridge, on which the team-travel moved in squads in alternate directions.

To prevent accidents from anything falling from above on pedestrians or teams below, the false works were covered with a platform of planks, which were afterwards used for the new floor. The upper staging was built up on the outside of, and to half the height of, the trusses to be erected; at that height a traveling derrick, 30 feet high, moved on a track of iron rails. All material for the channel spans was lifted (by a hoisting engine near the south Pittsburgh end) to the platform, on which a temporary track was laid, and all material transferred on push-cars.

With another hoisting engine, conveniently located on the up-stream end of pier No. 3, the material for both spans could be handled and put in place without moving the engines.

The Pittsburgh span was erected first. After the pier posts were put into position, the bottom chords and connecting web-members were put in place. The top chord sections, weighing from 7 to 9 tons, were picked up and placed on the verticals, one after the other, from each end in each truss. For closing the top chord, the two middle chord-sections were raised at one end till they met, and then sprung into line by pulling down these ends towards the bottom chord with block and tackle acting as a funicular machine.

The false works of the Pittsburgh spans had settled more than was anticipated. Before it was possible to close the top chords the different panel points had to be jacked up 2 to 6 inches. No such trouble was experienced with the other span.

During the erection of the channel spans no little anxiety existed at the possibility of an accident from some heavy weight dropping to the platform and breaking through to the constantly crowded old bridge below. Fortunately, the work was completed without such accident, but there were two casualties, which both resulted luckily. One man fell from a height of 80 feet into the river, but was picked up and next morning was at work. Another man fell from a height of 50 feet into shallow water; he was able to report for work after two days.

The iron floor construction was suspended to the trusses after these were swung.

The detail of connections in the Pauli trusses being simple, the erection of the steel and iron work went off smoothly, and with no more expense than in parallel chord trusses. It commenced in the middle of September, 1882, and was completed December 31st, 1882.

To the new iron floor construction the old bridge floor was temporarily suspended with iron rods on wooden blocks, laid crosswise on the bottom flanges of the stringers.

The towers of the old bridge on the down-stream side were in the way of the erection of the iron plate-girders at the south end of the bridge. These down-stream towers were removed first, together with the cable they supported. The old bridge floor where it was not suspended from the new bridge was held up on wooden trestles.

Three plate-girders in each span, supporting the down-stream track and sidewalk, equal to half the width of the new bridge, were put into position, and the paving for one car-track finished for the entire length of the bridge, without interrupting travel on the old bridge below.

Temporary wooden trestle approaches, with plank floors for one track, were built at both abutments, because the filling in would have interfered with travel on the old bridge. All this work was much retarded by a stormy and severe winter. Travel was turned over the new bridge on the down-stream track on March 19th, 1883.

During a high water, February 22d, 1883, a heavy mass of ice came down the river on a swift current, and tore away a part of the false works supporting the old bridge in a place where it was not suspended from the new one. The old bridge was then in danger of falling into the river; but by promptly suspending the old floor to the new one, first with ropes and chains, and then with iron rods, the old bridge, after one and a half day's interruption, was again safe. This was the only interruption of travel throughout the whole work.

After travel was turned on the new bridge, the gaps and openings in the abutments and piers were walled in, as stated before. The remaining old towers, cables and bridge floor were removed, and the up-stream half of the plate-girder approaches completed.

This was done by placing in position the remaining three plate-girders in each span, and the iron columns (supporting the girders near the abutments). At the same time the erection of the hand-railing and of the ornamental cast-iron tower progressed. The adjustment nuts of the diagonal ties in the channel spans were covered with ornamental castings, which prevent tampering with the sleeve-nuts.

The filling in and regrading of the approaches at both ends, and the building of the toll-houses and bridge office, were completed simultaneously with the superstructure.

The Flooring of Roadway and Sidewalk.

This consists of preserved wood, namely, gum-wood and white pine, preserved by the zinc-tannin process. On both the roadway and sidewalks the bottom planking distributes the weight on the iron and girders, so that the top sheeting or paving forms merely the wearing surface.

To the top of iron floor girders are bolted wooden bolsters, to which is spiked the bottom cross-planking, 3 inches thick for the roadway, and 2 inches thick for the sidewalks.

No provision is made to carry off the water sideways. The grade of the bridge is sufficient to carry off all surface water lengthwise. Besides, the durability of the preserved gum-wood is increased by keeping the floor moist (by sprinkling during the dry season).

The space between the track rails and in the middle of bridge is paved with preserved gum-wood blocks 3 inches thick and 3 inches high, laid with ¼-inch strips between.

Every paving block is fastened down to the bottom planking with diagonal spikes. The paving blocks for the tracks rest on a 1-inch longitudinal sheeting of preserved white pine, which serves to distribute lengthwise any uneven pressure to the cross-planking beneath. The joints between paving blocks were filled with a hot mixture of tar, pitch, rosin, lard, lime and sand in such proportions as to run freely from the ladle.

The space between the tracks and sidewalks is covered with a lengthwise top planking 3 inches thick.

The sidewalks are 9 inches higher than the roadway. The wearing surface consists of white pine, 1 inch thick, on the bottom planking of gum-wood, 2 inches thick. In the curb are openings 30 inches long, and on the average 3 feet apart for cleaning the roadway of mud and snow. Under the sidewalk, on the down-stream side, extends a box with a movable cover, the entire length of the bridge. This contains the water and gas pipes and telegraph cables. Every 150 feet are covered openings for hose attachment, provided for sprinkling the floor and for use during a fire.

A small fire occurred in April, 1883. It originated on the old bridge, and scorched the floor of the new bridge near the southern end. It showed the necessity of guarding against fire on the new bridge. All wooden flooring is to be protected by a paint of quicklime and glue water, and all crevices and joints in the wooden floor to be filled with it.

For the preservation of the lumber by the zinc-tannin process, the specifications stipulated that steaming in the curing cylinder should continue at 18 pounds pressure for four and a half hours; the vacuum should not be more than two pounds per square inch. Gum-wood should absorb 25 per cent., and white pine 12½ per cent. of the antiseptic solution under a pressure of 30 pounds per square inch. The solution is to be 5 parts (in bulk) of chloride of zinc to 95 parts of water. The lumber was to be left in it till each cubic foot of gum-wood had absorbed one and a half gallons, and each cubic foot of white pine 1.05 gallons of the anti-

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septic. After this a solution of tannin was forced into the cylinder, and the lumber kept immersed in it for 3 hours, under 80 pounds pressure.

Borings from the end of a stick which were analyzed, contained 0.379 per cent. metallic-zinc in weight, equivalent to 0.789 per cent. of zinc chloride. This was rather a high showing, as 0.5 per cent. of zinc chloride was all that was expected.

Borings taken from the middle of a stick 26 feet long, 12 inches by 6 inches, were found, on analysis, to contain 0.125 per cent. of zinc chloride, or only one-quarter of the amount intended to be injected; it is doubtful whether in long sticks the desired percentage can be attained, without very materially increasing the strength of the solution, which again would probably increase the percentage at the ends to such an extent as to render the lumber brittle after a while.

The borings were from freshly treated lumber. It is probable that the percentage is gradually increased to a limited extent in the heart of a long stick, owing to interchange of the solution by capillary attraction along the grain of the wood. The real antiseptic substance is the zinc chloride, while the tannin serves only to increase the adhesion of the precipitate to the wood fibres.

Care was required in the inspection of the lumber before treatment. Sap, loose knots, cracks, windshakes, are of course as much a defect in treated as in untreated lumber. Any unsound, weak or soft wood will not be improved by the treatment, which aims merely to make the lumber more durable, by preventing rotting. It may be added that lumber, treated by the zinc-tannin process, will not lose anything in its value as a combustible, as the experience with the fire at Monongahela Bridge proved.

The track rails on the bridge are 12 inches wide, and composed of flat bars 7 by $\frac{3}{4}$ inches, 30 feet long, riveted together with countersunk steel rivets to a section, as shown in drawing.

Splice-plates 12 by $\frac{3}{8}$ inches, 2 feet long under the joints, and $\frac{3}{4}$-inch round spikes, with conical heads, countersunk to the full thickness of the rail ($\frac{3}{4}$ inch), so that the spikes may hold down the rail, no matter how thin it may have worn.

The Ornamental Towers

are built of cast-iron, the roofs being of wrought-iron; they support merely their own weight; they incase the steel posts, which, to the eye, would seem very slender supports, and would appear out of proportion in comparison with the heavy piers and high trusses. The end posts can rock inside of the towers, which are not in any way connected with them. Where the trusses pass through the towers, room is left for expansion from temperature changes.

The architecture of the towers is so planned, and the composing parts so arranged, that the portals may be widened out to suit the entrance to a wider bridge, should it be required.

Painting.

Besides painting the metal with raw linseed oil at the mills, and iron oxide paint at the bridge shops, two coats of white lead paint were applied to the erected steel and iron work. The white lead paint was used without any dryer, and mixed with boiled linseed oil only. All joints and crevices where water might collect, were puttied all around and raw linseed oil poured in, as much as they would hold.

As the erection took place mostly in inclement weather, the shop paint came off in many places by dragging the pieces through slush and mud, which, especially in Pittsburgh, rusts iron rapidly.

Rusty places were coated with a thin lime paste, which, after drying, was scrubbed off with wire brushes and freshly painted.

All iron work under the flooring has been painted brown ; all iron and steel work above the flooring is blue. The towers have a stone color.

Loads and Unit Strains.

Beginning from the north end, there are :

1. One 40-foot span, six equal plate-girders, proportioned for a live load of 10 800 pounds per lineal foot of bridge.

2. One 81-foot span, six equal plate-girders, proportioned for a live load of 9 000 pounds per lineal foot of bridge.

3. One 87-foot span, six equal plate-girders, proportioned for a live load of 9 000 pounds per lineal foot of bridge.

4 and 5. Two channel spans, 360 feet each, two equal Pauli trusses of steel and floor construction of iron, proportioned for a live load of 4 500 pounds per lineal foot of bridge and in addition a concentrated load of 40 tons on a 20-foot wheel base for each track; of these loads the sidewalks were assumed to carry 100 pounds per square foot.

6 One span, 88 feet 3 inches, six equal plate-girders.

7. One span, 84 feet 9 inches, six equal plate-girders.

8 and 9. Two spans, 60 feet each, six equal plate-girders in each.

All of these plate-girder spans proportioned for 9 000 pounds live load per lineal foot of bridge.

The wind truss and lateral bracing under the floor is proportioned for a wind force of 400 pounds per lineal foot of bridge.

The above live loads, in addition to the load of the superstructure in the different spans, produce no greater strains per square inch of *useful* metal areas than :

FOR IRON.

8 000 pounds in compression flanges of all plate-girders, floor beams, stringers, etc.

9 000 pounds in tension flanges of all plate-girders, floor beams, stringers, etc.

8 000 pounds tension in suspenders and hangers of channel spans.

4 000 pounds shear in iron web-plates.

12 500 pounds bearing strain on iron in rivet and pin-holes.

FOR STEEL.

9 800 to 13 200 pounds in compression members.

15 000 pounds in steel eye-bars.

10 000 pounds shear on steel rivets and steel pins.

20 000 fibre strain on steel rivets and pins from bending-moment.

18 000 pounds bearing strain on steel in rivet and pin-holes.

The following quantities of material were consumed in the construction of the Monongahela Bridge :

Foundations	Lumber, feet B. M.	594 000
	Piles, lineal feet	10 800
	Concrete, cubic yards	1 280
	Iron, tons	32
Stone masonry, cubic yards		10 500
Superstructure	Iron, tons	1 070
	Steel, tons	740
	Cast-iron of towers, pedestals, etc., tons	196
	Preserved lumber for floor, feet B. M.	358 000
	Steel rails, tons,	134
	Hand-railing, 2 980 lineal feet, pounds	120 200

Approaches	Filling, cubic yards	10 000
	Sidewalk pavements, square yards	1 400
	Street pavements, " "	2 200

The total cost of construction amounts to about $160 000.

Condition of Old Suspension Bridge before removal, Tests with Iron from the same, and Conclusions.

As remarked before, the old superstructure was loose and undulating up and down about 3 feet in each span.

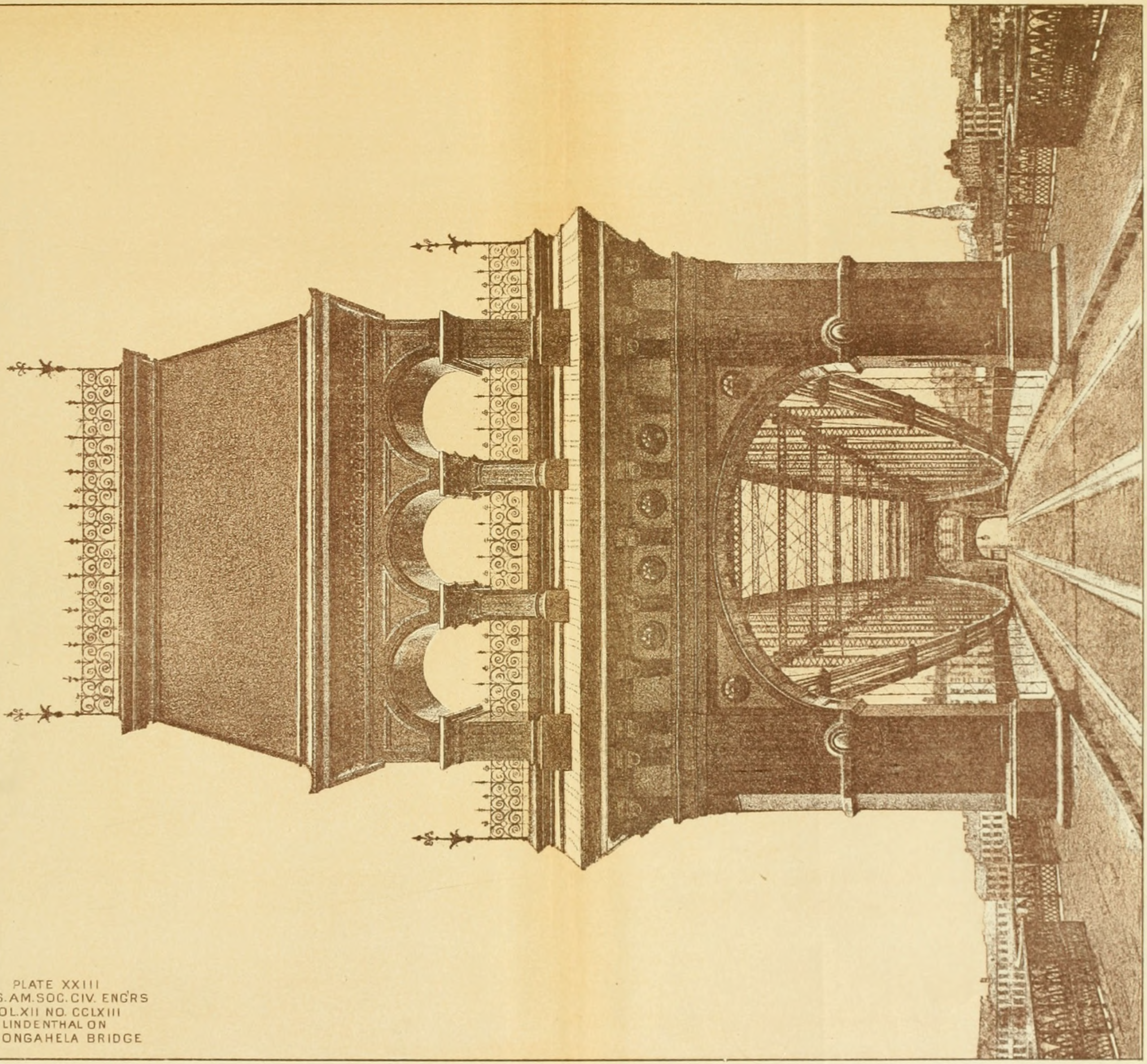
The Howe trusses at the ends of the floor beams were intended to give the required stiffness to the suspended platform, but they had become very pliable, and remained so in spite of the efforts on the part of the bridge repairers. "The splices would not hold." Nor was the cause difficult to discern.

Every passing load would deflect the wooden floor beams and raise their ends, the suspender from the cable acting as a fulcrum. At the ends of the floor beam was the Howe truss, which in this way was lifted up for a certain length. Hence, instead of distributing the load on the cable, it had just the reverse effect; it would increase the load on a suspender by its own weight for the length of several panels.

When in addition, the suspenders were badly adjusted (and it was no easy matter to adjust them right), the unfavorable conditions were still more increased. The bridge repairers had tried hard to make the Howe trusses do their duty, especially as the Bridge Company had to pay a handsome yearly patent royalty on the same, but in vain. Their value as stiffening finally consisted merely in their dead weight—about the most harmless service that could be expected of them in their position on the bridge. Nor was it surprising that the suspender rods from cable to floor would constantly break.

There were few days when no rods broke, and sometimes as many as eight broke in a day, mostly only the short suspenders (under 4 feet), which, in addition to the pull from the load and the action of the Howe truss, were subject to alternating bending strain from the vibratory motion of the floor at and near the middle of the span.

The size for the short rods had been gradually increased from $1\frac{1}{8}$ to $1\frac{3}{8}$ inches diameter, and steel had been tried for them. But experience proved that a soft, tough iron would last the longest; they invariably broke at their lower end in the thread behind the nut. If one of these



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rods lasted two (2) years it was considered very good iron. Soft steel rods lasted about 4 months. The stays $1\frac{5}{8}$ inches diameter had also become loose and useless.

The bridge, with a series of eight suspended spans, afforded the opportunity of observing the effect of an advancing load upon the cables when the bridge otherwise happened to be free from teams. The loaded span would, of course, deflect and the neighboring spans would rise. The movement in the floor seemed to the eye like an advancing wave, requiring a certain time to reach the anchorages. It strongly suggested the element of time required for a force to produce its full effect on a continuous bridge frame of greater length, whether it be a number of connected smaller spans, or one or more of greater span. Even in a suspension bridge, in which a stiffening truss of sufficient strength and in the plane of the cable equalizes the load on the same, the exact strains in both are not easily defined. For a uniform load, for instance, the stiffening truss is not needed, the material in the same is not strained, but the cable has a certain permanent strain. If now a partial load is to be prevented from deforming the cable, by means of a stiffening truss, the material in the same will be strained from zero to a certain maximum in certain places.

If we consider that the lengthening or shortening of the material takes place in an exact ratio with the strains which cause these changes of length (within the elastic limit), and that the time required for them is necessarily in the same ratio (since the strains are caused by the same load), then it follows that the full effects of the load on the trusses and on the cables take place at different times. For when the material in the trusses, from the above partial load, is strained from 0 to, say, 16 000 pounds per square inch for steel, the strains in the cable will (we assume) be raised from 12 000 to 14 000 pounds ; the ratio of the increase would be as 8 to 1.

(The ratio here assumed is depending on length of span, and becomes greater as the spans get longer and proportionately heavier. No allowance is made in this illustration for the time required by the force to travel up the suspenders to the cable.)

In other words, the maximum effect will be produced in the cable eight times quicker than in the stiffening truss. The deforming force will travel that much faster in the cable than in the stiffening truss, whatever the absolute time may be.

In this wise the writer would explain the observed movement of the cable in advance of the movement of the floor and trusses on the unloaded bridge, from a team advancing on it.

The cable would, on the entering of the load on the bridge, at once commence to move from end to end of bridge, and the grinding noise of the moving pendulums could be heard long in advance of any noticeable deformation in the floor or Howe truss. The cables were at the middle of each span close down to the floor, which could not move longitudinally. When the "advance movement" in the cable from a load occurred, the short suspenders at the middle were almost the only means of checking the extent of it; these short suspenders were subjected to a severe bending strain, as mentioned before, and would break off frequently.

Such effects of loads on a stiffened suspension bridge should receive consideration in designing the same; the writer's investigations in that direction convince him that stiffening trusses, if they were properly proportioned, are the least economical means in point of material of giving rigidity to a suspension bridge, even for longer spans, in which the inertia of the mass itself contributes to stiffness, and where the deforming loads are but a fraction of the permanent ones.

Loads coming upon the bridge in the form of shock did not seem to affect the cable much. A wagon jolting over defective rail-joints would cause severe shocks on the suspenders, but near the cable ends it could hardly be felt.

On taking down the bridge an opportunity was offered to examine the iron of some bridge parts which had been severely strained for 37 years.

The only tests which had been made by Mr. Roebling were with the wire, which had been tested with an ordinary lever beam to 1 200 pounds, and also by bending it repeatedly. So the writer learned from Col. S. M. Wickersham, in Pittsburgh, Pa., who had been Mr. Roebling's superintendent in building the Monongahela Bridge.

The old cable had 750 wires, consequently a strength of 450 tons.

The writer's calculations show that the pull in the cable from the dead

load alone was..............................	94.2 tons at the middle, 98.0 " at the ends,
and from ordinary daily loads..................	175 tons at the midde, 186 " at the ends,
from heaviest loads admitted, and occurring several times a week......................	245 tons at the middle, 256 " at the ends.

This shows that the wire was daily strained to $\frac{4}{10}$ and frequently to over $\frac{1}{2}$ of its ultimate strength. Still, on taking wires from the cable, some of them would coil up, presumably to the curve they originally had on the wire drum. " There was still life in them." Ten wires from different parts of the cable were tested in Pittsburgh, and showed an ultimate strength of from 990 to 1 340 pounds, equal to 72 700 and 95 700 pounds per square inch.

The lowest result (72 700 pounds) was with a wire from around the cast shoe on the pin. The elastic limit was from 67 100 to 78 600 pounds per square inch. Ten similar wires were sent to Mr. Collingwood, assistant engineer at the Brooklyn Bridge, and tested by him. Strange enough, the strongest wire there was from around the shoe, showing an ultimate strength of 100 000 pounds per square inch. The weakest specimen showed 91 241 pounds per square inch; this broke outside the marks. The average of the ten wires was 96 690 pounds per square inch, or over 3 000 pounds more than Trautwine gives for wire of 0.13-inch diameter.

Mr. Collingwood's results are somewhat higher ; it is probable that this may be on account of the different testing machines. The elongation was from 1.66 to 8.5 per cent. Reduction at point of fracture was from 35 to 75 per cent. A new ordinary telegraph wire of same gauge, tested in Pittsburgh, showed an ultimate strength of 99 980 pounds per square inch ; elastic limit, 81 550 pounds ; elongation, $5\frac{1}{3}$ per cent ; reduction, 57.4 per cent. The wire splices were 3 inches to 4 inches long, and consisted of the overlapping wire ends being wrapped closely with fine annealed wire. Three of them were tested ; they all broke in the splice with 980, 1 010 and 1 030 pounds respectively, or somewhat less than the full strength of the wire. The same could be bent to 180°, back again 360°, and would break then in straightening up.

The above tests proved that the cables had lost little if anything of their original strength.

One end of each cable was fastened to the pendulum pin with four (4) eye-bars 4 by $1\frac{1}{4}$ inches, and about $2\frac{1}{2}$ feet long from centre to centre of holes for 3-inch pins. The connection is shown in the sketch.

The total section of the 4 bars was 20 square inches, and assuming that they were strained all alike, the pull on them from dead load alone

was........................	98 tons, or	9 800	pounds per square inch
from ordinary loads was.....186	"	18 600	" " "
and maximum was...........256	"	25 600	" " "

But from the arrangement of the bars it is apparent that the inner cable links had to take the greater share of the pull, and before the pins had bent, must have had to resist the entire pull.

The iron in them was consequently strained far beyond the elastic limit, and they had stretched $\frac{1}{2}$ to $\frac{1}{4}$ inch, as was ascertained by measuring the length of the inner and the outer cable links. The 3-inch pins were all found to be bent, and two of them were found to be partly broken in old flaws.

The strain in extreme fibre from bending on the pin, by the pull of the cable from dead load alone, could not have been less than 46 500 pounds per square inch, and 90 000 pounds from ordinary loads, or about $4\frac{1}{2}$ times as much of what is considered safe practice in modern iron bridge work. The cable fitted over the pin with a cast-iron shoe, which in most places was found broken.

The pendulum pins on which both cables were fastened, were also all found to be bent; in all of them grooves had been worked out from the motion of the pendulum. The grooves were $\frac{3}{8}$ to $\frac{3}{4}$ inch deep; the corresponding pin-holes in the pendulum bars had been ground out to an oblong shape to the same extent.

Three pins were tested and bent on an anvil to ascertain the character of the iron; one pin broke clean off at a bend of 30 degrees, revealing a granulated fracture of fine grain on the convex side and coarser grain on the concave side.

The second pin broke in bending through a hole in the outer fibre, which was fibrous and crystalline in layers. In the hole was found a steel plug $\frac{3}{4}$ inch long.

The third pin gave the most surprising result. It bent 180 degrees around $\frac{2}{3}$ of its diameter without any crack. It had 4 grooves $\frac{1}{2}$ inch deep worked out from the pendulum bars.

All pendulum bars were 4 inches by 1 inch in section, or 16 square inches per pendulum. The pull on them was from dead load abone 56.4 tons, from ordinary daily loads 106 to 146 tons. Supposing the bars to have been strained all alike, the pull per square inch from dead load alone was 3.5 tons, and from live loads 6.6 to 9.2 tons. But owing to the bars being in pairs at each end of the pin, the inside bar had necessarily a larger pull to resist, and must have been strained sometimes to 15 tons per square inch. In addition to it was the torsion and friction on the pin.

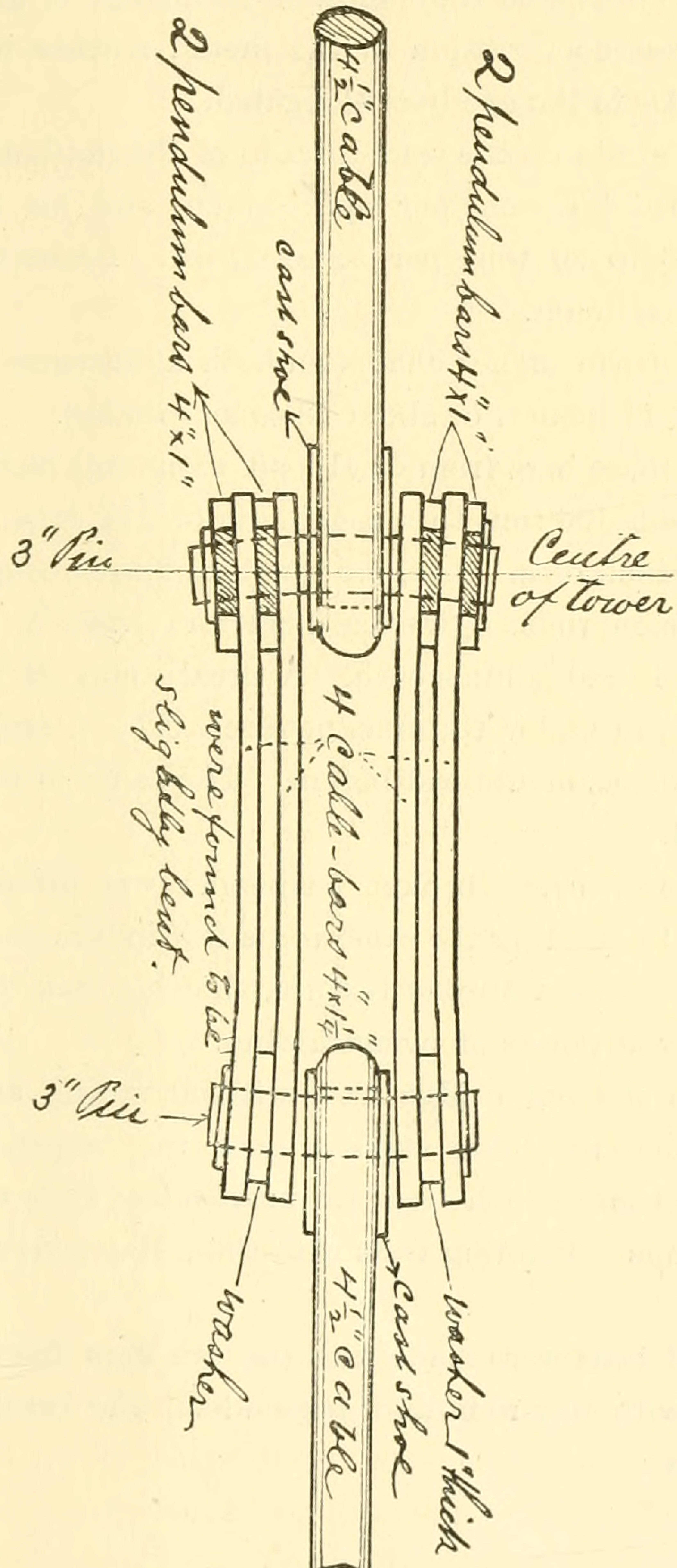
2 pendulum bars 4"×1"
4½" cable
cast shoe
2 pendulum bars 4"×1"
3" Pin
Centre of tower
4 Cable-bars 4"×1¼"
were formed to be slightly bent.
3" Pin
washer
cast shoe
4½" cable
washer 1" thick

FIG. 2.

The bearing strain on the lower pin-holes of the cable links was 6.5 tons per square inch from dead load alone; and from live loads it must have often reached 12 to 22 tons per square inch. But there was no motion on these pins, and they showed no marks or grooves, notwithstanding the great compression of the metal; neither were the corresponding pin-holes in the eye-bars elongated.

The bearing strain on the semi-intrado of the pin-hole (3 × 1 inches) was for dead load 4.67 tons per square inch, and for live loads must have reached 8.8 to 20 tons per square inch. Present practice would allow 7.5 tons maximum.

The section of the anchor-chains on both abutments was equal to 4 eye-bars of 4 × $1\frac{1}{8}$ inches, or about 18 square inches.

The pull on these bars from dead load alone was 56.0 tons, and from ordinary live loads 106 tons to a maximum of 144 tons, representing a pull per square inch of 3.1 tons, 5.9 tons and 8 tons, respectively.

The suspension rods, $1\frac{1}{8}$ inches diameter (1.15 □″), had to resist a pull of 2 500 to 12 000 pounds each. A great many of the shorter rods broke in the thread under the nut (net area, 0.7 □″), and were replaced with $1\frac{3}{8}$-inch rods, as mentioned before. The cause of the breakages has also been stated.

The fracture of every broken suspender was granular, of finer or coarser grain. In bending the same rod a few inches above the fracture, it would reveal mostly a fibrous texture, showing that deterioration had taken place only at points of overstraining.

The stays in the old bridge were without strain, and had been so many years; where the loose stays crossed the suspenders, the former were worn sometimes $\frac{3}{8}$ inch deep; the suspenders were worn but slightly at these crossings. Bending tests with these stay-rods showed a ductile tough iron.

A number of tests were made with the eye-bars from different parts of the bridge, with stay-rods and suspenders, the results of which are tabulated below :

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TESTS WITH OLD IRON EYE-BARS.

Position of Bars in the Old Bridge.	Nominal Section.	Area of Bar.	Length of Bar. Back to back of Holes.		Excess of Area in Head over Bar.	Elastic Limit of Bar. Pounds per Sq. Inch.	Elongation of Bar in Percents.	Fracture occurred at following Strain per Square Inch of Bar.	Remarks.
	Inches.	Sq. Inches.	Ft.	In.	Percents.				
Pendulum, inside link......	4 × 1	4.0	2	$8\frac{3}{16}$	$a = 12.5$, $b = 6.5$	....	3	34 850	Fractured in eye *b* before elastic limit could be ascertained. Crystalline appearance. Both pin-holes worked out $\frac{3}{4}$ inch.
" inside "	"	3.87	2	$8\frac{1}{8}$	$a = 15$, $b = 14$	31 330	6.9	47 350	Broke in eye *b*, with fibrous fracture. Both pin-holes worked out $\frac{3}{4}$ inch.
" outside "	"	3.73	2	8	$a = 16.3$, $b = 17.0$	34 000	5.3	40 840	Broke through pin-hole *a*. Crystalline fracture. Both pin-holes worked out $\frac{5}{8}$ inch.
Cable-bars, inside "	4 × $1\frac{1}{4}$	5.15	2	10	$a = 19$, $b = 25$	26 740	8.0	41 770	Broke through the eye *a* with a steely granulated fracture. Round pin-holes 3 inches diam.
" outside "	"	5.10	2	$9\frac{1}{8}$	$a = 17$, $b = 19$	31 530	10.3	44 140	Broke in eye *a* with a fibrous fracture. Round pin-holes 3 inches diameter.
Anchor-bars, north abutment	4 × $1\frac{1}{8}$	4.4	11	$4\frac{1}{8}$	$a = 27.7$, $b = 30.0$	33 090	2.92	42 230	Broke in the bar with an entirely crystalline fracture 5 feet from end. Round pin-holes 3 inches diameter.
" south abutment	"	4.52	10	$9\frac{1}{2}$	$a = 13$, $b = 16$	29 850	6.0	40 760	Broke through the eye *a*. Crystalline fracture.
" "	"	4.67	10	9	$a = 18$, $b = 19$	30 050	4.7	39 230	Broke through the eye *a*. Crystalline fracture.
" "	"	4.54	12	$2\frac{1}{4}$	$a = 22$, $b = 31$	30 280	9.6	45 790	Broke in the bar 4 feet from the end. Fracture mostly fibrous. Few crystals.
" "	"	4.84	12	$2\frac{1}{4}$	$a = 25$, $b = 31$	28 930	1.1	32 150	Broke in the bar through an old flaw. Fracture crystalline and fibrous in layers.

A cable bar 2 feet 10 inches long back to back of holes was nicked in the middle of the bar and bent; it would not break; it bent to a horseshoe 8 inches diam.

A pendulum bar 2 feet 8 inches long back to back of holes was nicked in the neck of the eye and partly bent; it would not break at a right angle; a artial fracture showed fibrous and crystalline layers.

TESTS WITH OLD IRON RODS, USED AS STAYS OR SUSPENDERS.

Position in Old Bridge.	Nominal Section.	Area of Bar.	Normal Length of Specimen.	Elastic Limit per Square Inch.	Elongation in Percents.	Ultimate Strength per Square Inch.	Reduction at Fracture in Percents.	Remarks.
	Inches.		*Inches.*					
Stay rod, mostly without strain in the bridge..................	$1\frac{3}{8}$ dia.	1.37	8	28 570	35	48 600	52.8	Fibrous fracture.
Stay rod, mostly without strain in the bridge..................	"	1.375	8	27 270	18.75	43 770	....	Fibrous fracture occurred at a worn portion of specimen.
Stay rod, mostly without strain in the bridge..................	"	1.352	8	26 380	13.87	49 720	....	Fibrous fracture broke in the grips.
Stay rod, mostly without strain in the bridge..................	"	1.375	8	29 290	33.75	48 000	55.6	Fibrous fracture.
Original suspension rod........	$1\frac{1}{8}$ dia.	1.158	8	25 710	8.75	48 380	8.8	Entirely crystalline.
Do. do.	$1\frac{1}{8}$ "	1.15	8	22 590	18.75	Fracture $\frac{1}{4}$ in. occurred at 46 150 46 820	....	Crystalline and fibrous in layers.

Conclusion from Tests.

The high elastic limit of the cable wires, sometimes so close up to the ultimate strength as to be impossible to define, and the small elongation, together with the large reduction, have been undoubtedly the result of the cold drawing of the wire, which generally has the effect (the same as in cold rolling) of raising the elastic limit and ultimate strength, bringing them closer together and reducing the elongation. In texture the iron was made steely and homogeneous.

The pliability of the cable protected the connections in the towers (which were the weakest parts) from shocks. Otherwise, the cable links and pins, overstrained as they were, would have parted under a sudden pull long ago ; as it was, all of the pins were found bent, and some partly broken.

Two of the cables, after having been in the river for six months, were found to be dry in the inside, so well had the old wrapping and paint protected them. The danger from rust is thus little more for the metal in cable form than for a solid bar, while the former is fully twice as strong. Steel and iron are strongest in the wire form.

The advantages of the cable form for tension members in long spans are, indisputable, and the judgment of Mr. J. A. Roebling (the originator) is fully corroborated by experience.

In the absence of any recorded tests with iron bars at the time of building the bridge, the conclusions from the tests stated above will be, necessarily, incomplete ; but this much can be said :

That the iron put in the bridge originally was not of a uniform quality. The bending tests on the pins and the fracture of the eye-bars prove this.

Of three pins which had been bent, each one showed a different quality of iron and texture, though they had been subjected to precisely the same strains in the bridge. The pin, doubled up in bending, proved, after being severely strained for many years, to be of a superior ductile iron, the fibrous texture of which had not been changed in any way. These pins were not only subjected to a very great bending strain, but also to torsion and friction.

The iron in the bars shows also a want of uniformity, which must have existed originally, which is plausible, considering the difficulty of making them in 1845. Each link seems to have been forged from one billet ; this is supposed from the variation in their section, not only as one compared with the other, but in the same bar. Most of the bars were full of slight flaws crosswise and lengthwise, suggesting the use of a light hammer and want of sufficient power to condense the metal, especially in the eyes, where the largest mass was. The fibrous fracture in some eyes, and the crystalline fracture in others, belonging to bars that had been exposed to like strains, proves that the crystalline texture must have been there originally, and was not a result of deterioration.

Of ten eye-bars tested, only three broke in the bar—one of them through a flaw—so that in only two bars the full ultimate strength of the iron was developed in testing. It reached 42 230 and 45 790 per square inch respectively. The former was of crystalline texture, had a high elastic limit (33 090) and low elongation (2.92 %) ; it was an anchor-bar, on which the strain per square inch probably never exceeded 12 000 pounds ; furthermore, it was never exposed to the slightest vibration in the bridge.

It is reasonable to assume that the bar was crystalline in texture when put in place. The other eye-bar, also an anchor-bar as the one before mentioned, in which the full strength (45 790 pounds) of the iron was developed, showed a high elastic limit (30 050), with an elongation of 9.6 %—a quality of iron which is often accepted nowadays for plates. The fracture was fibrous, with few crystals.

In all the rest of the tested eye-bars the ultimate strength of the iron was not developed, owing to the weak heads, which were all too small, and broke in testing. The smaller head broke in all bars ; the fracture was crystalline in some and fibrous in others. One of the bars, after being nicked, would not break, but bent, and showed tough iron.

The iron in the cable bars has been undoubtedly often strained beyond the elastic limit. As to the proportion of the heads to the bar, the tests show that where the excess of metal was below 22 % the eye broke without developing the full strength of the bar.

The original want of uniformity in the iron is also attested by the tests on the suspension and stay rods. None of them were strained more than 12 000 pounds per square inch. The fracture with crystalline and fibrous layers shows that the iron had been so put in the bridge originally. If deterioration would affect a fibrous texture it would be destroyed.

The above tests indicate that iron highly strained for a long number of years, but still within the elastic limit, and exposed to slight vibration, will not deteriorate in quality.

That if subjected to only one kind of strain it will not change its texture, even if strained beyond its elastic limit, for many years. It will stretch, and behave much as in a testing machine during a long test.

That iron will change its texture only where exposed to alternate severe straining, as in bending in different directions. If the bending is slight, but very rapid, as in violent vibrations, the effect is the same. (Mark failure of short suspension bars.) The deterioration may be explained as a loosening and destroying of the fine laminæ of slag (in the fibrous wrought-iron), causing the same to become granulated. In a fibrous iron the cohesion of the molecules is greater in the direction of the fibre than across it. When the dividing laminæ of slag are destroyed, the iron particles tend to restore an equilibrium of cohesion, as it were. Their cohesion longitudinally decreases and transversely increases, causing a condition of cohesion as in cast-iron.

Another such agency, changing the texture of iron, may be found in heat and oscillation combined. Latent heat may be evolved by violent continuous vibration itself. It seems certain that crystallization cannot take place unless the iron softens sufficiently under heat, and has rest to form crystals. Iron found to be crystallized, has either been so originally or has been made so by sufficient heat and rest in one form or the other.

The old bridge did not have any wrought-iron members in compress-iron. The cast-iron towers, the only compressed members, had 96 square inches section. The cast-iron was never higher strained than 2 000 pounds per square inch under most unfavorable conditions of loading. It was fine-grained and tough.

Two cast anchor-plates only were removed ; one of them was found with an old flaw through the entire width. It was taken from the ground whole, but in turning over and falling it broke, revealing the old flaw. It is very probable that on account of the friction on anchor-chain the full pressure against the anchor-plates had never been exerted.

The iron test specimens from the old bridge were exhibited at the International Exposition of Railway Appliances in Chicago, in June, 1883, and afterwards donated by the writer to the Western Society of Civil Engineers.

The lumber, 37 years old, in some parts of the bridge, where it was protected against rain and sun, was found to be still in excellent condition.

DISCUSSION

By F. Collingwood, T. Egleston and G. Lindenthal.

F. Collingwood, M. Am. Soc. C. E.—There are a few points in this paper which I would like to touch upon briefly, as they bear upon the work with which I have been concerned for the last fourteen years. The first one which struck me was the discussion of the question of stiffening trusses. I do not propose to take it up in detail, because it has been extensively discussed in the engineering journals, and I think there will always be somewhat of a difference of opinion among engineers as to the matter. *Theoretically*, no doubt, stiffening trusses are an imperfect device, but, practically, they do a very excellent service. As to the question of economy, that is one which, in each case, must be discussed by itself. I desire to mention, however, the action, as I observed it, about the day after the opening of our bridge in New York, of the trusses and the various connections. It may, and I think will be, of interest to the members of the Society to have it mentioned. I went to the driveways of the structure, and spent nearly the whole day in watching the effect of various loads upon it. As you may know, there has been no restriction placed upon the amount of travel, the only restriction being that there shall be no racing. Several times as many as three heavily loaded trucks came crowding along, and with horses trotting, on that structure. The only motion I could perceive, by sighting to some fixed object, was, as nearly as I could judge, a tremor—nothing which could be called a vibration. The tremor never exceeded a vertical motion of over $\frac{1}{8}$ of an inch, and I think it was less. In other words, the combined effect of the long stays and trusses was that the first effort of the vibration would stop ; it never went beyond a simple jar. Of course, when we speak of a vibration we mean a complete cycle or wave motion.

In order to test further whether there was any wave motion in the cable, I took a transit and set it in a shady and quiet place, so that the line of sight was tangent to the cable, but was not very well satisfied with my observations. I intended afterwards to continue the investigation at more length and with less haste, but have had no opportunity. I could see no motion of the cable except what I thought I detected to be a very long wave, for a period of some 3 to 5 minutes in its movement, and of but a very slight amount; but nothing which could be

PLATE XXV
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attributed to the loads on the bridge. I am not sure that I saw this long wave, because the instrument had been taken out in haste, and what I thought I detected might, quite likely, have been the motion of the instrument caused by the heat radiating to it from the walls by which it was surrounded, or other cause. I am certain we can say this—that when we come to a heavy cable like that of the East River Bridge, where the weight becomes very great in comparison with the weight passing over, we can disregard all light shocks, such, for instance, as the trotting of a heavy team.

There is another point suggested by this paper, which is of interest, viz., that, before making the connection of the long stays, and before the trusses were in position, the vibrations of the cables, although 16 inches in diameter, were very sensible. A change of position of 5 tons weight would make a perceptible change in the curve—showing that the trusses and long stays really do their work.

While upon this subject of long stays, I may be allowed to say that while very efficient for stiffening the trusses and cables, experience seems to show that it is not well to depend upon their full strength for supporting power. In making the repairs upon the Alleghany Bridge, partially burned two years since, I found that to transmit the full compression due to a full strain in the stays would require so much iron under the floor as to considerably reduce the amount of allowable live load. The floor beams were very light, but the flooring was so thoroughly laid and fastened together that its distributing power was large, and a load of 22 tons had passed over on a truck with safety. The floor, no doubt, in such a case assisted in transmitting the compressive strain.

There has been considerable said in this paper about granulation and fibre.

I believe it to be quite generally received at the present time that granulation and fibre in iron are but appearances. A piece of iron, if nicked and suddenly broken, will show a crystalline structure, while the same piece, if broken in a testing machine, may show altogether a fibrous structure.

In all puddled iron there is a certain amount of slag or other impurities, which, when the iron is rolled, produce the appearance of fibre. There is no doubt the crystals are in closer contact in the direction of the grain than across it, and for this reason the *strength* is greater also in this direction.

This does not alter the probable fact that all iron is crystalline in structure.

There has been another point touched upon, and that is as to the strength of full-sized bars. From some experience I have had, I do not think it will answer for us to criticise the iron of this structure too severely. In one of the bridges we built we had a large number of large-sized bars, and I had occasion to test some of them at full sizes. The tests on test specimens were fully up to requirements by specification, but considerably below them in the full-size bars. I had either to reject a whole lot of bars, and delay the work, or accept them and put them in. I found, as far as the elastic limit was concerned, they were fully up to the requirement—that is, up to 24 000 or 26 000 pounds per square inch, and more than double the working strain put upon them; but when it came to the full strength, instead of going up to 48 000 pounds, they only went up to 41 000 and 42 000 pounds per square inch. None of them broke, however, under less than four times the strain they would ever be under in the bridge, and I, therefore, made up my mind that I was perfectly safe in using them. The fact is, when we come to large-sized masses we cannot be sure that we are not breaking our bars, sometimes, in detail (just as we can tear a piece of cloth), and the question of a condition of strain in them caused by the peculiar heating which the heads have to undergo in manufacturing is something which has not been discussed, and is not as fully understood by engineers as it might be. I came to the conclusion, from what I saw at that time, that the only way in which we could ever be certain that our bars would not break in or near the head, even in those most carefully manufactured, would be by having the whole bar annealed afterwards. I am supported in that by the practice that obtains in the Phœnix Works, where, I am informed, they anneal all bars. A bar may be put into a furnace and a portion of it heated to a red heat, while another portion is scarcely heated at all ; the chances are that the furnace man shuts his door down upon it, and there will be a sharp line of demarkation between the heated and unheated portions : a certain part will be comparatively cold, while another part is red hot. The result of that is, inevitably, a state of strain between the parts, which is, in this case, a state of weakness.

There is another point which comes up, which is as to the state of wire used in bridges. It was a part of the experience in our bridge work to find that the wires, when brought to the test, did not all meet the requirements. Sometimes there would be two-thirds of a load, and perhaps nearly all the load, that would not come up to the test at all, but be fully ten per cent. below the required strength. Such wire would be laid aside for two or three days, perhaps a week, and then tested, and in many cases every bit of it would come up to the test. Now, the question is, what took place ? The wire had not been touched at all. The simple explanation is that the outer fibres were strained more than the inner fibres, and by letting the wire rest for a certain time there was a slow molecular change by which the strains were equalized, and the total section of the wire became of practically uniform strength. Another fact comes up in this connection—that wire which has been treated by sulphuric acid will, for a short time after removal from the acid, be much weaker. It is the occluded hydrogen which in this case seems to cause a weakness in the wire.

The writer of the paper just read has mentioned the fact that a bearing plate put in masonry may not, necessarily, be strained at all to what you calculate is the strain. In proportioning the anchor plates for the East River Bridge the puzzling fact was developed that the larger the plate the weaker it was as an anchor, other things remaining the same.

In discussing the size required I made a suggestion that the true method was to consider what was the strain upon the plates, and then, taking a very large margin of safety against crushing the stone, to figure what would be the largest area necessary to support the whole lifting strain safely. This would give the minimum size of anchor plate required ; which might have to be increased if it was found not to take hold upon enough masonry.

In reference to eye-bars, there is no doubt that we have made very great advances in the proportioning of the heads of bars to the bars themselves, and I think those who saw the specimens on exhibition at Pittsburgh will see how great the advance has been in this direction.

T. Egleston, M. Am. Soc. C. E.—Mr. Collingwood has alluded to the necessity of annealing. I wish to emphasize that fact, for I have had occasion recently, and a great many times within the last three or four years, to ascertain that the cause of weakness in a great many metals and alloys was from defective annealing and not from defect in the metal. This is very especially so when sulphuric acid is used to clean the metal, as it very frequently is. It is sometimes a dangerous thing to use acids to clean small pieces if they are left any length of time in the bath and are not subsequently put through the fire. I should like to have the attention of the Society drawn to the fact that a great many of the annealing furnaces used are worse than useless ; that in a great many instances the metal is weaker after improper annealing than if it was not annealed at all. I had occasion recently to examine a very large manufactory, where they were making alloys and annealing at such a high temperature that the metal was so "fatigued" that there was, in some cases, a commencement of a separation of the component parts of the alloy. I can also confirm, from my professional experience, Mr. Collingwood's remark, that it is not wise and is sometimes dangerous to heat parts of a piece red-hot, leaving other parts cold, without bringing all the piece up to a dull red, and then allowing the whole to cool slowly. But there is one matter that has been insisted upon, which Mr. Collingwood has alluded to also, that I do not understand—the distinction between granulation and crystallization. Granulation and crystallization are spoken of as though they were entirely distinct. I don't know of any granulation that is not crystallization, and I would like to ask how the distinction between the two is made.

G. Lindenthal, M. Am. Soc. C. E.—The distinction between granulated and crystallized iron, as I conceive it, is that the fracture of the former shows no defined crystals, but shiny, rounded, irregular grains of iron in an amorphous state ; such they appear even to be under the magnifying glass. The fracture of cast-iron has generally such granulated appearance. Perfect fibrous iron, or steel, whose texture has been disintegrated by some cause (other than sufficient heat to soften the metal) will exhibit a granulated fracture, in contradistinction to a fibrous or crystallized or homogeneous fracture. A perfect fibrous bar bent rapidly in different directions will, on breaking, show a granulated fracture similar to cast-iron ; but I never could discover any crystals in such fractures, and I cannot possibly conceive of a cause that would produce crystals in a cold, rigid body, whether iron or anything else, in so short a time, without softening of the metal. In crystallized iron there is an absence of a fibrous texture. Cast-iron is equally strong in every direction ; fibrous iron is strongest in the direction of the fibre, while crystallized iron is of uncertain strength in any direction. If you bend a fibrous bar of iron rapidly in different directions until it breaks, a granulated fracture will be exhibited, but no crystals or such defined shapes deserving that name.

T. Egleston, M. Am. Soc. C. E.—Every piece of fibrous iron that I have ever examined with a microscope was crystallized in the ends, and it was simply the drawing out of the crystals that made the fibres appear. I have never seen a fracture of cast or wrought-iron that did not show crystalline faces.

G. Lindenthal, M. Am. Soc. C. E.—Well, then, it all hinges upon the fact that granulated iron can be got more likely by taking any ordinary bar that you are certain is of a fibrous texture and bending it rapidly until it breaks. If that operation produced crystals, it would settle the fact in that way ; if it did not, it would settle the fact the other way.

T. Egleston, M. Am. Soc. C. E.—I do not think that any operation of bending would ever produce any but crystalline faces. Those faces may be flat, concave or convex. I understand that a distinction is desired to be made between the flat, concave and convex surfaces; that the flat are called crystalline, and the concave and convex granular. I have never seen fibrous iron, and do not believe any exists. All the fibres I have ever seen were an arrangement of crystalline faces. The appearance of fibre is always confined to a part of the broken end. It is a very notable fact that if almost any iron which assumes a definite structure be placed in a tube, heated, and chlorine passed over it, chloride of iron will be carried off, leaving a piece of exactly the same shape, but exceedingly light and porous, which is composed of the slag not removed in the manufacture of the iron. When this slag is large in amount, it sometimes affects the appearance of the fracture of the metal, as do all impurities which change both the texture and color. There is hardly any weld iron that does not contain a very perceptible amount of slag.

A Member.—I would like to ask one question. Were the pins which were found to be broken prepared in conformity to a scientific formula ?

G. Lindenthal, M. Am. Soc. C. E.—The bridge was built in 1845, when the art of iron bridge building was in its infancy. 3-inch pins were taken, probably, because they were thought to be strong enough. I could not discover by what formula they were proportioned. They were bent, that is all I can say. Two of th[illegible] were partly broken through flaws. How the bridge ever stood up with such pins I can't say, unless it was by sheer force of habit.

T. Egleston, M. Am. Soc. C. E.—You spoke of flaws in the pins. How were they apparent ?

G. Lindenthal, M. Am. Soc. C. E.—In different ways. One pin that I had bent to test the iron broke through a plugged hole, about ¾ inch diameter and ¾ inch deep in the middle of the pin. Perhaps some old iron that was taken had a hole in it, and it had been plugged up with a steel plug.

Mr. Egleston.—I understood you to say that the fractures in the pin were old flaws.

Mr. Lindenthal.—Yes. The two pins that were found broken in the bridge contained flaws. You could see them in the fracture.

Mr. Egleston.—Was there a rubbing where the break commenced ? If there was, this would seem to indicate that the fracture had commenced in a flaw, and been gradually propagated across the pin.

Mr. Lindenthal.—No ; the fracture was square through the pin. It was not in any way a fracture produced by the separation of the fibre of the iron. It was a fracture clear through the pin, showing undoubtedly and clearly that there was a flaw.

I wish further to remark, that if the pins had been proportioned for a 20 000 pound extreme fibre-strain per square inch, which is about the highest limit taken to-day, the pins would have to be about 5¾ inches in diameter. All these pins to which I have referred were only 3 inches in diameter. There is little wonder they were all found to be bent.

The illustrations accompanying this paper are :

Plate XXI.—Elevation and Plan of the Monongahela Bridge, showing also the lines of the former Suspension Bridge.

Plate XXII.—View of the two Channel Spans.

Plate XXIII.—The Portal.

Plate XXIV.—Elevation and Plan of Channel Spans.

Plate XXV.—Section of Floor Construction on Channel Spans.

Fig. 1.—Cut of Steel Testing Bar.

Fig. 2.—Cut showing Connections of Cable of former Suspension Bridge.