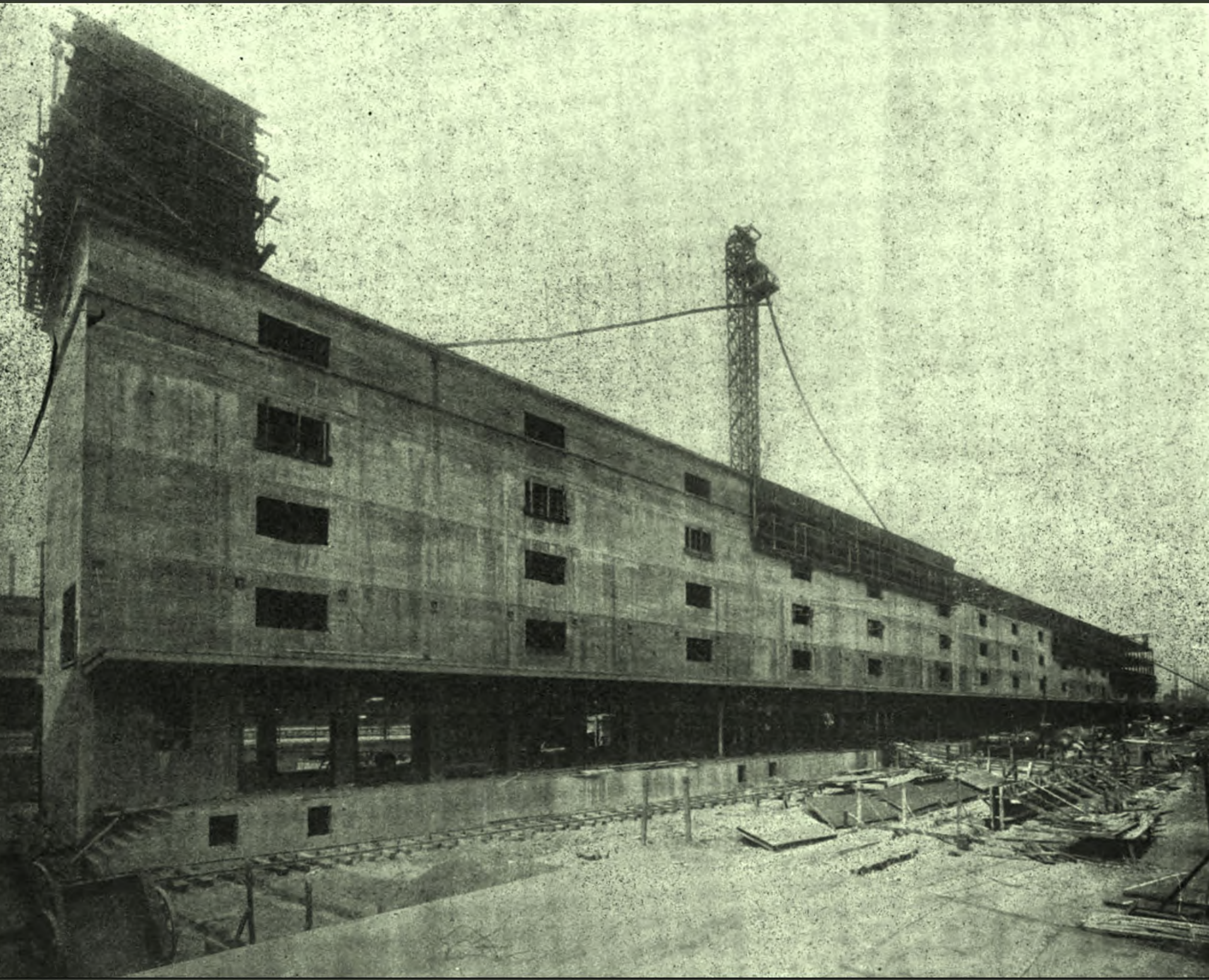


Railway age gazette

FIFTY-FIFTH QUARTO VOLUME

From July 1, 1913, to December 31, 1913

Railway Age Gazette

(Established in April, 1856)

FIFTY-EIGHTH YEAR

NEW YORK CHICAGO CLEVELAND LONDON

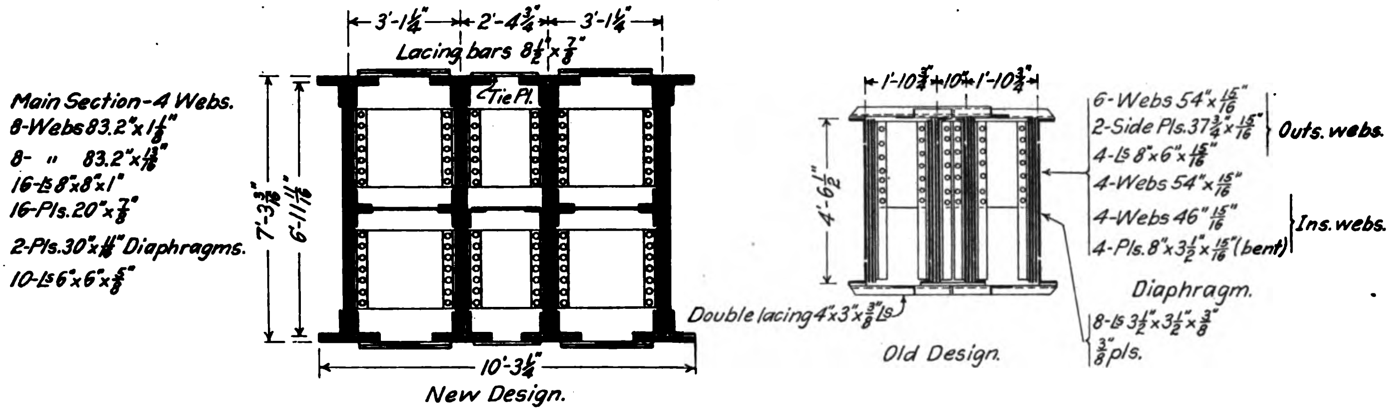
1913
SECOND HALF

ADOPTED DESIGN OF THE QUEBEC BRIDGE.*

Discussion of Elements Considered in Designing the Longest Span in the World by Member of the Board of Engineers.

By Ralph Modjeski,
Consulting Engineer.

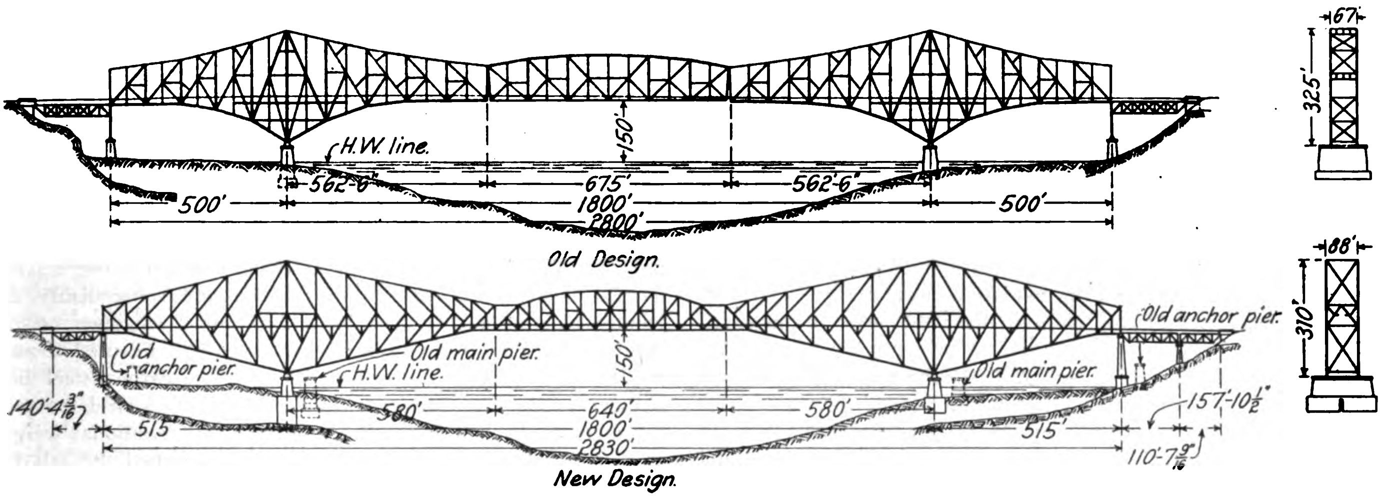
The clear height of the Quebec bridge above high water was fixed by the navigation interests at 150 ft., and the length of span 1,800 ft., is entirely due to the physical conditions of the crossing. The stream at this point is narrow and deep, the depth in the center being about 190 ft. The current velocity at ebb tide is very high—about nine miles per hour. Very heavy ice runs at times and tends to gorge. The bed rock, as shown by the borings, while accessible near the shore lines, dips rapidly towards the center of the stream. All these conditions made it imperative to build a span of great length. The information as to bed rock which we now have would indicate that the original project could have been designed with a somewhat shorter span, yet we should remember that this original project was undertaken by a private corporation, and we should perhaps recognize the value to it of such advertisement as the building of the longest span in the world would obviously afford. The next longest span is that of the Firth of Forth bridge, 1,700 ft. It is doubtful if a shorter span than 1,700 ft. would have been practicable at the location adopted for the Quebec bridge. I consider it perfectly legitimate to build a more expensive structure than economy of the work itself would call for, if the more expensive structure will afford sufficient advertisement and publicity to compensate for the additional expenditure.



Lower Chords of Old and New Quebec Bridge Designs.

A project to build a large bridge at Quebec, presumably in the same location as the present one, was seriously considered in 1884 and 1885. Messrs. James Brownlee, A. Luders Light and T. Claxton Fidler designed a structure with a clear span of 1,442 ft. The description of that project mentions rock foundations. The more complete information we now have, which was obtained by a costly series of borings, shows that at the present location rock could not have been attained in both piers with any known method of foundation if the piers had been spaced only 1,442 ft. apart, even if the great depth of water could have been overcome.

After the disaster of August 29, 1907, the Dominion government took up the reconstruction of this bridge. A board of three engineers, including myself, was appointed to design and construct the bridge. After some study of the situation, the board decided that the new bridge should be made wider between trusses and designed to carry heavier loads than those originally contemplated; that, further, none of the old steel work could be used to advantage. It also decided to keep the same location. The final outcome is a double-truck span of 1,800 ft., with a width of 88 ft. between centers of trusses. The old piers were not large enough for the



Old and New Quebec Bridge Designs.

*Abstracted from a paper presented at the meeting of the Mechanical and Engineering Section of the Franklin Institute. Copyrighted by the Franklin Institute.

new design and could not, therefore, be used. The two main piers will be designated as north pier and south pier respectively. At first the board contemplated building an entirely new pier 57 ft. south of the present north pier and enlarging the foundation of the south pier by sinking additional caissons adjacent to the old caisson. The necessary span length would then have been 1,758 ft., and it was on that length of span that tenders were asked. It developed later, from the experience of sinking the north caisson, that the method proposed for enlarging the south foundation would not be safe, even if it were practicable, and so an entirely new foundation and pier were decided on for the south shore. The new north pier could not be placed farther out in the river because of the sloping bed rock and great depth of water. The south pier could not be placed on the north, or river, side of the old south pier, because of the old wreckage, so it was placed 64 ft. 8 in. south of the old pier, or as close as possible to it. Both new piers being placed 64 ft. 8 in. south of the old piers, measured between centers, the new span remains 1,800 ft. long.

With this span length and with the materials now at the disposal of the engineer, the practical limit of cantilever construction has very nearly been reached. In fact, if economy alone is to be considered, a cable suspension bridge would have been cheaper for a span of 1,800 ft. The cantilever structure presents a greater rigidity under moving load, and this greater rigidity was the determining factory in the decision of the board to adhere to the cantilever type. Tentative plans of the suspension type with wire cables were, however, partly worked out by the board in the way of study. The comparative rigidity of the cantilever system and the suspension type may be gaged by the deflections at the center of the span under full load.

New Quebec span, total live load	11¾ in.
A cable suspension bridge, trial design—live load only, over	2 ft.
A cable suspension bridge—with 120 deg. variation in temperature and full live load—between highest and lowest position about	7 ft.

The new bridge was finally designed with two anchor arms 515 ft. long, a suspended span 640 ft. long, and two cantilever arms 580 ft. long. The moving loads adopted are two Cooper's Class E-60 engines on each track followed or preceded, or followed and preceded, by a train load of 5,000 lbs. per foot per track. In addition to the actual dead load of the structure, a load of 500 lbs. per lineal foot on the suspended span and 800 lbs. on the balance of the bridge was allowed for snow. The wind loads were taken as follows: A wind load normal to the bridge of 30 lbs. per sq. ft. of the exposed surface of two trusses and 1½ times the elevation of the floor (fixed load), also 30 lbs. per sq. ft. on travelers and falsework during erection; a wind load on the exposed surface of the train of 300 lbs. per lineal foot applied 9 ft. above the base of rail (moving load); a wind load parallel with the bridge of 30 lbs. per sq. ft. acting on one-half the area assumed for normal wind pressure. The assumed wind pressure is equivalent to about 35 per cent. of the uniform live load near the piers and to about 20 per cent. of the live load near the ends of the cantilever arms.

A pressure of 30 lbs., according to German experiments with electric cars, would correspond to a wind of a velocity of over 100 miles per hour. Other experiments made at various times on small surfaces show that a velocity of 85 miles would correspond to a pressure of about 30 lbs.

With a wind of this velocity there would be no traffic on the bridge—empty freight cars or even light passenger cars would be overturned. Velocities of over 85 miles may occur in cyclones and tornadoes over restricted areas. Such storms are very rare in Canada; but even should such an extraordinary disturbance happen, causing a wind pressure of as much as 60 lbs. to be applied to the entire bridge the stresses in the truss members would be less than with the maximum live load and a 30 lb. wind, and although the stresses in the laterals would be increased above the specification limits, they would still remain within the elastic limit of the members.

Where there are no other considerations beyond the actual working stresses in the finished structure, the most economical length of the suspended span for a total span of 1,800 ft. would be in the neighborhood of 1,000 ft. But to erect a simple span of such unprecedented length, either by floating or by the cantilever method, would be impractical. Furthermore, the cantilever method of erecting a suspended span of even a moderate length always requires additional material, both in the cantilever arms and in the suspended span, to take care of the erection stresses. The longer the suspended span in relation to the total main span, the greater will be the required addition—so that whether it be contemplated to erect the suspended span by the cantilever method or by floating into position, the length of the suspended span finds itself limited not by mere economic considerations of the finished bridge, but by either the excess of material required during erection by the cantilever method, and difficulties arising therefrom, or by the difficulties attending the floating of a very long and heavy span into position. These difficulties increase very rapidly with the length of the span to be floated. In the new design the suspended span is the longest which the board considered safe to float, and it fits the entire design very well. The erection of this span by floating made it possible to design it with the view to greatest economy. Its various members will not be subjected to any greater stresses during erection than they would be in a simple span of the same length resting on two piers. It was, therefore, possible to design it as economically as to weight as a well designed simple span would be. It is more important to save weight in a suspended span than in an independent simple span, because each pound in the former requires several pounds in the entire structure to carry it. One pound uniformly distributed over the trusses of the suspended span needs 3 lbs. of metal added to the bridge to carry it, making an addition of 4 lbs. in all. This accounts for curved top chords in the span in question, as well as for the use of nickel steel for the trusses.

It has been pointed out that the length of the anchor arms is uneconomical—that a shorter arm would have been cheaper. It must not be forgotten that a shorter anchor arm increases the pier reactions, as well as the steel in the anchorage proper. The present anchor piers are founded on rock ledges which dip rapidly toward the river. To move them nearer to the river would have involved much more expensive foundations.

It may be remarked here that, while an addition of dead load in the main span will require several times the weight of metal to carry it, an addition of dead load in the anchor arm requires no increase of metal to carry it when there is an upward or negative reaction on the anchor pier. This is explained by the fact that any load placed between the main piers or on the main spans increases all moments and shears over all the spans, while any load placed on the anchor arm, if the reaction on the anchor pier is negative, decreases that reaction and consequently the moments in the anchor arm, but has no effect whatever on the main span. For this reason carbon steel will be used mostly in the anchor arms of the new design. The carbon steel unit stresses adopted are generally 5/7 of the nickel steel stresses, the former requiring heavier members. This additional weight in the anchor arms is a source of economy when the relative prices of carbon and nickel steel are considered.

An opinion has been expressed that the height over the piers is not great enough for economy. Actual calculations show that for economy the height of 310 ft. is too great by about 20 ft. for the "K" system of trussing adopted; further, that this height would have been at least 40 ft. too great for the original system of the official design. The height of the Forth bridge towers, while 26 ft. greater than the Quebec bridge, though the span is 100 ft. shorter, is no doubt economical for the form of trussing adopted. The economical height is not only a function of the length of the span, but also of the panel length next to the pier. This height should be such as to correspond to an inclination of the diagonals not far from 45 deg. A double intersection system with very long panels near the pier, such as adopted in the Forth bridge would have been economical for the Quebec bridge, except that it requires a system of secondary members or sub-posts, or very heavy longitudinal girders, or both, to carry the load from panel to panel. Then, too, it is well to reduce in the members the stresses due to their own weight—which in long panels become quite important. The 20-ft. excess in height of the present Quebec design over what would have been the economical height is justified by the resulting reduction in the sections of the bottom chords, which are of considerable size at best.

In long cantilever spans the bottom chords of the cantilever and anchor arms should be straight when possible. With a curved chord the joints must be made at the panel points. These joints are of great importance, as has been shown in the report of the Royal Commission on the Quebec bridge disaster. They should be fully spliced to take care of secondary stresses due to deflections of the span during erection and under the action of live load. It is advisable, therefore, to place them outside of the point of connection with the diagonals and keep them clear of gusset plates. The same objection does not exist in top chords of simple spans, which are of moderate sizes, even in the longest spans known. The economy in simple spans resulting from such curved chords is worth while and quite important, while if any economy were to result from curving the bottom chord of the cantilever and anchor spans, such economy would certainly be of little importance in comparison with the resulting disadvantages. The vertical deflections from live loads are not as great in a straight chord design as in a curved chord design. Another consideration in favor of the straight chords is that the most important, in fact the bulk, of the wind forces travel to the pier through the bottom chords of the cantilever and anchor arms and the wind bracing or lateral system situated in their plane. The straight bottom chords carry these stresses directly to the piers without transmitting any appreciable components to the web system of the trusses. Not so with curved bottom chords. At each joint where the chord's direction is changed a component stress is transmitted to the web. This means that while a pair of straight chords with its lateral system deflects under the action of the wind in the plane of the chords only, a pair of curved chords, by transmitting shear to the web members, causes the trusses to deflect, the windward truss downward, tending to flatten the curve, and the leeward truss upward, tending to make the curve more pronounced. The rigidity of the straight chord design against lateral deflections and oscillations is therefore greater than that of the curved chord design.

One of the reasons why curved bottom chords were used in the cantilever arms of the original Quebec bridge design was the fact that it was the aim of that design to provide full headroom of 150 ft. on a width of 1,000 ft. The bottom chords of the anchor arms were then made curved also for the sake of symmetry. This width on which the full headroom will be obtained has been reduced in the new design to about 760 ft., which certainly is more than ample to accommodate navigation. Only the highest vessels will be limited to this width of 760 ft., and that only at high water.

The top chord of the Quebec bridge cantilever and anchor arms is straight. The Forth bridge cantilever arms have straight top chords also. While there was good reason for making the Forth bridge top chord straight, there was no serious reason, beyond a slight increase in vertical rigidity, for making it straight at Quebec. The two trusses on the Forth bridge are in planes inclined toward each other at the top. The two top chords are parallel. Had they been made curved they could not have been parallel, since they must necessarily be situated in the inclined planes of the trusses. The appearance of tension chords having a greater distance apart at the center of the arm than at either end would have been very bad. But there is no such reason at Quebec. The trusses are in vertical planes and the top chords could have been curved without serious inconvenience, but also without any advantage. The board considered that, aside from the additional vertical stiffness, a straight chord will present an appearance of strength which a curved chord would not.

With regard to the distance between trusses and their position relative to each other, the trusses of the new Quebec bridge will be in two vertical and parallel planes. The distance, center to center of trusses will be 88 ft. One of the first preliminary sketches made after the board was created contemplated placing the trusses in planes inclined in the same manner as in the Forth bridge, namely, with the tower posts converging toward the top and the bottom chords of both the anchor and the cantilever arms converging toward their respective ends. Another sketch contemplated trusses in vertical planes, but converging for the anchor and cantilever arms toward their respective ends. Both these plans would be economical in the amount of metal required in the finished bridge; but erection of a structure of this magnitude is extremely difficult, and some sacrifice of economy is necessary to make the field work as safe and easy as possible. It was during the erection that the old Quebec bridge collapsed. The board consulted several of the best authorities on erection of large structures, and, while their opinion differed somewhat, it was decided, after much deliberation, to make the trusses parallel throughout. In doing so we had in mind not only the erection which was the principal consideration, but the greater simplicity of details at such important points as the pier posts and the points of suspension of the suspended span. The connections at these points become quite complicated when the anchor arm, cantilever arm, and suspended span trusses are not all in the same plane. It would have been possible to design the bridge with trusses in two planes inclined toward each other, parallel to the axis of the bridge and passing through the end supports of each truss. In this manner all connections of truss members would have been nearly as simple as in the adopted design. Such a design was also suggested and considered. But it was soon decided that the erection of heavy members in an inclined plane of the truss would be too hazardous, and this plan was abandoned. It may fairly be asked, since the Forth bridge, with its curved bottom chords, inclined and flaring trusses, has been so successfully constructed, why was it not possible to follow a similar design in the Quebec bridge? The difference is all in the labor conditions prevailing on the two continents at the respective times of building these bridges. At the Forth bridge 3,200 to 4,100 men were employed when the work was proceeding full swing; their number attained 4,600 for a short period. At Quebec such a large force could not be mustered. The contractors contemplate now using approximately 400 men in the field and not over 1,000, including men in the shops. In the Forth bridge the material was all manufactured at the bridge site. By using a large force of men it was possible to build up the various members of single plates or shapes so that no heavy pieces were handled. The admirable design, consisting principally of tubes, of which there are nearly six miles in the bridge, was built up in a similar manner as boilers are made—piece by piece. The various connections were laid out in the field, plates bent to suit, drilled and riveted on. This method of procedure would be impossible in Quebec. Not only are the men not available, but while on the Firth of Forth the climate is such that work may go on at all seasons of the year, in Quebec work aloft is impossible during more than seven months in the year. Here, then, the bulk of the work must be done by machinery to save manual labor, and must be done in the shops to permit a continuous progress. The work in the field must be reduced to the minimum or to the assembling of large pieces—as large as it is practicable to handle. The American type of pin-connected construction lends itself best to these conditions, but with that type the details will be much simpler and the erection much easier with trusses situated in two vertical and parallel planes

The system of trussing was from the beginning the object of discussion and diversity of opinion among the members of the board. The design submitted by the St. Lawrence Bridge Company, with what may be called a "K" system of trussing in the cantilever arms and anchor arms, was finally recommended by the majority of the board and later endorsed by an enlarged board appointed by the Minister of Railways and Canals for the special purpose of selecting the best tender. The main reasons for recommending the design in question are given in the enlarged board's report as follows: (*a*) The type of design offers greater safety to life and property during erection, as well as economy and rapidity in construction. (*b*) The design contains the minimum number of secondary members and requires few, if any, temporary members during erection. (*c*) The system of triangulation, by dividing the web stresses, reduces the members to more practical sections and simplifies the details of connections. (*d*) The design economizes material, as shown by the calculated weights of the two designs. (*e*) The general appearance of the structure is, in our opinion, improved. There are two advantages of this "K" design which are not clearly brought out in the above reasons, and on which I wish to lay considerable stress, namely, uniform deflections and regularity of erection operations from panel to panel. Secondary members, or those which receive their maximum stress from partial live load only, such as the vertical suspenders carrying one panel of floor, or members which carry dead load only, such as vertical sub-posts supporting the top chord, or members which normally have no stress in them, such as struts which serve to reduce the unsupported length of main compression members, are the source of local bending in the main members to which they connect. Of the designs submitted, the one adopted has the least number of secondary members. It should be remarked that the same advantage could have been obtained with a double intersection Warren truss by arranging the panel lengths in such a manner as to eliminate the intermediate vertical secondary members supporting the chords.

The regularity of erection operations consists in the fact that, starting from the pier, the position of members in each panel in the "K" design is just like the preceding one, and that coupling up of members in each successive panel, as the traveler moves forward, requires the same succession of motions as in the preceding one, except that pieces become lighter as the erection proceeds. Experience shows that the oftener an erection crew goes through a series of the same motions, as, for instance, in erecting a succession of simple spans all alike, the more rapid their progress becomes.

The lateral wind-bracing has been omitted between the top chords of the cantilever and anchor arms. All wind forces are taken directly to the pier through substantial bracing between the bottom chords. This arrangement not only makes the distribution of wind stresses perfectly definite but permits the spreading of tracks to 32 ft. 6 in., center to center, instead of the usual 13 or 14 ft., which results in a saving in the floor system, and consequently in the entire structure. With the tracks spread, a load on one track only produces a torsion in the cantilevers, and the presence of wind-bracing between the top chords would produce undesirable and excessive stresses which would have to be taken care of by a large addition of metal to the lateral and sway systems and to the trusses.

The floor system is of carbon steel throughout. It is, therefore, stiffer than if made of nickel steel. The long floor beams deflect less and the secondary stresses produced by their deflection are thus reduced. Even then some of the connections of floor beams to posts had to be made by means of pins. The top chords of the cantilever arm and of the anchor arm as now designed are of carbon steel eyebars. The originally submitted design contemplated nickel steel plates riveted throughout for the cantilevers, and carbon steel plates for the anchor arms. By substituting eyebars a better design is obtained and much easier erection assured, and, although nickel steel is replaced by carbon steel in the cantilever arm, the substitution results in a saving when both the cantilever and anchor arms are considered. Carbon steel will be used in the entire anchor arm, in the top chord and pier members of the cantilever span in the top lateral system of the suspended span, in all the floor system and all sway bracing. Nickel steel will be used in the trusses and bottom laterals of the suspended span, in the trusses except top chords and pier members, and in the lateral system of the cantilever arms. The anchor bars which hold down the ends of the anchor arms have been made very long to reduce bending stresses from expansion.

The suspender eyebars which support the suspended span are subject to oscillation in the plane of the trusses, due to expansion. A total expansion of 16 in. must be taken care of at these two points of suspension—besides the extension of the bottom chords under the live load. Manganese bronze bushings will be provided in these eyebars to permit of easy turning on the pins. But, even should these fail to turn, there is sufficient metal in these eyebars to prevent overstress from bending.

Friction brakes will be installed to prevent excessive longitudinal oscillations of the suspended span under tractive forces of trains.

All latticing of compression members is designed in proportion to the sectional material of each member. The latticing is made strong enough to transmit in transverse shear 2 per cent. of the direct stress of the member.

The bottom chords of the anchor and cantilever arms and their details were the subject of a great deal of study and of many tests. Little is known about bridge compression members when compared to tension eyebars. The Quebec compression chords are members of unusual size. It is only in work of great magnitude that the engineer has an opportunity to make tests on a large scale; the expense of such tests is trifling in comparison with the importance to the structure of the results obtained. It is not sufficient to know that in some bridges a compression member is still standing and is subjected to a certain stress. What we should know is how much greater stress it would take to destroy that member. Such a member may be in the stage of danger from the last straw. The board made a number of tests on models of chords and posts, both for the official design and for the final one. The tests gave generally better results for model members representing the latter. The board feels, therefore, that a good design for these heavy members has been obtained.

There never was any serious doubt among the members of the board as to the advisability of making the bottom chords of the anchor and cantilever arms riveted throughout without pin joints, except at the main pier bearings, to avoid excessive secondary stresses. This was done and will result in a stiffer bridge.

The original design as submitted by the St. Lawrence Bridge Company contemplated top chords built of plates entirely. While this was approved at the time, later studies proved that by building the top chords of carbon steel eyebars there will be a slight saving of weight and cost, and the change was authorized. A tension member built of eyebars is the most reliable type by reason of the large number of full-size eyebar tests which have been and are constantly being made. It is the logical form of construction for transmission of tensile stresses. Their use reduces the secondary stresses. In a chord built up of wide plates with riveted joints, making it continuous, the secondary stresses resulting from bending due to the deflection of the span would be considerable, but owing to the uniform deflection of the "K" design they could easily be taken care of.

Secondary Stresses.—I shall not dwell long on this latest addition to bridge calculations. That secondary stresses exist is a fact. They may be from three sources: *First.*—Weight of member. *Second.*—Temperature. *Third.*—Bending from loads.

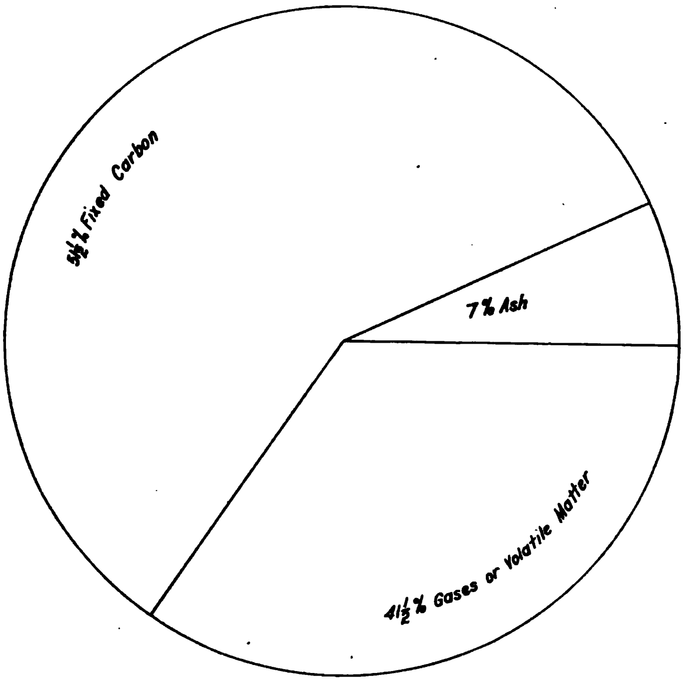
In the new Quebec design all secondary stresses were calculated and taken care of, but as a result of tests made by the board, the stresses in tension members due to their own weight will be neglected. It is quite possible that if similar tests could be made for other secondary stresses it would be found that the metal adjusts itself to a large extent in such a manner as to reduce the importance of those secondary stresses and their influence on the elastic limit of the member. Personally, I feel there is a tendency at present to overrate the importance of secondary stresses. They should, of course, be considered in designing a structure; it should be the aim of the designer to reduce these secondary stresses to the minimum, but excessive refinement should be avoided, and unit stresses for direct loads should be made low enough to include these secondary stresses where they may exist.

FUEL ECONOMY BY THE ENGINE CREW.

By M. E. Wells,

Assistant Master Mechanic, Wheeling & Lake Erie, Brewster, Ohio.

During the fall and winter our traveling engineers were furnished with counters so that they could check the number of shovels of coal used by any fireman on any part of the road. Different fireman were checked over the same piece of road and compared. This showed very favorably in some cases; as for example, the three firemen on No. 3. These men all did good work and the amount of coal they used checked to within a few shovels; but cases were found where some men used very much more than others on the same run. Some interesting comparisons obtained by means of the counters are given below.



Analysis of Average Ohio Coal Used on the Wheeling & Lake Erie.

ECONOMICAL CUT-OFF.

Under this heading there are two classes of engineers. While the ranks of one of these classes are getting thin, there are a few left. There are engineers who honestly believe in a long cut-off and a light throttle; while the other class, who may truly be called Progressives, believe in a short cut-off and an open throttle, thereby getting greater expansive force out of the steam. An interesting comparison between these two classes of men was shown on train No. 193, between Canton, Ohio, and Kent. There were 1,950 tons in each train and they had the same fireman. There were six days between the two trips and each made two stops. The engineer using the long cut-off made the run in two hours and the fireman used 327 shovels of coal, while the other engineer made the run in 10 minutes less time and the fireman used but 300 shovels of coal. This was a saving of almost ¼ ton of coal or about 9 per cent., in other words, 9 shovels of coal were saved in every hundred. If only two shovels of coal in every hundred were saved we could reduce the coal bill on our railroad $10,000 in one year, and in the United States this would amount to a saving of four million dollars per year.

HEAVY SLUG FIRING VERSUS LIGHT CAREFUL FIRING.

Slug firing is the most common of all wasteful practices, and is the hardest to correct. The following trips from Kent to Canton were compared and were otherwise the same except that fireman No. 1 had 98 tons more in his train. No. 1 used 8 shovels of coal to a fire, whereas fireman No. 2 had a lighter train and used from 10 to 20 shovels of coal to a fire. Fireman No. 2 used 254 shovels of coal, while fireman No. 1 used 228 shovels of coal, or about 10 per cent. less than No. 2. We have another comparison where the same fireman made practically similar trips between Adena, Ohio, and Rexford. On the first trip he was allowed to fire according to his regular methods. On the second trip he was coached in lighter and more careful firing with the following result: On trip No. 1 he used 220 shovels of coal, and on Trip No. 2 he used 179 shovels of coal, making a saving of 41 shovels of coal; or about 19 per cent. less than on his first trip.

OVERLOADING TANKS AT COAL TIPPLES.

The coal is handled very carefully at Brewster. The hostlers that spill the coal are required to pick it up at once, and it is done. On account of this they spill very little, and do not overload the tanks. This crusade has been carried to the tipples at other points, with good results, but it must be constantly watched and followed up.

If the enginemen and firemen on our road could only know of the poor fuel that is used in some other parts of the United States in locomotives, they would certainly feel that they were very fortunate. On many roads the coal contains so much ash and clinkers that the fireman's work is greatly increased and, in many instances, double what it is with us. The firemen should use every care to have all the available heat units in the coal consumed. The percentage of heat producing material in the average coal on the Wheeling & Lake Erie is shown in the accompanying diagram. It will be seen that 93 per cent. of the coal is available for heat, but not all of it is utilized. There is no more important matter regarding fuel economy than getting locomotive firemen to realize that practically one-half of the heat producing material in our coal is in the form of gas, and that just as soon as the coal strikes the fire these gases are driven off. If this process is not carried on slowly a large percentage of this available heat producing material goes off in gas and is wasted. A locomotive firebox can be made either a furnace for producing heat, or we can make of it, if we wish, a gas retort. If the fire is kept thin and hot so the air can get through it and the fuel is placed in the firebox in small quantities and at reasonable intervals, the gas, as I have described, will be driven off in small quantities and most of it will be burned in the firebox and produce heat as it should. After the gases are driven off there is left on the grates what is known as the fixed carbon of coal (coke) which burns without flame, in an incandescent manner. If, however, heavy charge firing is practiced and 10, 15 or 20 shovels of coal are put in at one time a gas retort is, in fact, made of the firebox. In the first place the fire is very materially cooled by the heavy charge; also much heat is consumed in driving off the gases, the result being that large quantities of gases are driven off at a temperature too low to burn with the oxygen present. In this way a large amount of the available heat producing properties of the coal pass out of the