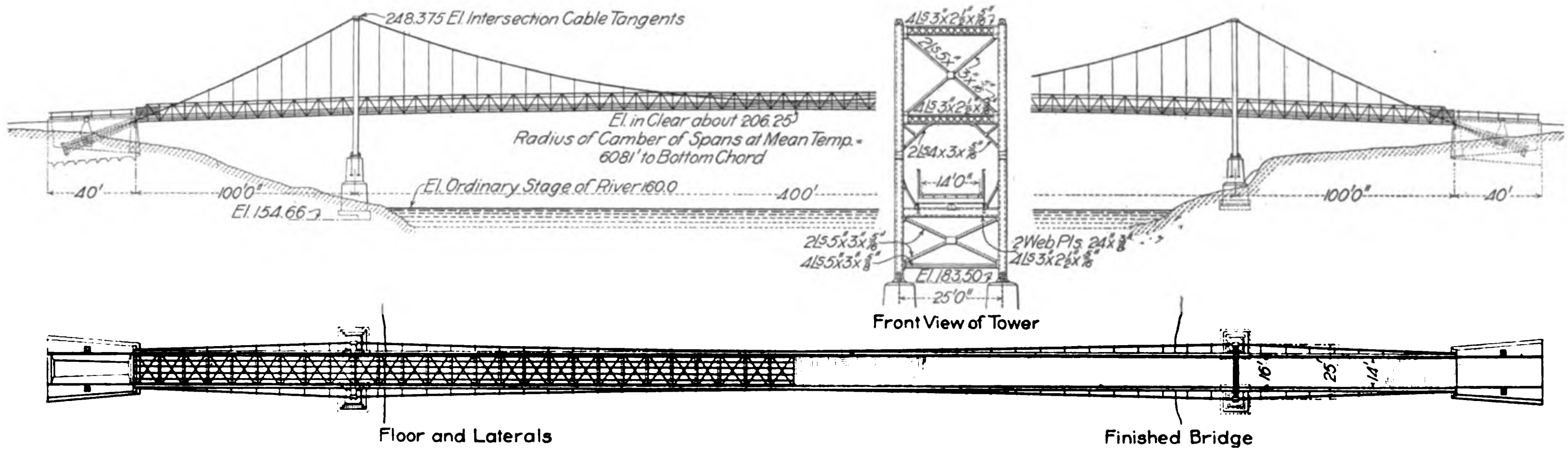
Massena Center Suspension Bridge

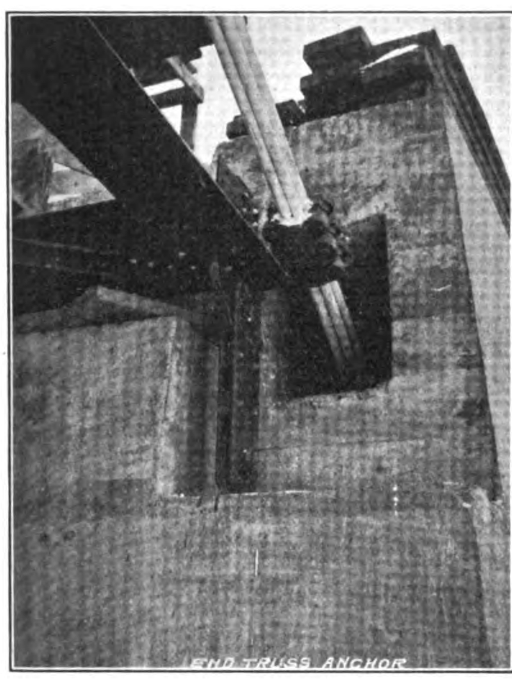
A 400-Foot Span with Twisted Cable Strands and Saw-Tooth Anchorage Footing

By John Berg, Resident Engineer

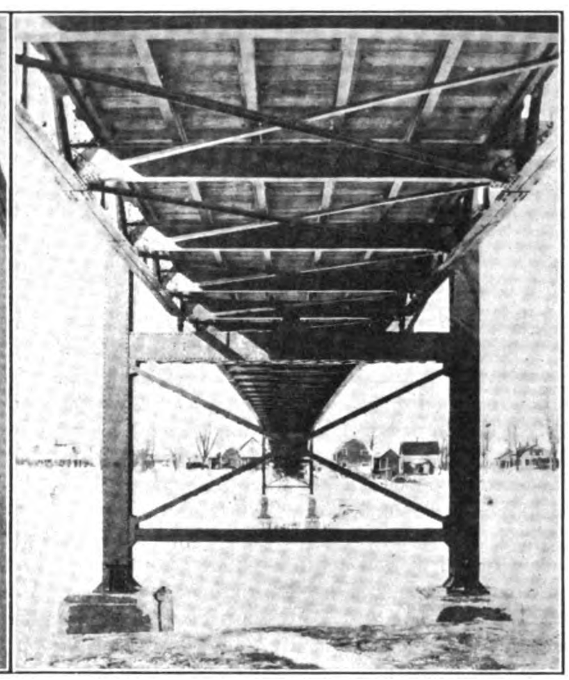
The new highway suspension bridge over the river La Grasse at Massena Center, St. Lawrence Co., N. Y., is a three-span structure with a main span of 400 ft., divided into 36 equal panels, and two side spans of 100 ft. each, divided into nine equal panels, all spans being suspended from two cables. The anchorage roadway of 40 ft. at each end makes the total length of bridge 680 ft. excluding grade approaches.



Highway Suspension Bridge at Massena Center, New York



Support of Truss at Anchorage



Lateral and Floor Systems

There has been more or less agitation for a bridge at the Center for some 40 years, but it is only within the last folr or five years that progress has been made toward getting it. The Grasse River is a tributary of the St. Lawrence and navigable to a point about three miles above the bridge. Since the construction of the St. Lawrence Power Company's canal the Grasse River has served as the tailrace for the plant.

The War Department gave its consent to a 35-ft. clearance for a distance of at least 250 ft. at ordinary high water during the season of navigation, which is at about el. 160.00. Local interests, however, wanted this clearance increased 5 ft., and, as the owners of the power canal are already making extensive additions to their development, it was considered not impossible that the ordinary stage of the water level in the tailrace might be raised to 165.00. The design of the bridge thus had to provide a clearance of 45 ft. for a distance of 250 ft. in the middle of the channel. This made a grade of 4.94 per cent necessary on the anchorage approaches.

SUBSTRUCTURE

The highest water level on record being at el. 183.50 this height was fixed as the top of the masonry piers. The foundations for these, as shown on the drawings, were carried to different elevations. On the south side of the river the excavations were carried several feet into a compact bed of coarse sand mixed with large stones. Soundings in the river channel with a gas pipe showed this to be continuous. On the north side a bed of gravelly hardpan was encountered a little above the proposed depth for the foundation. The excavation was accordingly carried a little lower than proposed, and a toe 5 ft. 2 in. long and extending for the full width of the foundations was carried down 2 ft. below the latter. The maximum pressure on the pier foundations is approximately 2 tons per square foot.

The estimated weight of the south anchorage is 1314 tons, and of the north, 1350 tons. On an assumption of a friction factor of 0.50 on a plane parallel to the foundation bottoms of the anchorages, the factor of safety of the south anchorage is nearly two, and that of the north anchorage is more than two, in both cases without considering the increased stability due to the abutting earthen bank in its natural state in front of the anchorages, and earth pressures on their sloping side walls. On the north side the excavation was filled in to the original ground level elevation at the front of the anchorage. On the south side the excavation was partly filled in in a similar manner.

Specifications for the masonry called for a 1:2½:4½ concrete for the piers, and a 1:3:5 mixture for the anchorages. Local gravel pits could not be found which contained enough of the coarse aggregate to make these mixtures without screening, and hence the concrete was made richer in cement, the proper quantities to use being determined by frequent tests of the gravel as the work progressed. In order to get the volumes required, one-man stones and "nigger heads" were embedded in the concrete. The drawings show the outline of the masonry work. Rock and gravel ballast filling was used under the roadway in the anchorages. This is believed to be a departure from ordinary practice for obtaining weight for anchorages in suspension bridges.

SUPERSTRUCTURE

Each of the two cables is made up of 7 bridge cable strands of 1½ in. diameter each. A test at the plant of the manufacturers, the John A. Roebling's Sons Company, gave an ultimate strength of 267,800 lb. for one strand, equivalent to a unit strength of a little more than 200,000 lb. per square inch. Their modulus of elasticity is about 20,000,000. The main span cables are cradled to a batter of about 1 5/32 in. per ft., and the plane of the land span cables is coincident with that of the corresponding main span cable. The versine of the main cables from center to center of towers is 38 ft. at the assumed "normal temperature," at which there is no stress in the stiffening trusses under dead load.

The main supporting columns of the towers are vertical and spaced 25 ft. on centers. Each column or leg is anchored to the corresponding masonry pier with two 2½-in. round steel rods embedded in the concrete to a depth of 8 ft. The tower columns are enclosed, except for manholes at the top and bottom, and are made of 3½x3½x7/16-in. angles, two 24x 7/16-in. web plates, and two 22-in. cover plates, 7/16-in. thick in the upper half and 9/16-in. thick in the lower half. Horizontal diaphragms are spaced at intervals of about 5 ft. inside the columns.

The stiffening trusses are 16 ft. on centers and are 8 ft. 3½ in. deep back to back of chord angles. In the main trusses the chord section is made up of two 6x6x⅝-in. angles, in the side spans of two 6x6x⅜-in. angles. Stresses in the stiffening trusses were figured according to Melan's formulas as modified by Professor Turneaure and published in Mr. Modjeski's report on the Manhattan Bridge, New York. Attention has been called to the fact that these formulas make no provision for unequal parameters in the curves of main and side span cables (See Engineering Record, of Feb. 5, 1910, and Engineering News, Vol. 63, No. 9, for that and other corrections by the writer). To adapt these formulas to a suspension bridge with cables of lower elastic coefficient than that of the trusses it was also necessary to introduce a factor E_o/E_t in the denominator of the term $l^2c^2IL_s/8fF$, where E_c is 20,000,000 for the cables, and E_t is 29,000,000 for the trusses. For purposes of checking computations an estimated dead load of 780 lb. per lineal foot of bridge, and a live load of 51 lb. per square foot (Class C specifications for country bridges) on a 14-ft. roadway, has been used in the formulas.

Loading and Stresses

The cables and cable fastenings, towers, anchorages, and piers were designed for a maximum uniform live load over the whole bridge at minimum temperature, which was assumed at 40 deg. Fahr. The floor was designed for a load concentration equivalent to a 15-ton road roller. In the adjustment of the cables provision has been made for equalizing positive and negative moments in the main span under the critical loadings for maximum moments. This provision puts a small negative moment under dead load only in all trusses at any temperature, and limits the maximum live and dead load unit chord stress to less than 11,000 lb. per square inch, gross section, at the critical temperatures of 110 deg. and 40 deg. even under a full uniform load per lineal unit for a portion of the bridge only, the other part having no live load on it whatsoever, a hypothetical possibility but a condition which is not within the realm of probabilities.

The limits of maximum stresses in the towers and anchorage steel, including bending stresses in the towers due to temperature changes is about the same as that of the live and dead load in the trusses. The maximum unit stress in the cables under the extreme full live load on the entire bridge at minimum temperature is less than 49,000 lb. per square inch while that of the suspenders, which are made of the same grade of steel wire, is only 17,000 lb. per square inch.

North Anchorage Foundation

Mention has been made of the fact that the estimated weight of the north anchorage is 36 tons more than that of the south anchorage. This increase in weight is due to the concrete sawteeth across the foundation bottom, and to an increase of concrete to neat lines in the excavation proper over and above that put into the excavation on the south side. On the south side the excavation was in fine gravel mixed with stones. The material from the surface down was so compact that the excavation up to about 12 ft. in depth stood vertical without caving for fully six weeks, when the excavations were filled with the concrete. With the safety factor allowed on the basis of the friction factor assumed there could be no doublt about the security of the south anchorage.

On the north side, however, the foundation was in a hard, compact clay. It was so compact that it was with difficulty a wooden stake could be driven into it. The excavating was done by taking thin shavings with the shovel. This shaved surface became very smooth during a rainstorm, and we decided to reinforce our book knowledge on friction factors for clay by making some simple experiments. We took a small rectangular piece of limestone, with laminated surface downwards, and added just enough weight to make a total of 20 lb. The tangential pull up the foundation slope of 8.2 per cent. to overcome the friction due to this weight of 20 lb. was as follows:

On the original surface as left by the excavators, 14 to 15 lb.

On a newly smoothed surface, 12 lb.

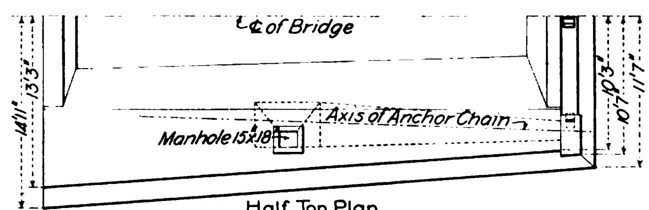
On a newly smoothed surface just flushed with water, 8½ to 9 lb.

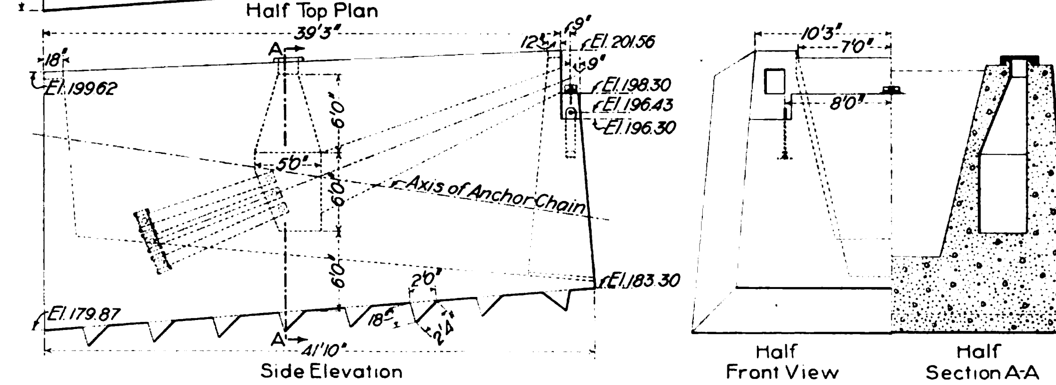
On a newly smoothed surface flushed with water and allowed to stand a few minutes drying in the sun and then wetted by pouring water over and around the stone, 13½ to 14 lb.

full dead load had come on the bridge, but absolutely no yielding could be detected.

Miscellaneous

Mr. Holton D. Robinson of New York City was the engineer for the Town Board and, as such, prepared plans for the bridge. About July 1, 1910, he resigned his position of engineer and the writer was appointed to fill the vacancy and thus acted as resident engineer for the Town Board during the construction. Mr. Robinson entered the field as bidder for the construction of the bridge on his own plans, and signed the contract on July 16, 1910. Although he formally sublet the masonry to a local contractor, Mr. L. N. Gross as superintendent and his representative was in constant and continuous charge of the construction. Earth was broken on the excavation on July 23; the masonry work was completed on Oct. 13; the first person walked across the bridge two weeks later, and the first rig drove across on Dec. 20. The bridge, with the exception of some painting, was completed on Jan. 9, 1911. In this connection it might not be amiss to state that the weather was generally very severe after Nov. 28th. Local residents informed us it was the severest continuous cold weather they had had in





North Anchorage of Suspension Bridge

The results are all consistent and the last one significant. To overcome the tangential component of the force of gravity required a pull of 1.6 lb. Deducting this and taking averages the resulting friction factors vary from 36 per cent. for the newly flushed surface to 65 per cent. for the original surface as left by the excavators, while that for the recently wetted surface after standing a few minutes and wetted again is 61 per cent. This is nearly a fourth better than the friction factor assumed, and with a safety factor of more than 2 there really was no question about the security of the foundation.

To make assurance doubly sure, however, it was decided to cut the sawteeth as indicated on the drawings. The function of an uncertain friction factor of a large concrete mass on a hard clay does not now enter into consideration. The direction of the extreme resultant pressure is actually down the slopes of the sawtooth surfaces. The proper factor to assume, then, is one of hard clay on hard clay in its original bed, and this is constantly reinforced by the factor of cohesion.

By the wedging effect of the anchorage itself and its grip on the foundation base it is evident that the resistance of hundreds of tons of earth is co-existent with that of the anchorage itself. To add further reinforcement to the argument for the security of the anchorages we made a very careful survey with a triangulation transit to determine if there could have been any possible movement of either anchorage by the time the

20 years or more, and the thermometer frequently remained below zero all day with a stiff wind blowing at that. In fact some of the steel work was done at 20 deg. Fahr. and below.

The cement was furnished by the Alsen's American Portland Cement Works Company, the structural steel by The Eastern Steel Company, the cable material, suspenders and fastenings, by the John A. Roebling's Sons Company; and the saddle castings and suspender rod U-Bolts by The A. and F. Brown Company. Stowell & Cunningham, Albany, N. Y., inspected both shop and mill work on the structural steel.

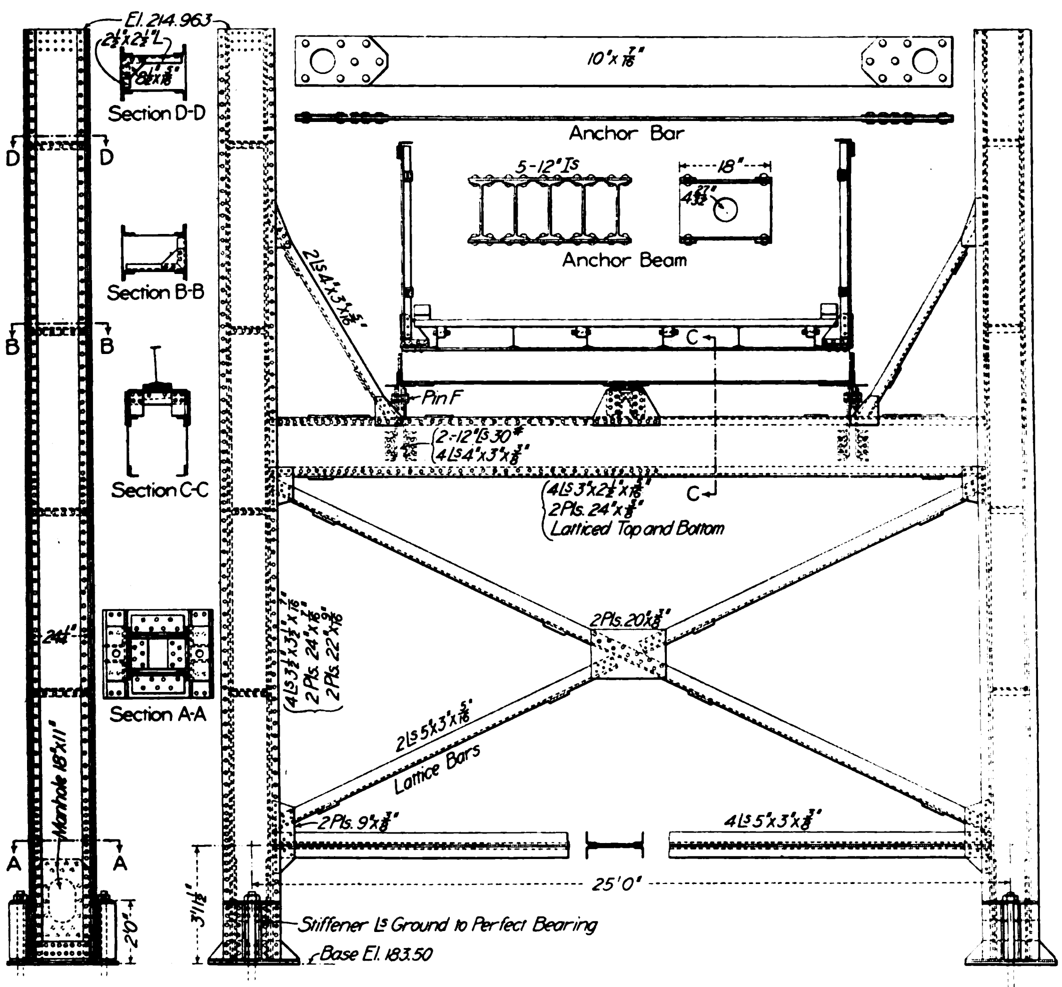
The cost of the bridge to the Town of Massena with railing, fence, painting, approaches, and engineering, complete, was $41,990. This is fully $18,000 less than the lowest estimate received by the Town Board on other types of bridges affording the same clearance and carrying the same live loads, as specified under Class C for country bridges.

THE FORT LARAMIE UNIT of the North Platte irrigation project, Nebraska-Wyoming, has received from the Secretary of the Interior the authorization necessary for construction by the U. S. Reclamation Service. No construction is to be begun, however, until at least 95 per cent of the private holdings have been properly pledged for the repayment of their proportional cost of the project. The Fort Laramie unit will cover approximately 170,000 acres.

Superstructure and Erection of the Massena Center Bridge

Suspension Bridge with 400-Foot Central Span Erected Without Steam Power

The La Grasse River highway suspension bridge at Massena Center, N. Y., with a 14-ft. roadway proportioned for ordinary traffic and for a 15-ton road roller, is 680 ft. long, exclusive of approaches, and has a suspended channel span of 400 ft., as described in the Engineering Record of Oct. 5, 1912. It is notable in that the total cost to the owners is within the engineer's preliminary estimate of $42,000, and more than 30 per cent less than estimates for other types of structures with the same clearance and live load capacity. The bridge is reported to be very rigid and without noticeable deflection or vibration under ordinary loads.



Lower Part of Tower Bent

Towers

The towers are simple transverse bents about 65 ft. high, each made with two vertical posts connected by horizontal transverse struts and X-bracing and by a main transverse girder to which the stiffening trusses are connected. Each post is designed for a maximum stress, including bending, of about 11,000 lb. per square inch. To carry the loads there is provided a cross-sectional area of 57.2 sq. in. with a 22 x 24-in. closed rectangular section made up of two built channels and two cover plates. They are field-spliced near the middle, are stiffened by interior horizontal diaphragms spaced about 5 ft. apart, and have 36 x 44-in. base plates with rust joint bearings on the concrete piers, to which they are anchored by pairs of vertical bolts nearly 11 ft. long. They engage 12 x 12-in. reaction plates embedded 8 ft. in the concrete, and are proportioned to resist a total maximum tension of 13,000 lb. per square inch. The upper ends of the columns have riveted horizontal cap plates to which there are bolted tough gray iron saddles affording fixed bearings for the cables. The interiors of the posts are accessible through manholes at the top and bottom, but are not provided with permanent ladders, the horizontal diaphragms affording a means of ascending and descending. Pairs of gusset plates are shop-riveted to the inner flanges, forming jaws to which the transverse struts and X-bracing are field-riveted.

Cables

The galvanized seven-strand cables 4½ in. in diameter are neither wound nor jacketed, but the exposed strands were, after erection, thoroughly painted with two heavy coats of Atlantic white lead and boiled linseed oil, and clamped together at intervals of about 11 ft. by the heavy cast-steel bands which afford connection for the suspender ropes. At the ends of the main span they have fixed supports in cast-iron saddles bolted to the tops of the main tier posts. The saddles have deep grooves molded to fit the surfaces of the strands and inclined to correspond with the cradling and with the inclination of the main and shore span tangents. The top of each saddle is covered with a steel plate bolted on tight over the cable.

To prevent possible movement under the unbalanced stresses at the top of the tower post each cable is secured by a heavy cast-steel stop clamped tightly to it on the river side and bearing against the saddle casting. Each end of each cable strand is socketed and receives a short 2¼-in. diameter U-bolt. The U-bolts engage pins through reinforced plate anchorage bars which at their opposite ends engage pins through reaction girders. These girders consist of horizontal sets of I-beams with top and bottom flange plates which are embedded in the pier masonry normal to the cable stress. Manholes and inclined tunnels in the anchorage piers give access to the upper ends of the anchor bars and to the ends of the cable strands within the anchorage masonry. The strands diverge from a heavy collar clamped to the cable just beyond the river face of the anchorage.

Trusses

The stiffening trusses about 8¼ ft. deep and 16 ft. apart on centers have their chords uniformly curved from anchorage to anchorage to a camber height of 7 ft. 4 in. at the center of the river span, the radius of camber being 6081 ft. They are divided by light vertical members into panels of about 5½ ft., each of which has a single light diagonal made with a pair of angles riveted back to back. The chords are proportioned for maximum stresses of 18,000 lb. per square inch net section and have a net cross-sectional area of 12.0 sq. in. made up of two 6 x 6 x ⅝-in. angles in the river span and in the side spans an area of 8.0 sq. in. made up of two 6 x 6 x ⅜-in. angles riveted back to back. The trusses were shipped without any assembling, chord sections being three panels long and weighing about 1850 lb.

The center span trusses terminate at the river piers and their lower chords are pin-connected to short vertical rocker posts engaging the transverse tower girders and allowing for an estimated maximum longitudinal movement of about 2½ in. at each end of the truss. The ends of the top chords are bolted through slotted holes to the top chords of the end spans.

The bottom chords of the end spans have offset connections to the top pins of the rocker posts. At the shore ends they are pin-connected to similar vertical rocker posts 19½ in. long on centers made of pairs of sheared plates riveted together with separators and engaging at their lower ends vertical anchor plates, with wide flange angles at their lower ends, built 3 ft. into the masonry.

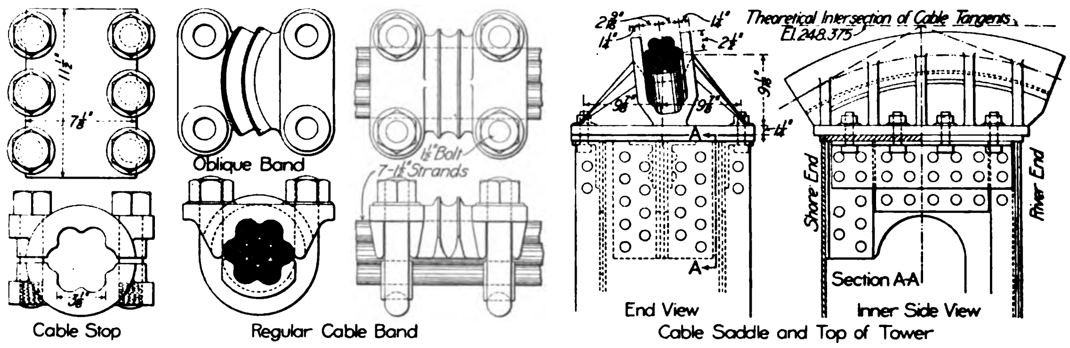
The tower posts at each pier have one panel of X-bracing at the top and one panel at the bottom, and the intermediate panel through which the traffic passes is braced by top and bottom kneebraces. The lower kneebraces are pairs of angles riveted to the backs of two built channels 24 in. deep and 24 in. apart, which form the main transverse girders connected to the stiffening trusses. The webs of these channels are connected 8 ft. each side of the center line by a pair of vertical diaphragms parallel to the axis of the bridge. These diaphragms are made of pairs of 12-in. channels with their webs bored to receive the pins in the lower ends of the rocker posts.

Lateral Bracing

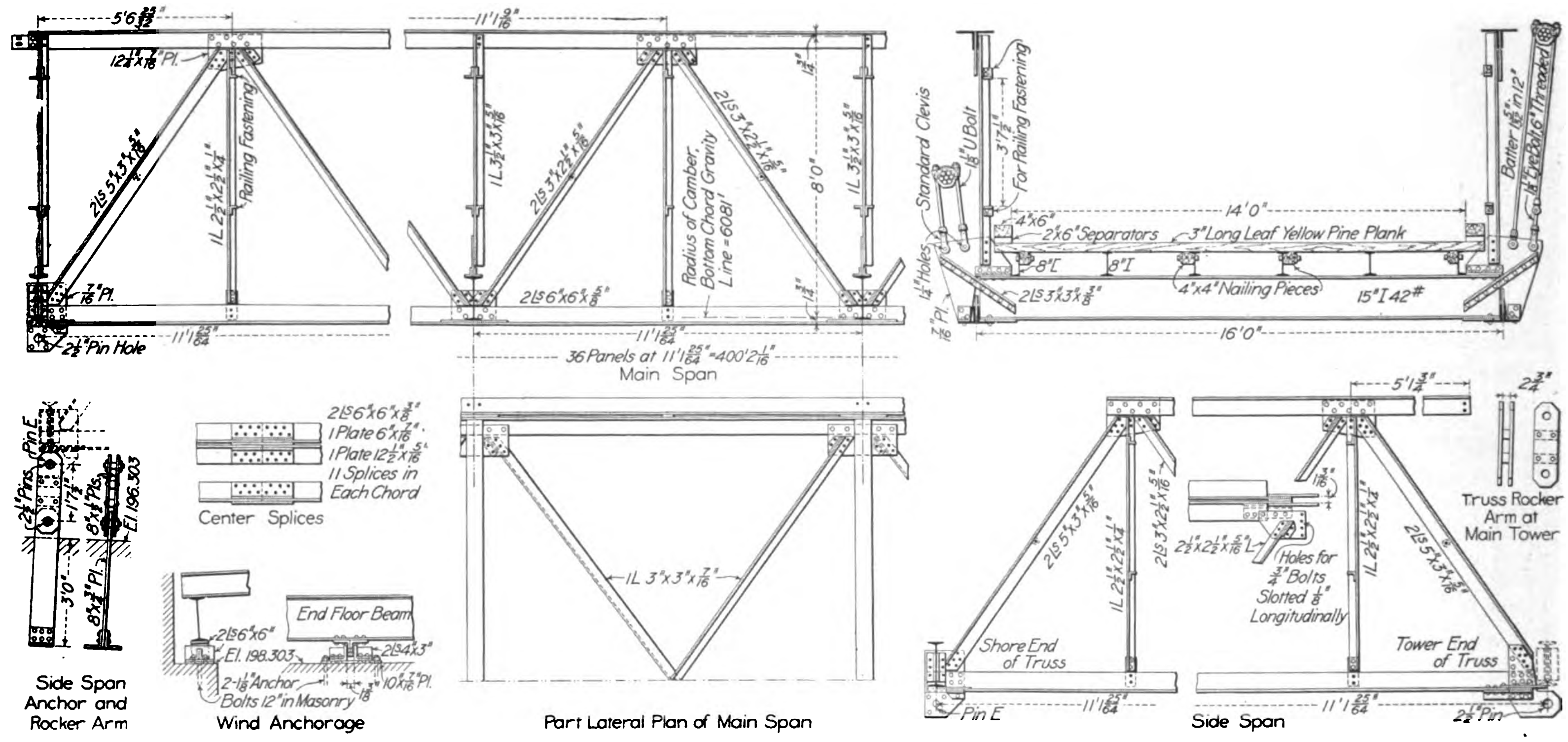
The bottom chords of the stiffening trusses are X-braced by 2½ x 2½-in. and 3 x 3-in. angles, field-riveted through both flanges to horizontal connection plates shop-riveted to the bottom flanges of the lower chords and of the floorbeams. The lateral stresses in the end spans are taken to the center points of the end floorbeams, whence they are transmitted to the anchor pier masonry through sliding joints which permit longitudinal movement, but prevent transverse displacement and receive all horizontal shear. The connection is made by a pair of angles parallel to the axis of the bridge, riveted through the center line to the bottom flange of the floorbeam so that their projecting vertical flanges are engaged between the vertical flanges of reinforced parallel angles riveted to a base plate which is anchor-bolted to the pier masonry. At the tower piers there are similar sliding connections between the floorbeams and extensions of the main transverse girders.

Floor System

The floorbeams, about 11 ft. apart, are single 15-in. I-beams nearly 15 ft. long with their square ends cut to clear the webs of the bottom chords of the stiffening trusses. The floorbeams are extended about 15 in. beyond the centers of the trusses by notched vertical plates which clear the lower chords and are riveted to the truss verticals, to the top flanges of the floorbeams and to the bottom flanges of the bottom chords. They are also connected by diagonal pairs of reinforcement angles to the floorbeam webs. These plates are bored to receive the clevises at the lower ends of the suspenders from the main cables.



Details of Cable Fittings



Stiffening Trusses and Floor System

The trusses are supported through the floorbeams by pairs of inclined suspenders at each main panel point. Where the distance between the cables and the floorbeams is very short steel rods are used for suspenders. At all other points the suspenders consist of two parts of high strength steel rope strands 5/8 in. in diameter with their bights engaging the cable bands and their socketed lower ends pin-connected to the floorbeam plates. They have an ultimate strength of 42,000 lb. and a maximum working stress of 4000 lb. for each part.

The floor is laid with 3-in. transverse planks spiked to nailing pieces bolted to the webs of alternate longitudinal I-beam stringers. The ends of the planks are bolted to guard timbers and the roadway which serves both for vehicles and pedestrians is guarded on both sides by a fence of wire netting 3½ ft. deep with 4-in. diamond-shape mesh. The netting, received in 330-ft. rolls, is attached directly at intervals of about 3 ft. to the top and bottom 2 x 2-in. horizontal railing angles. It was put under considerable longitudinal tension developed by an ordinary fence stretcher, and was transversely stretched by hand levers and securely wired so as to be very tight and rigid.

Anchorages

The first portion of the excavation for the anchorages was done with horse scrapers, drawing the earth out of the pit in the usual manner over a ramp. When the depth became too great for this the remainder of the excavation was made, as was that for the main piers, by hand, no pumping being required in any case except to remove a small quantity of surface water. The ground was so good that the sides stood vertical without sheeting. Part of the concrete was deposited against the face of the excavation and part of it in forms of simple construction. All of the concrete was made with Alsen's Portland cement and local sand and gravel in varying proportions, as fixed by frequent screening tests to determine percentage of sand. About 1400 yd. in all were required for the substructure. Sand and gravel delivered in wagons were stored in heaps and carried in wheelbarrows to a continuous Eureka trough mixer and to a Nims double cube mixer. The concrete was taken from the mixers in wheelbarrows and deposited through chutes.

Erection of Towers

The lower sections of the tower posts, weighing about 4 tons each, were erected by a gin pole set on the ground alongside the main pier. As soon as they were set in position the anchor bolts were screwed up, giving them abundant stability, and the transverse girders and lower X-bracing were assembled to them, completing the lower halves of the tower bents. The gin pole was then raised up to a position on one end of the transverse girder and there erected the upper section of the adjacent post, after which it was moved to the other end of the same girder and erected the other post. The other tower was erected in the same way except that the lower sections of the posts were handled by a stiff-leg derrick located on the surface of the high ground adjacent to the pier.

Erection of Cables

Each cable strand, about 678 ft. long, weighed 3300 lb. and was shipped, without a drum, in a coil 4 ft. in diameter inside. The coils were shipped in box cars and some difficulty was anticipated in unloading them until a locomotive crane was secured which was run alongside the track and removed them from the car without difficulty.

A wooden drum with one head was inserted in each coil and the other head fastened to it, making a reel which was placed in a frame and from which the strand was unwound and laid on the surface of the ground parallel to the river. One end of each strand was permanently fastened to the bars in the anchorage pier and the other end, after passing through a snatch block fastened to the tower column, was taken across the river on a flatboat towed by a horse on the opposite bank. When the boat approached the shore a steel rope was connected to the end of the strand and the latter was dragged to shore by the horse. A bight of the strand was then lifted up and put in a snatch block attached to the base of the tower post and the free end was made fast to a double set of falls and pulled by the horse to the anchorage and there made fast to its permanent connections. The bight of the strand was then taken out of the snatch block and hauled up to the top of the tower by a four-part tackle operated by the horse or by a horse windlass manned by the laborers.

As the strands were erected they were accurately adjusted by levels taken at the center of the span, allowance being made for about 1 in. deflection of the tops of the towers toward the center of the river. The strands were erected at the rate of seven in two days, and when all were in position they were immediately clamped together by the cable bands which, together with the suspenders, were set in position from a traveling platform.

The platform had a light 6 x 6-in. floor suspended at either end from two two-wheel carriages 6 ft. long running on the cables and maintained in position on them by the double vertical suspension pieces, one on each side of the cable, which were attached to the axles of the wheels. The platform was lowered from the top of the tower to the center of the span and then drawn up to the top of the opposite tower, making a continuous passage from tower to tower instead of being removed at the center and lowered from the opposite tower as at first contemplated.

The main span cables were fixed at the towers by clamping the stops on them next to the saddles, and then the land span cables were adjusted by screwing up the anchorage adjustments until the tops of the towers were deflected about 0.3 ft. toward the anchorages. The cable bands were then set for the land spans, the floorbeams and stringers of which were erected on light falsework.

Erection of Suspended Structure

The steel work was delivered on the bridge by a two-wheel lever cart. In the erection of the main span floor two of the three-panel stringers, being anchored and cantilevered one panel in advance of the last floorbeam set, provided tracks on which the floorbeams were successively skidded out to a position where they were secured to the suspenders and launched into space, after which the permanent stringer connections were made, a plank track laid for the material cart, the next panel of floor erected, and so on. All of the floorbeams and stringers for the 400-ft. span were thus erected in about 3 days.

The bottom chords of the stiffening trusses and the lateral system were assembled in position and the field riveting in their splices and connections was accomplished while waiting for the top chords and web members of the trusses. When the latter were received they were erected by a four-part hand tackle suspended from a light dutchman pushed over the floor as required, and the truss diagonals were put in position and the riveting and painting finished, completing the bridge.

No expert bridge erectors were employed on the work, which was done by a force of 20 local laborers, most of them St. Regis Indians. Excavation was commenced July 23, 1910. The first cable strand was anchored Oct. 29, and the work, with the exception of some painting, was completed Jan. 9, 1911. All field rivets were hand-driven and all steel erection was done by manual labor and a single horse. The structural steel, cables and steel castings, weighing altogether about 220 tons, were furnished by the Eastern Steel Company, the John A. Roebling Sons Company and the A. & F. Brown Company.

The bridge was designed by Mr. Holton D. Robinson, Assoc. M. Am. Soc. C. E., New York, and was erected under the direct supervision of Mr. John Berg, resident engineer, and Mr. L. N. Gross, superintendent.

Construction of Surfaces with Bituminous Materials

A résumé of typical current practice and a review of some of the causes of failure of the different types of bituminous surfaces and bituminous pavements was presented at the American Road Congress in Atlantic City, N. J., by Prof. Arthur H. Blanchard, of Columbia University. An abstract of his paper is given below:

Bituminous surfaces, as defined by Professor Blanchard, are those consisting of superficial coats of bituminous materials with or without the addition of stone or slag chips, gravel, sand or materials of a similar character. Bituminous macadam pavements are those consisting of broken stone and bituminous materials incorporated together by penetration methods. Bituminous gravel pavements are those consisting of gravel and bituminous materials incorporated together by penetration methods. Bituminous concrete pavements are those having a wearing surface composed of stone, gravel, sand, etc., or combinations thereof, and bituminous materials incorporated together by mixing methods.

The failure of bituminous surfaces is many times due to failure to place the surface in satisfactory condition before the application of the bituminous material. Bituminous materials often are applied over a surface in which are pot holes and ruts, or which is dirty. With certain kinds of materials a damp condition of the surface has resulted in failure. Failure is due also to materials which do not possess the proper characteristics for the conditions under which they are employed. An example is a prominent thoroughfare in one of our large cities which is subjected to motor bus traffic and a large amount of motor car and horse-drawn vehicle traffic. This road is constructed of gravel upon which has been applied an asphaltic oil and gravel top dressing. The surface is full of ruts caused by the traffic pushing the material from side to side. Again the large percentage of volatile constituents contained in certain asphaltic oils has rendered surfaces constructed with them unsatisfactory because of the long period required for these surfaces to "set up." In certain cases the use of light oils on tar or asphalt surfaces has softened the original bituminous surface.

From the standpoint of construction failures are due both to the use of too little and too much bituminous material. Uneven distribution is accountable for many failures, while in other cases a lack of sufficient covering of stone chips has rendered the surface sticky, mushy, sometimes in the first season but sometimes not until the second summer. A mat type of construction has proved inefficacious where horse-drawn vehicle traffic has been more than a certain amount in combination with motor-car traffic.

The efficacy of bituminous macadam or gravel pavements depends upon the combinations of sizes of broken stone or gravel and the combinations of bituminous materials used when two applications are employed. Many engineers do not appreciate the fact that different types of bituminous materials have entirely different physical properties and require entirely different treatment in use. Overheating of the material has likewise proved the cause of many failures.

Under the heading construction, we find failures due to the uneven distribution from the improper use of hand-pouring pots or distributors of other types. Many unsatisfactory bituminous macadam pavements result from the use of the wrong sizes of broken stone. One instance will be cited where a hard broken stone ranging from 2 to 3½ in. was used for the wearing surface. After rolling, 1½ gal. of bituminous material were applied and the road finished with a layer of chips. The rapid formation of fine cracks due to the rocking movement of the individual stones under traffic, finally resulting in raveling and general disintegration, is of common occurrence. Segregation of sizes of stone preventing uniform penetration results in weak spots in some cases and "fat" spots in others. In certain cases after a rain the construction has been carried on before the broken stone immediately below the surface has dried out.

Bituminous concrete pavement is constructed usually by one of three methods. Type A consists of so-called "one-size" crusher run stone mixed with bituminous material. This term, as used here, refers to the product which passes through a screen of one size of holes and is retained upon a screen having smaller holes. The product thus obtained does not usually consist of stone of uniform size. It is evident that variation in size produces a more stable pavement than if the aggregate consisted of broken stone of uniform size.

Type B consists of one size crusher-run stone and sand mixed with bituminous material. The wearing surface is sometimes finished by rolling in fine stone chips but generally a seal coat is used together with fine mineral matter for a top dressing. When constructed on a commercial scale, the mineral aggregate is always heated and mixed in a specially constructed machine. Usually the same grade and type of bituminous material is used for the mix and the seal coat.

Type C consists of a wearing course of graded broken stone and sand with or without other mineral matter. The aggregate is mixed after being heated with a bituminous cement in a specially designed machine. As with type B this pavement is finished with and without seal coats of bituminous material. The Topeka and the Bitulithic pavements may be cited as examples of type C. The percentage of failures of bituminous concrete pavements is much smaller than in the case of bituminous macadam and gravel pavements.

Experiments in bituminous concrete road construction in Rhode Island and in the borough of Queens, New York City, seem to have demonstrated that high carbon tar of a certain consistency is not as satisfactory for a seal coat as some types of asphalts, when the percentage and volume of horse-drawn vehicle traffic is large. In some cases an apparent cause of failure has been an excess of flux or of the volatile constituents in asphalt cements. Pavements constructed with such materials many times are wavy, due to the movement of the surface under heavy traffic. Many cases are reported where materials have been overheated at the construction site.

Cases have occurred where failure, even under very light traffic, was due to using large uniform size broken stone for the mineral aggregate of the mix. Poor combinations of sizes of broken stone and sand have resulted in segregation during mixing, transportation or spreading, resulting in a pavement of varying density and stability. Overheating of the mineral aggregate has caused burning of the bituminous material in some instances or the formation of a thin film of bituminous material over the broken stone which is not of sufficient amount to bind the adjacent stones together. Many failures are due in both the case of bituminous macadam and bituminous concrete pavements to poor foundations.