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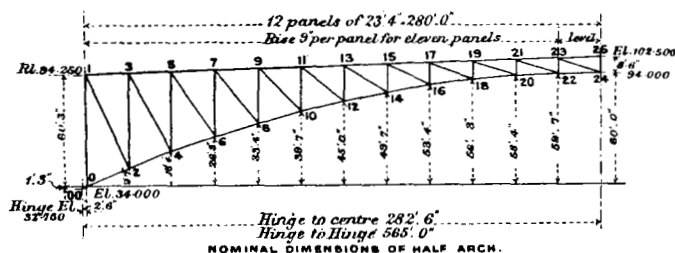
## "Design and Erection of St. John Arch."

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THE object of this Paper will be to review, in reasonable detail, the various calculations, observations and diagrams, which were made or prepared under the Author's direction, and will refer to such other features of the erection as may be of interest.

The general description of the bridge has already been given and will only be referred to here in the briefest way. The span is, nominally, a 565 foot two-hinged spandrel-braced highway arch with a rise of 61 feet 3 inches, and the unusual shallowness of the

Fig. 1.



arch had a very important bearing upon the erection scheme in so much as, in order to retain the correct elevation, very careful attention had to be paid to the length of the steelwork. If  $c$  denotes the length of the parabolic chord from the centre of the bottom chord axis to either end hinge and  $h$  the rise, then with  $L$  as total span,  $c^2 = h^2 + \left(\frac{L}{2}\right)^2$  whence  $\frac{dh}{dc} = \frac{c}{h}$  and for  $L = 560$ ,  $h = 60$ ,  $\frac{dh}{dc} = 4.76$  showing that if the half-arch built out 1 inch during erection the centre would lift  $4\frac{3}{4}$  inches unless the total span was changed.

In the plans prepared by the consulting engineer, the late Mr. C. C. Schneider, and issued with the invitation to tender, it was foreseen that the scheme of erection must be that of cantilevering each half and provision was also made for swinging the span as a three-hinged arch, and completing it as a two-hinged arch. As the contract placed the whole responsibility for correctness of figures, adequacy of material and safety of structure upon the contracting company, it became at once the business of the Author to institute a thorough and independent investigation into these questions. The important points to be borne in mind were the weight of the structure in all its varying stages and the erection scheme. The general idea of the erection was that of anchoring back each half-arch to the rock by means of a chain of tension bars and of introducing into this chain a system of adjustment and control. The system adopted was to convert the chain of bars into the upper half of a diamond toggle with hydraulic jacks in place of the usual turnbuckle rods. It will be desirable at this point to refer more exactly to the effect of the erection scheme upon the design of the truss, and for this purpose the stages of the transformation from two detached cantilevers to a complete two-hinged arch will be studied.

Under the cantilever condition, a determinable system of loading would exist, and definite stresses be carried by each member of the frame. As the two ends or arms reached the mid point, they were to be swung onto one common pin per side and the anchorage system loosened. This scheme would result in all the load then supported by the main trusses being carried by the span as a three-hinged arch, the centre hinge being in the bottom chord. When the arch had been finally permitted to take up its natural three-hinged condition under loads that were definitely obtainable, and at a temperature which could be recorded, the centre piece of the top chord could be inserted and riveted up under no stress at the existing temperature. For all subsequent loads as well as for temperature changes, the arch would now be two-hinged, and, therefore, live load, wind load, and further dead load after the mere erection dead load, would be calculated as applying to a two-hinged structure. It is believed that one of the advantages of this arrangement lies in the fact that the rigidity and stiffness of the two-hinged arch is secured without sacrificing completely the determinancy of the three-hinged lay-out. To have attempted to join up both chords at once before slacking off the anchorage system would have produced a very complicated and indeterminate structure; while to adopt the idea carried out at Niagara, in the Clifton Arch,

of artificially introducing a known load into the upper chord, after meeting on the lower chord pins at the centre, would have served to throw more of the load into a two-hinged distribution, a result not deemed to be desirable in a spandrel arch, although required in an arch rib like the Clifton Bridge.

Concerning the weight of the structure, the possession of the original plans was a distinct advantage and the checking of the stress and material diagrams was commenced in March 1913, by careful estimate of all dead loads. The actual weight of the bridge being particularly heavy and the loading only of highway type, it was apparent that the dead load would greatly exceed the live load and that every care must be taken in the determinations of dead load stress under either cantilever, three-hinged arch or two-hinged arch conditions. There were, naturally, certain items that could not be determined at such an early stage in the proceedings, such as the exact erection conditions and loads, and the precise weights of details or the consequent distribution of dead load concentrations, but, as more particulars became obtainable on these points, the corresponding corrections were applied. The first step was the preparation of tables and diagrams giving full geometrical and trigonometrical properties of the skeleton shape. Following this, simple graphical and analytical methods were used to determine the erection or cantilever stresses and the three-hinged dead load stresses. The two-hinged arch loads being statically indeterminate, a considerable amount of work both graphical and analytical was needed for their investigation.

Referring now exclusively to the two-hinged arch conditions, the first operation was the obtaining of the H curve, i.e., the curve for the horizontal component of the hinge reaction for a unit load at any point on the span between the hinges. It will be at once noticed that to make the skeleton shape susceptible to graphical analysis the end vertical 1-0 must be replaced by a member 1-00 (*Fig. 1*). For all cases where a graphic diagram was found desirable, this temporary replacement was adopted, but for the actual determination of the stresses in members 1-0, 2-0, 00-0, the exact geometrical relation was of course retained. To secure the ordinates to the H curve the following method was employed. A unit load was applied horizontally at one of the hinges 00, the reaction at the other hinge being equal and opposite, so that the stress distribution in the frame was symmetrical about the centre point 25. A graphical diagram of the stresses in the members of one-half the truss was made, and, using the gross areas of the issued material diagram, the extensions,

positive or negative, of each member under this system of loading were calculated. A deformation diagram was then drawn to show the distortion of the frame and the horizontal and vertical movements of each top chord panel point. The point 25 was chosen as the beginning point, and the direction of motion of point 24 with respect to point 25 was known to be vertical, the actual amount being zero, so that with a fixed point and a fixed direction the deformation diagram could be carried out without any correction for rotation being necessary. The scaled dimensions gave, therefore, the movement of the point in question with respect to point 25.

By the ordinary application of Maxwell's reciprocal theory,

$$H_p = \frac{\nabla_p}{\nabla_h}$$

where  $\nabla_p$  and  $\nabla_h$  are respectively the vertical deflection of the point and the horizontal displacement of the free hinge, both with respect to the fixed hinge and both measured when the system is under stress from one pound, applied horizontally at the free hinge, and  $H_p$  is the horizontal reaction at each hinge of the two-hinged arch when a vertical load of one pound is hung at the particular panel point under review. Table I gives the scaled displacements and the values of  $H$  derived therefrom. From the Table of figures necessary to compute the extensions of the individual members under a stress caused by the one pound horizontal load, it is very easy to calculate analytically the horizontal movement of the free hinge. Thus, calling the stress in any member from this specified source " $u$ ," the gross area  $A$  square inches, the length  $l$  inches with  $E$  as the modulus of elasticity, we have for the extension  $\frac{u l}{AE}$ , while

the horizontal deflection is given by  $\Sigma \frac{u^2 l}{AE}$ . Carrying this out the figure 293.845 was obtained, comparable to 293.06 by the graphic method. This was considered to be a sufficient check, and although the probabilities of superior accuracy lay with the analytical method, the fact that ratios were employed in the evaluation of  $H$  led to the retention of the graphically determined figure. As an interesting semi-check on the values of  $H$ , the formula in the case of a parabolic arch rib, with the conventional variation in the moment of inertia, was evaluated for the span under consideration, taking the rise of the bottom chord as " $h$ ." The expression is  $H = \frac{5}{8} \times \frac{L}{h} (K - 2K^3 + K^4)^{-1}$  for unit-load placed at the panel

<sup>1</sup> See Johnson, Bryan and Turneaure, "Modern Framed Structures," Part II, New York, 1911, p. 140, eqn. 19.

point designated as distant KL from an end, L being the horizontal span. (Table I and Fig. 2.)

TABLE I.

Determination of "H."					Approximate H from Arch-Rib Formula $H = \frac{5}{8} \cdot \frac{l}{h} \cdot$ (K - 2 K <sup>3</sup> + K <sup>4</sup> ).
Horizontal Load 1 lb. applied at Free Hinge and resisted at Fixed Hinge.					
Horizontal Movement of Free Hinge with respect to Fixed Hinge = 556.12 × 10 <sup>-7</sup> inches = ∇h.					
Top Chord, Point No.	Vertical Movement Relative to 25.	Differences.	Vertical Deflection Relative to 0.0 = ∇p.	H = $\frac{\nabla p}{\nabla h}$ .	
	10.7 Inches.		10.7 inches.		
25	0.0		1073.56	1.840	1.802
23	19.12	19.12	1059.44	1.808	1.786
21	59.81	40.69	1018.75	1.738	1.743
19	123.56	63.75	955.00	1.629	1.670
17	203.47	79.91	875.09	1.493	1.570
15	297.13	93.66	781.43	1.333	1.444
13	397.76	100.63	680.80	1.161	1.293
11	502.40	104.64	576.16	0.983	1.119
9	610.39	107.99	468.17	0.799	0.926
7	721.36	110.97	357.20	0.609	0.717
5	833.60	112.24	244.96	0.418	0.495
3	948.16	114.56	130.40	0.2224	0.263
1	1063.91	115.75	14.65	0.0250	0.255
Hinge } 0.0 }	1078.56	14.65	0.00	0.0000	0.0000

Another and an exact check was also carried out for the centre value of H. Taking the arch as three hinged a unit vertical load was placed at 25, and all stresses graphically obtained. H was calculated by moments, as  $= \frac{282.5}{2 \times 61.25} = 2.306$ , and used as an internal check on the graphic work.

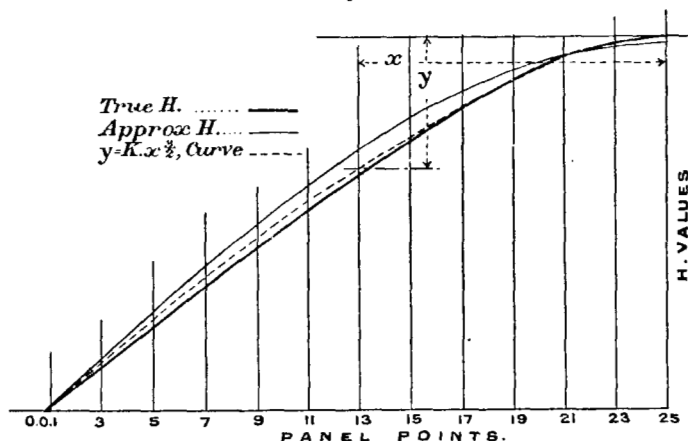
These stresses were tabulated under the heading "p". Using the stresses "u" already obtained, the movement of the free hinge  
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with respect to the centre hinge under the stresses  $p$  due to unit load at 25 was found as  $\nabla = 18.8879 \times 10^{-7}$  from  $\sum \frac{p u l}{AE}$ . Following this a unit horizontal load was applied at 25, the half-arch still being considered as hinged at 24 and 00. The  $H$  from this load is obviously  $= \frac{8.5}{61.25} = 0.1388$ , and the system of stresses is, therefore, simply 0.1388 times the system found previously for unity applied horizontally at 00, i.e.,  $0.1388 \times "u."$  The horizontal motion of the free hinge with respect to 25 under this particular load is, therefore,

$$\sum \frac{(0.1388 u)^2 l}{AE} \text{ or } (0.1388)^2 \times 293.845 \times 10^{-7} = 5.64 \times 10^{-7}.$$

(See p. 272.) Calling this deflection  $\delta$  and noting that it is of opposite

Fig. 2.



sign to the deflection  $\nabla$  above, it can be easily seen that the load necessary to be applied horizontally at 25 to reduce  $\nabla$  to zero must be  $\nabla/\delta$  or 3.35 units. The  $H$  resulting from the load would be  $3.35 \times 0.1388 = 0.465$ , whence the total  $H$  due to unit vertical load at 25 under two-hinged arch conditions, with ends fixed against motion, will be  $2.306 - 0.465 = 1.841$ , a very satisfactory check on the figure 1.840, otherwise obtained.

From the  $H$  curve and the dead load concentrations the total  $H$  for the two-hinged dead loads was at once obtained. The dead load to be applied to the two-hinged arch was designated " $d_2$ " in contradistinction to " $d_3$ " for that part of the dead load which

came on the three-hinged arch. Thus the expressions  $Hd_2$  and  $Hd_3$  will be readily understood. By the method of moments, and with the help of the tables of trigonometrical quantities previously worked out, all  $d_2$  stresses and reactions could now be determined. For the live load, however, there remained the question of distribution of loading, and to deal with this, the system of influence lines was resorted to. Influence lines were constructed for every member of the half-arch except 0-1 and 24-25. The influence diagrams were in all cases made out for moments, the Table of perpendiculars being used to determine stresses. They also were all made out with the H curve as a base. (Appendix I.)

For temperature stresses the coefficient of expansion was taken at 0.0000065, and within a range of  $\pm 75^\circ \text{F.}$ , the natural increase in span would be  $\epsilon t L = \pm 6.5 \times 10^{-6} \times 75 \times 565 \text{ feet} = 0.2753 \text{ feet}$ . From previous figures the movement of one hinge with respect to the other, if free, under  $H = 1$  was found to be  $586.12 \times 10^{-7} \text{ inches}$ , whence  $H$  due to the temperature must be  $\frac{0.2753 \times 12}{568.12} \times 10^7 = 56,300 \text{ lbs.}$ , and the stresses in the numbers

are 56,300 times the "u" stress given by  $H = 1$ . The stresses from the temperature thus derived did not enter into the determination of required section in any member of the arch. The wind load on a half-arch during erection was computed as follows: The top chord concentrations were considered as being carried down by the vertical sway bracing to the bottom chord, producing shear in the bottom laterals and vertical loading due to overturning moment on the trusses, downward in the case of the leeward truss, and upward for the windward truss. From the geometrical properties the effects of the vertical induced wind loads were easily obtained, but for the horizontal concentrations on the lower chord the latter was taken as a plane truss of length equal to the developed length of the chord, and loaded at the actual points determined by the panel points. By this means the maximum stress of 938,500 lbs. was arrived at for lower chord member 0-2. The sum of the erection stress and the erection wind stress being less than  $d_2$  plus  $d_3$ , the erection wind did not enter into the stress combination governing size of sections. For the case of wind on the finished structure it was again assumed that the top chord loads would be brought down to the bottom chord by the sway bracing, and the vertical loads from overturning moment were calculated from the H curve and influence line diagrams. The horizontal loads on the bottom chord together with the induced shear from the upper concentrations were treated as a uniformly distributed load on a continuous span equal

in length to the developed length of the chord, about 580 feet. The stresses in the bottom lateral bracing were obtained by assuming the chord developed out into a horizontal plane and equal concentrations of  $850 \times 23 \cdot 333$  applied at the panel points. The lateral system is double intersection, but each member was made sufficiently large to take the whole shear in tension. The remaining stresses to be considered were those due to erection conditions. For the chords and the whole anchorage system the governing case was obviously that occurring as each half-arch was being completed. The web members receive their worst erection loading when the traveller is just ahead of them lifting the bottom chord sections of the panel next to be placed. Inquiry was made for each member, and the stress sheet included as erection stresses the maximum loads obtained. The section of the top chord for the greater part of its length is determined from erection considerations only. Progress sheets for the instruction and guidance of the erection foreman were drawn up, showing consecutive positions of the traveller, and giving the necessary stages of riveting.

After all material had been proportioned, dead loads checked and approval of the necessary revisions obtained from the consulting engineer, the question of deflections and distortions was taken up; firstly, to enable detailing to be commenced and, secondly, to prepare for erection considerations. With regard to camber, it was decided to counteract the distortion from total dead load ( $d_2 + d_3$ ) by introducing changes in the length of each member equal and opposite to the extensions, positive or negative, caused by this load. The actual changes varied from zero to five thirty-seconds of an inch, the vertical 1-0 receiving the maximum addition. A diagram was then prepared giving the shop lengths of each member with all angles between members meeting at any panel point. A deformation diagram showing the distortion due to reverse dead loads was also plotted from the normal geometrical shape as the original. By using the same extensions or shortenings as were given to the members for detailing, this diagram indicated the shop shape under no load and thus gave the camber of top chord panel points. Deflection diagrams were plotted for maximum cantilever loads, for three-hinged dead loads, for two-hinged dead loads, and for uniform live load. With the end vertical 0-1 held vertical the sinking of point 24 under maximum cantilever conditions was determined as 11·08 inches, the diagram also showing that to keep point 24 on the theoretical centre line of the span the top point 1 would need to be tilted back 0·925 inch thus reducing the actual deflection of 24 to 6·82 inches. Again, commencing with the shop or no-load



shape with the point 24 on the centre line of the span, the addition of the three hinged dead loads ( $d_3$ ) produces a deflection of 6.555 inches at 24 and a tilt forward of 0.278 inch at point 1. Adding on the  $d_2$  loads the deflection of 24 is increased by 2.146 inches, the point 1 moving 0.256 inch inwards, thus reaching its position in the normal or geometrical shape at the mean temperature of 60° F. As will be seen later, considerable use was made of these diagrams.

The work at the site consisted of three main divisions, the provincial engineer's work in location, the substructure contractor's work on the abutments and approaches, and the superstructure contractor's work on the erection of the bridge. It is to the last division that this Paper, essentially, applies, but certain references must be made to the joining up of the steel contractor's work with the other divisions.

On April 7th, 1913, it was decided by the New Brunswick Government engineer to move the centre line of the span slightly from the originally intended position shown on the issued plans. The plans, which had by this time been carried out to some degree of detail, had to be modified for changes in contours, which changes were considerable, due to the peculiar situation of the work on extreme points of rock formations. The general erection scheme was in no way affected by this change in location, and the necessary adjustments between the Government and the Author's Company having been made, with regard to possible large increases in excavation necessary to the adopted scheme, field work was actually commenced.

The first work on the site was the laying out of the anchorage system, the preparation of the pedestals and of the anchor pits. The loads on the anchorage bars, the lengths of bars necessary and available, the shape and extent of pits were determined in the office and placed at the resident engineer's disposal as early as possible so that progress with the excavation could proceed smoothly. The preliminary field work also included the checking of the actual span across the river, and the control of the construction of the skew-back masonry to the extent of providing the seats for the steel shoes at the correct elevation and distance apart. The proximity of the existing suspension bridge, and the fact that its axis was but 3 inches out of parallel with the axis of the new bridge in its length, very naturally suggested its use as a base line, and consequently a level line was run on the south side of the north truss parallel to the axis of the arch span. The centre line of the original location of the arch had been determined by two iron eye-bolts sunk in the rock, one on either side of the gorge, and from

these points the centre line of the actual span was obtained, and the points established for the backwall faces. On 13th September, after the west end half of the arch had been erected and the masonry completed for the east end skew-backs, a piano wire previously calibrated was stretched across the gorge on the actual centre line of the south truss. This method of final checking was considered advisable as the correct setting of the castings was, of course, exceedingly important. (Appendix II.)

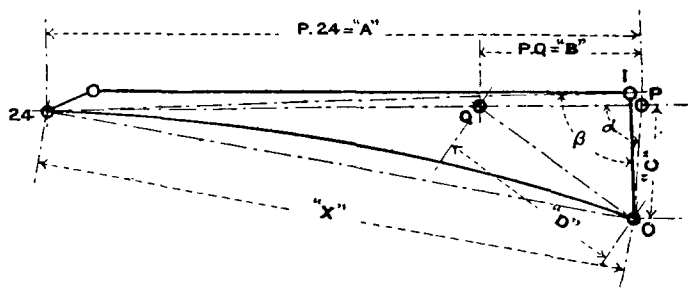
The steel and equipment for this preliminary work was unloaded from the Canadian Pacific Railway tracks at a siding already existing on the west approach to the bridge. As the anchorage work progressed preparations were made for the locating and building of proper approach tracks with private sidings. The facilities were obtained from the Canadian Pacific Railway, city and county authorities, and Railway Commission for the construction of sidings on each side.

To cross over Douglas Avenue bents were built at the east edge of the street and at the curb of the western sidewalk, and 12-inch broad flange beams were used to span the intervening 25 feet of road. The west end trestle carried the Canadian Pacific Railway shunting engine over its whole length, the railway flat cars being backed up to the end of the span where the 12-ton stiff-leg derricks unloaded the steel onto the narrow gauge trucks which ran out on the span and served the main traveller. At the east end the Canadian Pacific Railway merely shunted the cars onto the siding whilst one of the contractor's own yard derrick cars transferred them to a point near the bridge end, but east of the street where the stiff-legs again unloaded the steel on to the small trucks. Thus the crossing over Douglas Avenue had to be designed for the lorries, or possibly the flat cars or the yard derrick, and not for the railway company's locomotive. The comparative axle-loads were as follows: Canadian Pacific Railway's small locomotive 46,000 lbs. per axle, three such axle-loads at 5 feet 7 inches centres, flat car and load of steel 24,000 lbs. per axle, yard derrick 25,000 lbs. per axle. The lifting capacity of the yard derrick being only about three or four tons, both twelve ton stiff-legs were needed to handle the heavier members at either end, and they were consequently set up together just off the abutments at the west end and beyond the street at the east.

Dealing now with the arch trusses, the first observations were in the nature of a shop survey of each half truss as it was assembled. These surveys were intended to determine the true no-load shape of the frame, and were thus a check on the camber allowances. A certain camber was aimed at by altering the lengths

of the members and the use of these new lengths produced a distorted shape. To insure that the shape thus produced was actually obtained in the manufacture, a camber diagram is usually added to the assembly drawing, and definite ordinates are calculated, analytically or graphically, from some easily established base line to certain panel points (*Fig. 5*). In the present case the top chord, being normally a straight line of some 280 feet in length, became an obvious base line, and the distortion of this line into a curve, concave upward, presented the required opportunity for providing the desired ordinates. Unfortunately, however, this camber diagram was omitted from the first assembly drawing issued to the shops, and the first half truss was wrongly assembled with the top chord approximately a straight line. The changes of length had been properly introduced into the individual members, yet by plentiful reaming of the sub-drilled holes in the ends of the web members,

Fig. 5.



the truss was forced into an incorrect shape. The assemblers actually noticed that at the centre the diagonals required an undue amount of reaming to secure proper matching, and that the faced joints of the bottom chord had to be forced more than usual to bring them into true bearing. The shop survey not only made this error manifest but made it possible to minimize the effect in the field, and to prevent its repetition in the shop. The exact position of point 24 was established on the steel by filling the pin-hole with a wooden plate and carefully scribing thereon the centre lines of intersecting members, the resulting point being the centre of the circular hole, as bored. The points O were also carefully established at true intersections, but the points P where the instrument was set up were not coincident with point I. From the taped distances B, C, D. the angle  $\alpha$  or O-P-24 was calculated and then checked by actual observation. From the measured distances A and C and the angle

$\alpha$  the length  $X$  was figured. No attempt was made to check this on the ground, due to physical conditions, obstructions, and the difficulty of supporting the tape. The points  $Q$  were any convenient points on the steel near panel point 7, and on the line  $P-24$ . The actually observed angles  $\alpha$  were not directly comparable due to the slightly different location of  $P$ , but initially they were only intended as means to an end, namely, the distance " $X$ " and the subsequent use made of them was not originally anticipated. A simple calculation will, however, determine the angle  $O-1-24$ , the  $\beta$  of the table, which is a strictly comparable function. A study of Table II will indicate that the east end trusses cocked-up more than the west end. It may, perhaps, be considered a somewhat unusual proceeding to call special attention to actual mistakes, but it is felt that in a large degree it is the mistakes that are important, not simply to those engineers who make them or experience their effects, but also to others who may wish to note them with a view to their future avoidance.

The first important observation in the field was the checking of the exact position of points 1 at the top of the end verticals, relatively to the point 00, the bottom or skew-back hinge. From a study of the deflection diagrams under full cantilever load and a consideration of the operations incidental to closing the span as a three-hinged arch, a definite position of point 1 was desired. It was intended so to arrange matters that the jacks in the toggle systems would not have to be raised, but simply lowered during closing operations. To achieve this it was necessary to secure that the extreme points 24, which, ultimately, would become coincident on the centre pins should be primarily located distinctly high. After making due allowances, the excess height of  $3\frac{1}{2}$  inches was fixed upon as the necessary extra-elevation of these points. Obviously with a known shape of frame and the point 00 fixed in space the choosing of an elevation for point 24 determined the co-existing position of all points in the frame, including point 1. In the normal geometrical shape, theoretically that existing under full dead load, the point 1 was to be vertically above the point 0, and as any movement of the arch frame must of necessity be one of rotation about the hinge pin, the end verticals would, once erected, become convenient if not ideal indicating members as to the position of the trusses. Their inclination to the vertical in the plane of the arch was therefore constantly being observed and recorded. The intended original position of point 1, previous to any appreciable load being carried on the toggles, was settled as  $4\frac{1}{8}$  inches behind the vertical. The eye-bar toggle system was laid out to bring about this backward

TABLE II.

Truss.		Measured Items. Not Comparable.						Calculated Items.		
End.	Side.	B	C	D	A	α	Not Comparable.		Comparable.	
							α	β		
1 West	North	Ft. Ins. 70 0 $\frac{3}{4}$	Ft. Ins. 59 8 $\frac{2}{16}$	Ft. Ins. 91 2	Ft. Ins. 280 0 $\frac{1}{4}$	90° 34' +	90° 34" 23'	Ft. Ins. 286 5 $\frac{2}{16}$	89° 58' 30"	
2 West	South	73 2 $\frac{1}{4}$	57 7 $\frac{7}{32}$	93 6 $\frac{5}{32}$	280 1 $\frac{3}{16}$	90° 28' +	90° 28' 40"	286 5 $\frac{3}{16}$	89° 56' 00"	
3 East	North	70 2 $\frac{7}{8}$	57 9 $\frac{1}{4}$	91 5 $\frac{1}{32}$	280 0 $\frac{1}{8}$	90° 36'	90° 36' 37"	286 6 $\frac{1}{2}$	90° 06' 01"	
4 East	South	70 0 $\frac{1}{2}$	57 5 $\frac{1}{2}$	91 1 $\frac{1}{8}$	280 1 $\frac{1}{8}$	90° 41 $\frac{1}{2}$ '	90° 41' 47"	286 7 $\frac{7}{32}$	90° 06' 58"	

<sup>1</sup> Measured up about 21st June, 1914, at a prevailing temperature of 82° F.<sup>2</sup> " " " 14th June, 1914 " " 80° F.<sup>3</sup> " " " 10th August, 1914 " " 75° F.<sup>4</sup> " " " 31st August, 1914 " " 75° F.

tilt of the end vertical, and plate links were necessary to extend the eye-bars to reach to the top chord of the arch. Owing to another error of transmission of information between offices, these plate links on the first or west end were not shortened for this initial hold-back, and when the Resident Engineer made his checking observations on the primary positions of point 1 he found the end posts to be plumb. The only course was to compromise, and the jacks were run up to bring back the vertical end posts to a 3-inch tilt. This upward movement of the jacking pin involved quite serious changes of load, particularly on the jacks themselves, and careful calculation as to final conditions had to be made. To continue progress with the end post 01, initially vertical, would have landed the point 24 some 19 inches lower than had been determined as desirable. The  $4\frac{1}{8}$  inches had therefore to be dissected again and its component parts studied. They consisted of three portions, firstly,  $1\frac{3}{8}$ -inch representing the movement of the pin at 1 as part of the toggle while extending under the gradually increasing load, a second portion equal to 2 inches which represented the theoretical tilt of the arch end post, when the pin at 24 connecting the two halves could be entered, and finally  $\frac{3}{4}$  of an inch which was a safety allowance adopted to make certain that the end points 24 would finish high and not low, of the pin-connecting position. This  $\frac{3}{4}$  of an inch allowed for the  $3\frac{1}{2}$  inches extra height at 24, part of which was expected to be taken up by the droop of the truss. It was intended originally that the maximum load should be reached when the jacking strut had reached a vertical position. With the  $4\frac{1}{8}$  inches initial backward tilt, the pin at the top of the jacking strut was to be 1 inch behind the pin at the bottom, moving forward this 1 inch as the full cantilever load came on, and, as the jacks were lowered while the span was changing from two cantilevers to a three-hinged arch, this upper pin was expected to travel riverwards about two more inches, finishing with a tilt of 2 inches in the jacking strut when the condition of slack toggles was reached. The error, under review, sadly interfered with these arrangements on the west end, but they were very well followed on the east end. Holding back 3 inches instead of  $4\frac{1}{8}$  inches eliminated, at once, the safety allowance of  $\frac{3}{4}$ -inch. It also rendered it exceedingly likely that lifting on the jacks would become inevitable, but the higher the jacks were set initially the worse would the lifting load become, if lifting had eventually to be done. Thus the compromise figure was adopted, and the actual lifting was very small and was quite successfully accomplished.

Beginning from these known and observed initial conditions,

forecast drawings were prepared of the movements of the critical points, such as point 1 and the top pin of the jacking strut, and constant checking was carried out at the site. The prospective movements of the jacking strut and toggle members could be studied, and due precaution taken that the limits of movement and strength were in no danger of being exceeded. The effects of temperature were also studied from the progress reports, observations for alignment and elevation of the two trusses being taken, with any weather change (Appendix III).

Having arrived at the three-hinged stage the determination of the exact dimensions of the centre top chords became necessary so that these latter could be entered into place without taking load under present conditions, but so fitted that for any further dead load they would come at once into action and enable the span to operate as a two-hinged arch as far as the newly applied loads were concerned. The gusset plates at points 23 were riveted to the chords 19-23 and were thus in place with their open holes ready to receive the centre members. These members were manufactured and completely assembled but with their extremities as yet unfaced and with sufficient extra length to be certain of meeting the field conditions. On October 20th, when the toggles had been completely slackened and the span had thoroughly settled into its three-hinged shape, very careful measurements were made of the opening between the upper chord points 23 on either truss. The members were marked and faced to the exact dimensions so provided, but the open rivet holes were left  $\frac{3}{8}$  inch in diameter so that final reaming to perfect fairness could be carried out when the chords were in place. On the 29th the north side member was entered. The bottom edge being 40 feet  $9\frac{3}{4}$  inches against 40 feet  $9\frac{1}{4}$  inches on the top at 48° F., the prearranged temperature, the member could not be lowered into place without drawing up the span by the toggles. It could not be entered from below because of the wind bracing connection plates nor from the sides because of the gusset plates so that the jacks were pumped up and the opening enlarged by  $\frac{3}{16}$  inch. The gauge on the jacks registered 250,000 lbs. at each jack when this condition had been secured. The north chord 23-23 was lowered into position and bolted sufficiently to hold it until ready to ream.

The fit of the centre chords was perfect in all particulars and the stress conditions are, therefore, as near those intended as could possibly be secured. The span now having been swung, the effect of temperature on the elevations of the centre pin became a necessary item of study.

Table III gives the relative elevation of pins at 24 N. and S. to points 1 N.E., S.E. and S.W., as observed on the dates shown and as corrected to normal temperature.

TABLE III.

Date.	Temperature Fahrenheit.	Relative Position of Centre and End Pins.	
24 October .	50°	$\left\{ \begin{array}{l} 24 \text{ N. } 3\frac{3}{4} \text{ inches below 1 N.E.} \quad . \quad . \quad . \\ 24 \text{ S. } 2\frac{3}{4} \quad , \quad , \quad 1 \text{ S.E. and 1 S.W.} \end{array} \right.$	Readings observed.
"	60°	$\left\{ \begin{array}{l} 24 \text{ N. } 2\frac{3}{4} \quad , \quad , \quad 1 \text{ N.E.} \quad . \quad . \quad . \\ 24 \text{ S. } 1\frac{3}{4} \quad , \quad , \quad 1 \text{ S. and 1 S.W.} \end{array} \right.$	As corrected by formula. <sup>1</sup>
6 November	35°	$\left\{ \begin{array}{l} 24 \text{ N. } 5\frac{1}{4} \quad , \quad , \quad 1 \text{ N.E.} \quad . \quad . \quad . \\ 24 \text{ S. } 4\frac{5}{16} \quad , \quad , \quad 1 \text{ S.E. and 1 S.W.} \end{array} \right.$	Readings observed.
"	60°	$\left\{ \begin{array}{l} 24 \text{ N. } 2\frac{3}{4} \quad , \quad , \quad 1 \text{ N.E.} \quad . \quad . \quad . \\ 24 \text{ S. } 1\frac{1}{8} \quad , \quad , \quad 1 \text{ S.E. and 1 S.W.} \end{array} \right.$	As corrected by formula. <sup>1</sup>

Again, starting from the actual three-hinged arch at 60° F. and adding the theoretical two-hinged deflection of  $2\frac{1}{8}$  inches, the figures  $4\frac{3}{4}$  inches and  $3\frac{3}{4}$  inches are obtained as the final normal distances by which 24 is lower than 1, remembering that the  $d_2$  load lowers point 1 by  $\frac{1}{8}$  inch. Reference to the geometrical shape will show that this was intended originally to be 3 inches so that the actual arch is  $1\frac{3}{4}$  inch and  $\frac{3}{4}$  inch lower than specified at the centre pin on the north and south sides respectively, the mean of these two amounts being  $1\frac{1}{4}$  inch.

Reference will now be made to the length of the span and to the observations in connection with it. Measurements between points 0 and 24 were made on the west end on 1st September, and by comparing the results corrected for temperature and erection stresses with the shop survey of the trusses, the amount of *build* was obtained. This phenomenon of *building out* is a well recognised one in steel erection and only becomes a serious matter in cases such as this where a long stretch of framework is cantilevered out from either side of an opening to meet in mid-air at a specified

$$^1 \Delta = \frac{0.0000065 \times t \times L^2}{4h} \quad \text{Where } L = \text{span between hinges, } t = \text{temperatures}$$

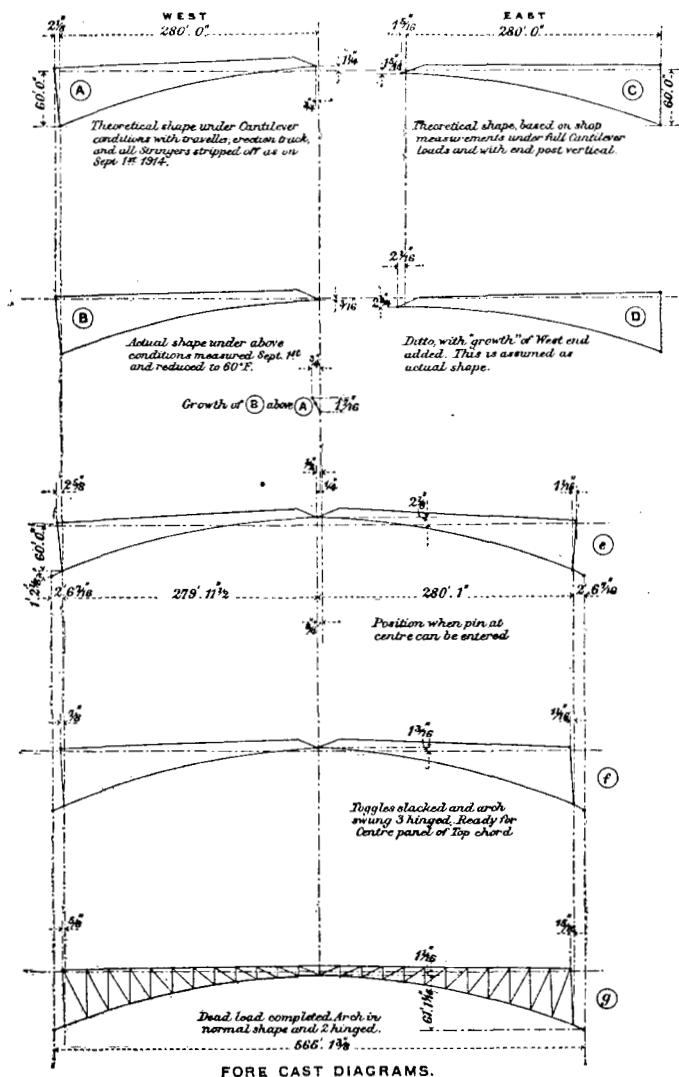
change,  $h$  = normal height from hinge to centre pin, and  $\Delta$  the variation in elevation of centre pin, due to  $t$ .



point. The observed amount was about  $\frac{3}{4}$  inch, and this was then adopted as the probable build on the east side not yet begun. By means of the diagrams of shapes and movements the effect of *build* and *droop* were studied. *Build* by itself would demand that the span be made longer to accommodate the increase in steel length, but the accompanying phenomenon of *droop*, due largely, of course, to certain inevitable slackness of joints and slipping of rivets or bolts, operated in the opposite direction and necessitated the tilting up of the trusses to retain the true elevation of the centre-pin, and this tilting meant a corresponding shortening of the span. It was decided that an increase of  $\frac{1}{2}$  inch in the span between skewback hinges would give the most desirable compromise, and the east end shoes were set accordingly. The *droop* recorded on the west end was  $1\frac{7}{8}$  inch, and although it was confidently believed that the east end could be erected with less than this amount, an equal allowance was made in the above mentioned calculations. A comparison between the expected elevations after correction and those actually recorded on 24th October and 6th November indicates that the north-east truss must have drooped slightly more than the allowed amount, with the result above mentioned, that the final elevation of the N centre-pin was about  $1\frac{3}{4}$  inch low of the geometrical shape instead of only  $1\frac{1}{8}$  inch as forecasted. Similarly the south-east truss must have behaved better than the west end (*Fig. 8*). The shop shapes are the cause of the considerable differences in the conditions of the two halves as shown in the diagrams of this figure. The change of  $\frac{1}{8}$  inch in elevation of centre hinge between *Fig. 8 e* and *f* needs explanation in view of the previous statement that the deflections from the normal shape with 24 held on the centre line were  $6\frac{1}{8}$  inches for the cantilever loading and  $6\frac{9}{16}$  inches for the three-hinged. This applies to the east end where the loading was that for which these figures were obtained, but for the west end the cantilever deflection under existing loads was but 4.34 inches, tilting back to keep 24 on the centre line, while the three-hinged deflection under the same loading was 6.41 inches. The calculated result from these figures indicates that on linking up at the centre and slacking off at the ends the pin at 24 would sink about  $\frac{1}{8}$  inch. *Fig. 8 f* also shows the east end post leaning forward  $\frac{1}{8}$  inch and the west leaning backward  $\frac{7}{8}$  inch. On October 26th at 51° F., when the conditions of loading were exactly as required, the actual tilt of the end posts was 1 inch back on south-west,  $\frac{1}{8}$  inch back on north-west,  $\frac{9}{16}$  inch forward on north-east and  $\frac{7}{16}$  inch forward on south-east. The correction for temperature would theoretically increase the west end figures and reduce the east end

by  $\frac{3}{16}$  inch, leaving  $1\frac{3}{16}$  inch and  $\frac{1}{4}$  inch to compare with  $\frac{7}{8}$  inch and  $1\frac{1}{8}$  inch on the diagram. On the same date the length of the two ends

Fig. 8.

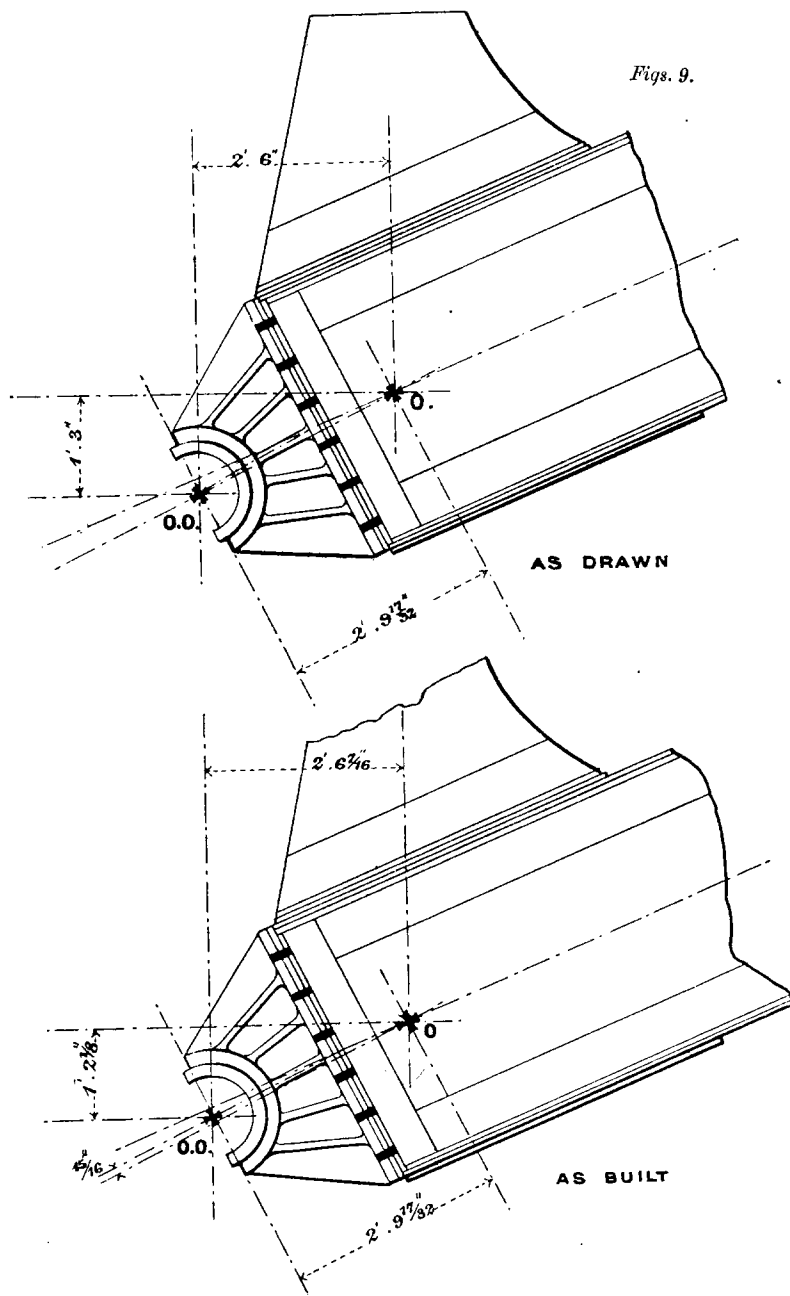


was taped and the figures corrected for temperature were 280 feet  $\frac{3}{32}$  inch for the east and 279 feet  $11\frac{1}{2}$  inches for the west end. Due

to weather conditions these measurements were made on the north side, but there is no reason to doubt but that the south side, which is that shown in the figure, is substantially the same in this respect. The field height on October 24th, when corrected to 60° F., was 60 feet  $1\frac{3}{8}$  inch on the south and 60 feet  $\frac{3}{8}$  inch on the north against 60 feet  $1\frac{3}{16}$  inch for the south on the forecast diagram, variations in *droop* causing these discrepancies. The final dead load shape is indicated in Fig. 8g, but there are no field checks available for comparison. As, however, the deflection and deformation for the  $d_2$  loads may be presumed to follow the theoretical calculations and no further chance for error would appear to exist the final shape may be stated with a very close approach to certainty. Still alluding to the south side, in so far as a distinction between sides is needed, the span is 565 feet  $1\frac{3}{8}$  inch, the rise between hinges 61 feet  $1\frac{3}{8}$  inch, the slope of the end verticals  $\frac{1}{2}$  inch forward at the east end and  $1\frac{5}{8}$  inch backward at the west end.

In Fig. 9 as well as Fig. 8e it will also be noticed that the vertical and horizontal dimensions between 0 and 00 are not those originally laid out. The change of  $\frac{7}{8}$  inch in the vertical dimension is mentioned in Appendix III and opportunity will now be taken to offer a fuller explanation. On checking elevations early in August when the first panel of the west end had been erected and the toggle bars had become sufficiently well strained to permit the removal of the timber blocking and spacing pieces from between the bars, the vertical height from the skewback hinge pin 00 to the pin at point 1 was found to be short. Repeated observations gave point 1 low but point 00 correct, and it was also found that the vertical component of the dimension 00-0 only scaled 1 foot  $2\frac{1}{8}$  inches instead of 1 foot 3 inches (Fig. 9). A comparison of the drawings with the steel revealed the discrepancy which it was discovered had resulted from a careless reading of the casting drawing by the pattern-maker. The drawings were made so that the centre rib of the casting had its axis along a line radiating from the pin centre and meeting the planed surface at the point where the geometrical central axis of the chord member 0-2 intersected this surface. The matching bolt holes were drilled symmetrically about this intersection point both in the casting and in the bearing plate of the chord, the ribs of the former being so designed to permit bolt heads to come between them. The thrust line 0-00 normal to the planed bearing surfaces intersected the line of the bearing about  $1\frac{5}{8}$  inch below the above-mentioned point, and it was the omission of this  $1\frac{5}{8}$  that caused the error. The central rib was made to coincide with the thrust line and the bolt holes

Figs. 9.



were symmetrically disposed about it, so that while everything fitted together perfectly the chord had virtually been slid down the face of the casting and was  $\frac{7}{8}$  inch too low and  $\frac{1}{16}$  inch too far forward. The slope of the line 0-00 was no longer 1 in 2 and was not normal to the bearing surface. It was not practicable to effect any correction for this error, and the only procedure was to prepare the east end skew-back seat, as yet uncompleted, to permit the centre of east hinge pins to be moved  $\frac{7}{8}$  inch eastwards and to ensure that the seat of the approach girders on the abutments would accommodate the lowering of the arch end of these girders by  $\frac{7}{8}$  inch. The question of eccentricity of thrust was studied, and calculations showed that the maximum shear along the member 00-0 normal to this line as an axis was increased from 199,000 lbs. to 339,000 lbs., the worst condition being for full deadload and partial live load, and whereas in the intended condition this cross shear varied with varying load from 199,000 lbs. to - 90,000 lbs., it now remained constantly of the one sign, the minimum being about 9,000 lbs. and the maximum as stated above. The bending moment caused by this shear equals 11,360,000 inch lbs. and must be taken up by the pin friction, the bending value of the vertical 0-1 and the bending value of the chord. The latter will take by far the larger proportion and may well be considered as absorbing the whole. In such a case the induced fibre stress will be given by

$$f = \frac{My}{I} = \frac{11,360,000 \times 22 \text{ inches}}{83,300} = 3,000 \text{ lbs. per square inch,}$$

whereas for the original condition when the cross shear would have reached 199,000 lbs. this unit would be 1,760 lbs. per square inch. The sign of the moment is such that the compression occurs in the upper fibres of the member. Any considerable pin friction would not only relieve this chord of a certain portion of this amount but would reduce the amount occurring at the planed bearing surface where it would not be desirable to vary the unit pressure too much. There is, of course, no danger of any tension occurring across this joint as, even assuming the moment to vary from zero at the pin 00 to the maximum at the point 0, the fibre stress at the break, counting on the section of the chord as above, would be about 1,840 lbs. per square inch as against about 13,840 lbs. per square inch, which is the co-existing pressure unit. Under these conditions it was felt to be quite safe and satisfactory to let the matter remain as it was, to lower the arch span  $\frac{7}{8}$  inch and lengthen the span between skewback hinges by an equal amount, i.e.,  $\frac{1}{16}$  inch for each half-arch. The  $\frac{7}{8}$  inch thus obtained, together with the  $\frac{1}{2}$  inch otherwise determined, explain the lengthening of the span

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to 565 feet  $1\frac{3}{8}$  inch (*Fig. 8 g*). It may be inquired why the fact that the centre hinge finishing low, being foreseen, was not prevented by keeping the span somewhat shorter, say  $(1\frac{1}{8} \times 2 \div 4.6 = 0.46$  inch) 580 feet between hinges, which would just about have kept point 24 at its proper level. To lift the west half any more than actually was done would not only have been acting in conflict with the original hopes and intentions that no upward jacking would be needed, but would also have increased the load on the jacks to an amount nearer to their rated capacity than was considered desirable. The maximum load registered on the west end jacks was 330 tons per pair, and this amount was recorded as the truss of the half-arch was let down to enter the pin on the north side. A slightly smaller amount was shown by the gauge when the west end was raised to permit the entrance of the bottom chords of the east end (22-24). It was recognised as possible, that to enter the pin it might have become necessary to raise the west end slightly, such as would have been the case had the east end "drooped" less and "built out" more and the predicted remaining jack capacity of 50 or 60 tons was accepted as sufficient provision for this possibility but not as sufficient to warrant an arrangement which would call for a further and deliberate raising of the half-arch. The greatest recorded load on the east end jacks occurred at the same time as on the west. Although no actual movements were made, the load in the jacking struts was transferred from the blocking to the jacks and the gauges read practically 150 tons on each jack on the south-east corner, but, unfortunately, failed to work on the north-east toggle. The calculated amount for this condition was 139 tons per jack. Several items contribute to explain the deviation, among them being the tilt of the end post, temperature and various small inaccuracies in lengths and assumed extensions under load. Incidentally as a point of interest, it may here be stated that at the rocker shoe above the mouth of the pit a movement of the rocker on the pin equal to  $\frac{1}{16}$  inch at the circumference was observed in all the four cases, this occurring between the condition of no-load and the condition of maximum tension in the toggle bars.

Throughout the anchorage system the steelwork was figured on a basis of 20,000 lbs. per square inch working load under tension.

A very troublesome situation has since arisen due to the subsequent erection of a sulphite process paper-mill on the riverside, a few hundred yards away from the bridge. The fumes from this mill, mixing with the salt air are wafted by the air currents underneath the bridge and lodging under the floor have quickly eaten the paint and

caused corrosion of the steel floor stringers. The main structure is unaffected as the currents keep the air and fumes moving, but close up under the concrete the rusting effect is distinct and even serious.

The roadway surface has not proved an unqualified success, as a certain seepage of rain-water takes place around the tramway rails.

Lastly the author would make his acknowledgements to the numerous young engineers and assistants, who have aided him from time to time on the preparation of the diagrams. No one regrets more than the Author the untimely death of Mr. Walter E. Bradshaw, B.Sc., A.M. Can. Soc. C.E., the resident engineer on the work, to whose careful work at the site so much of the success and all of the interesting and useful observations are due. Mention must also be made of the excellent work of the erection foreman, Mr. Chas. E. Fitzsimmons, who, together with so many of the assistant engineers, draughtsmen and others employed on this bridge is now (1915-1919) on active service overseas.

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## APPENDIXES.

## APPENDIX I.

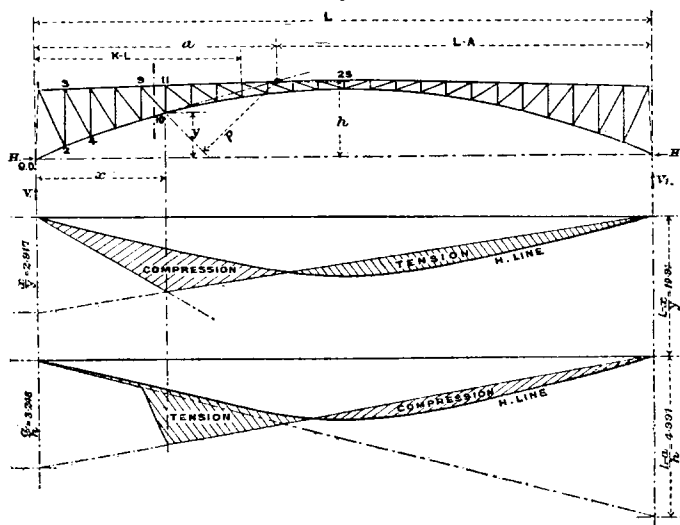
## INFLUENCE DIAGRAMS.

A typical computation for a chord and web member indicating the method followed.

Example 1. Chord 9-11 (*Fig. 3*).

Centre of moments is point 10.

*Fig. 3.*



For a load at any point between 11 and right end such as KL from 00,  $V = (1-K)$  times load, or if load = 1;  $V = 1 - K$ . Also, H is shown by ordinate to H line under point, KL from 00.

$$\begin{aligned} \text{Moment at 10} &= Vx - Hy \\ &= (1-K)x - Hy = y \left( \frac{(1-K)x}{y} - H \right). \end{aligned}$$

The quantity inside the brackets is plotted on the diagram, the  $y$  outside remaining as a multiplier. The H line being previously laid down, the expression  $\frac{(1-K)x}{y}$  remains to be determined. It is obviously a straight line passing

through zero when  $K = 1$ , that is at right end and having value  $\frac{x}{y}$  when  $K = 0$  that is at left end. For a load between left end and point 11, the point of loading still being designated as KL from 00,

$$\begin{aligned} V_1 &= K, \text{ and the moment at 10} = V_1(L-x) - Hy \\ &= K(L-x) - Hy = y \left( \frac{K(L-x)}{y} - H \right). \end{aligned}$$



By similar means to that above detailed the straight line  $\frac{K(L-x)}{y}$  is plotted, the right end ordinate being  $\frac{L-x}{y}$  as  $K = 1$ . This line naturally cuts the first line directly under point 10. The resulting shaded figure represents the expression for moment, when multiplied by " $y$ ," the sign being the same as the sign of the moment due to  $H$  alone when the curve of  $H$  is below the straight lines bounding the shaded area, and of the opposite sign where the  $H$  curve is above the straight lines. In the case under review for chord member 9-11, the horizontal reaction  $H$  would by itself put tension in the member, so that the expression for moment  $Vx - Hy$  represents compression if positive in sign, that is, if the "straight line" term involving  $V$  is numerically greater than the "curve" term involving  $H$ . In this manner the portion of the span to be loaded to produce maximum compression is indicated by the diagram, and consists of that length between the left end and the point where the  $H$  curve crosses the straight line, while if the load were confined to the other portion of the span between the crossing point and the right end, the maximum tension in 9-11 would be obtained.

Example (2), Diagonal 9-10 (*Fig. 3*).

Centre of moments is on line of top chord at a distance " $a$ " = 222.750' from left end. For unit load at  $KL$  lying between 1 and 9,  $V_1 = K$  and  $M = V_1(L-a) - Hh = K(L-a) - Hh = h \left( \frac{K}{h}(L-a) - H \right)$  whence  $h$  becomes the multiplier, and  $\frac{K}{h}(L-a)$  the "straight line" term. For the position of the line, inserting  $K = 0$  indicates that it passes through zero at the left end, and by  $K = 1$  the slope is determined, the right-hand end ordinate becoming  $\frac{L-a}{h} = 4.991$ . The line can then be drawn from the left end to the vertical from 9. For unit load lying between 11 and the right end  $V = (1-K)$  and

$$\begin{aligned} M &= Va - Hh = (1-K)a - Hh \\ &= h \left( \frac{(1-K)a}{h} - H \right) \end{aligned}$$

whence the multiplier is still " $h$ ," the straight line being  $\frac{(1-K)a}{h}$ .

For  $K = 1$  the right end ordinate is zero, and for  $K = 0$  the left end is  $\frac{a}{h} = 3.248$ . The line can now be drawn from the right end to the vertical from 11, and by joining the points made by the intersection of the influence lines with the verticals the diagram is completed. The intercept at any point between the  $H$  line and the straight line bounding the shaded figure represents  $\frac{M}{h}$ , or when multiplied by " $h$ " represents the moment due to a unit load placed at that point. Thus the shaded area represents the total amount due to uniformly distributed load. The sign is determined as for the chord influence diagram, and the portions to be loaded to produce maximum tension or compression stress are at once seen from the figure. As the specified loading was uniformly distributed the mere areas of diagrams such as those delineated above had to be determined for each case of every member. This integration was performed by planimeter.

The area of the diagram giving, as it does, in all cases  $\frac{M}{y}$  or  $\frac{M}{h}$  the dimension of the result is moment divided by length or, merely, pounds.

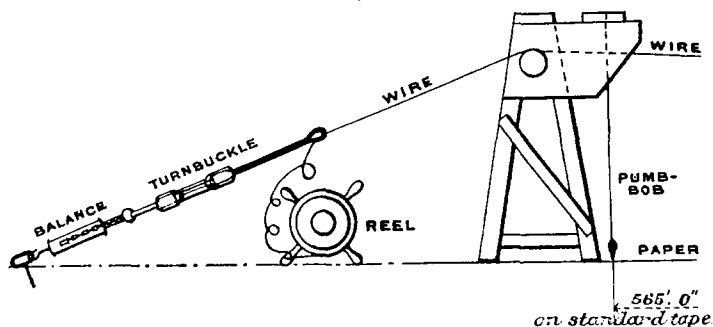
The H line being actually polygonal the area between it and the base line representing the total H when span is fully loaded could be computed from the successive trapezoids, as well as planimetric integration. The possibility of such a check was taken advantage of, and the respective figures were 918·912 by computation against 918·00 by integration. As before mentioned, member 0-1 had to be specially treated both because of its not meeting the hinge, and because of its receiving direct load from the approach span. This extra load at point 1 has no appreciable effect on the stress in any other member of the system.

## APPENDIX II.

### CALIBRATION OF PIANO WIRE.

The calibration of the piano wire was carried out in the girder shop of the Company's works at Lachine, and in the following manner: The wire, about 750 feet long, was coiled on two reels, specially made for the purpose, and placed some 600 feet apart on the floor of the girder shop, and the wire stretched across two wooden trestles about 567 feet apart. On the floor, for a few feet in the neighbourhood of the trestles, was pinned drawing paper and a standard 100-foot Lufkin tape, under 9 lbs. pull was used to establish accurately two fine hard-pencil marks on the paper 565 feet apart. A definite pull was introduced into the suspended piano wire by means of spring balances properly checked (*Fig. 4.*)

*Fig. 4.*



CALIBRATING PIANO-WIRE.

The wire when submitted to this 57 lbs. pull took up a position of equilibrium, giving 1·8 feet of sag. On each trestle a shelf bracket carried a sliding support from which hung a plumb bob, whose point was within  $\frac{1}{32}$  inch of the paper on the floor. The line of the plumb bob touched lightly the wire as it passed, and when, by means of the adjustable support, the bob was set precisely on the pencil mark, a brass tag brazed on to the wire was marked with a file opposite the bob-line. Both ends were so marked and all notes as to conditions taken. The temperature was 62° F. throughout and the shop entirely free from wind, sun or vibration, as the calibration was effected when the works were not in operation.

The result of the piano wire check was that the span, as already laid out, proved  $\frac{3}{16}$  inch longer than the wire, but the original length was adhered to, as representing 565 feet.

### APPENDIX III.

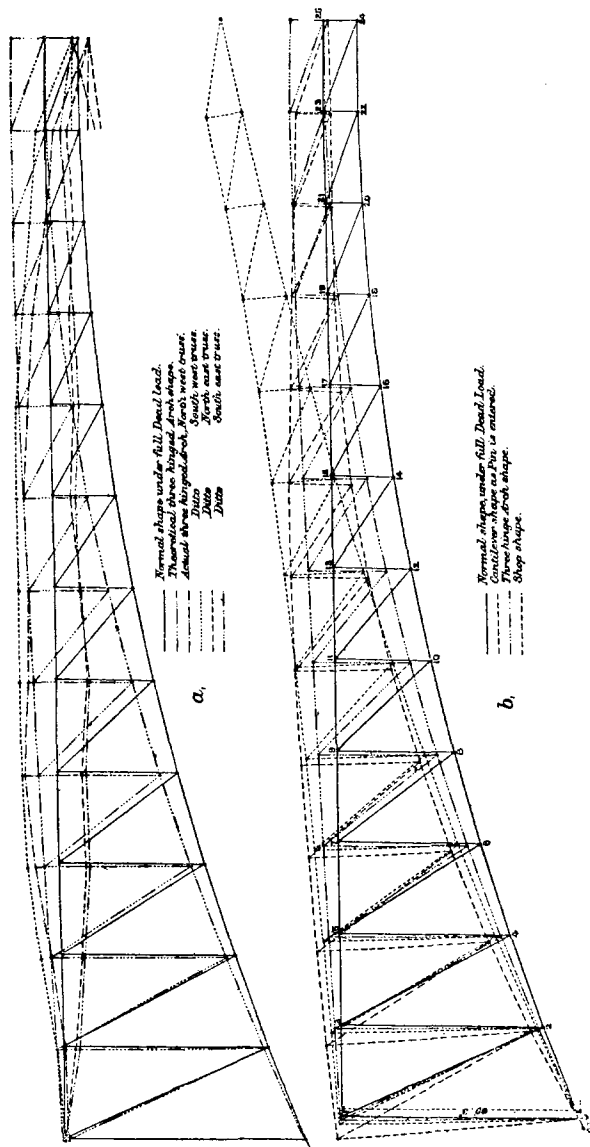
#### DEFORMATIONS DUE TO CHANGES IN TEMPERATURE.

A pronounced temperature effect was recorded in connection with the erection of the west half during August and September, 1914. The south side truss being much more directly heated by the sun's rays, had lengthened during the placing of members, and riveting was done under this uneven temperature condition. The last four panels were placed in very warm weather, at the end of August, and when the erection was finished for the one arm, the levels of points 23 on both trusses were taken. Usually such levels had been taken in the early morning when the effect of temperature was least noticeable, but on this particular occasion (Tuesday, August 25th) the comparison was made in the afternoon of an exceptionally warm day toward the end of a continuous hot weather period covering between one and two weeks. The observation showed the south truss  $2\frac{1}{2}$  inches lower than the north, measured at the intersection of centre lines, on top of the top chord cover plates at point 23. On Sunday, August 30th, the Resident Engineer and the Author went out together to check and study this situation. Saturday had been much cooler, rain had fallen in the night, and Sunday was a drizzling raw day, and the observation taken about 3.30 p.m. showed a difference of  $1\frac{1}{2}$  inch. The weather continued to be damp and misty until the next morning, when the figure had diminished to about  $1\frac{3}{8}$  inch. The weather then became sunny and hot during the middle of the day such that  $1\frac{1}{2}$  inch was recorded towards 5 p.m. on Monday, 31st August. On the 17th October when the east end had reached the centre of the span, at  $53^{\circ}$  F., the difference was found to be a bare  $1\frac{3}{8}$  inch. Incidentally, on the east end, which had been erected during the temperate and even weather of October, the corresponding difference was  $\frac{3}{8}$  inch, and in the same direction, but the Author cannot claim that this was entirely due to climatic effects. On 24th October, after the centre pins had been inserted and the span swung as a three-hinged arch, the difference at the centre had changed to  $1\frac{3}{8}$  inch, the north side being the lower. The difference of  $1\frac{1}{2}$  inch, which might be termed the final deviation from cross level on the west end itself, was doubtless made up of various items, including shop inaccuracies and initial inequality of elevation over the skewbacks, as well as the temperature effects. The pins at point 1 were not absolutely level, the north-west corner being  $\frac{1}{8}$  inch lower than the south-west. The two east-end corners were exactly level one with the other, and both with the south-west corner.

Referring somewhat at length to these observations, it should be stated that on 17th October, 1914, when the observations referred to above were made, the end verticals were not absolutely parallel. The south-west end post leaned backwards  $2\frac{5}{16}$  inches, while the north-west tilted  $2\frac{7}{16}$  inches. From these two figures together with the shop survey and the difference in elevation of  $\frac{1}{8}$  inch at point 0, an attempt was made to approach theoretically the  $1\frac{3}{8}$  inch, and by so doing to discover, perhaps, some approximation to the real effects of temperature. The difference at points 0 is not susceptible of a very satisfactory explanation. The points were originally set level, and not until the 24th October was the

difference actually realised, Fig. 6a. An observation was made from the north side early in August when the two points 0 (north-west, south-west) were checked

Figs. 6.



DISTORTION EXAGGERATED TO 48 TIMES.

for position in a plane normal to the longitudinal centre line and found correct. At this time the vertical height from 00 to 1 on the north side only, was measured

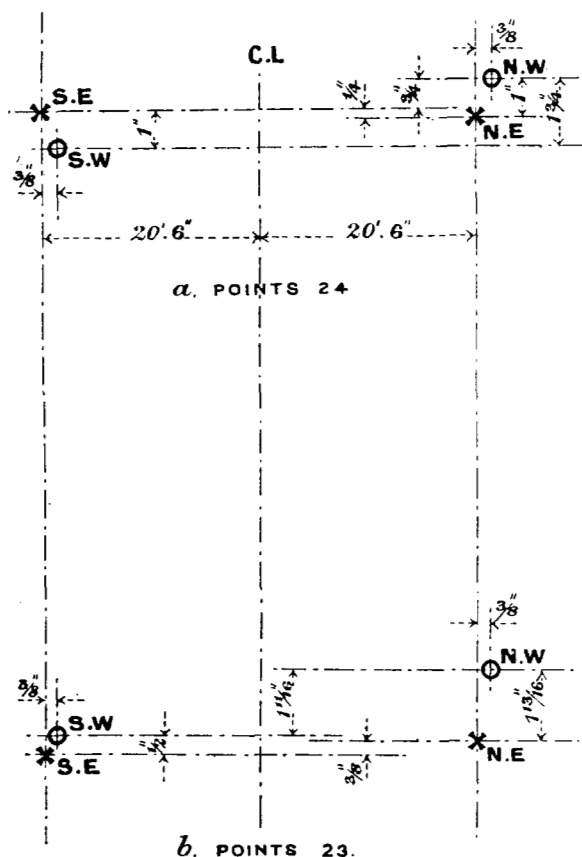
and found to be nearly  $1\frac{1}{4}$  inch short of the expected figure. Of this amount  $\frac{7}{8}$  inch was accounted for by another error which is mentioned on p. 287. The length of the end verticals was exactly the same on the shop survey, so that, as the observations of 24th October indicated, the point 1 on the north-west was certainly  $\frac{5}{16}$  inch lower than the points 1 on the other three corners. A study of the shop survey results, in Table II, shows a relative super-elevation of point 24 north-west above 24 south-west, of  $2\frac{7}{16}$  inches, but in the field on 17th October this would be decreased by the  $\frac{5}{16}$  inch just alluded to and increased by  $\frac{9}{16}$  inch which is the effect at 24 of the excess backward tilt of  $\frac{1}{8}$  inch in the end post. The result of these modifications is to give  $2\frac{1}{4}$  inch as the theoretical super-elevation of 24 north-west above 24 south-west which compared with the  $1\frac{3}{4}$  inch actually observed. It is to be noted that uniform temperature conditions would not affect this comparison, and the discrepancy of  $\frac{1}{8}$  inch must represent the result of unequal temperature conditions together with other field inequalities such as rivet slip. It is, perhaps, rather a large amount, but the effect of the great changes in temperature, both as between day and day, truss and truss, chord and chord, and arch and anchorage, is somewhat difficult to gauge. On the eastern half where the conditions were very much more uniform, a distinct improvement can be shown.

Points 0 were perfectly level in this case, but the north side vertical 0-1 was again tilted backwards  $\frac{1}{8}$  inch more than the south,  $1\frac{1}{8}$  inch against  $1\frac{3}{8}$  inch on 17th October at 53° F. The effect of this variation in tilt was to raise the north point 24 by  $\frac{3}{16}$  inch. Again referring to the shop survey, Table II, the difference in upward set (cock-up) brought the south-east 24  $\frac{1}{8}$  inch higher than the north-east. Combining the two effects the south-east 24 should theoretically be  $\frac{5}{8}$  above the north-east 24. This, it will be seen, is the observed condition, which suggests that the other field effects and inaccuracies mutually balanced. It may be said here that the workmanship on the east end trusses was probably more accurate than on the west, due to experience and increased care. Further relative elevations were obtained on 17th October, after the last bottom chords on the east end were entered, namely, between the points 24 on all four trusses. Comparing the two trusses of one end, these points bore, as near as could be read, the relations as shown in Fig. 7 a, the simultaneous elevation of points 23 being shown in Fig. 7 b.

The climatic conditions may be quoted as very probably responsible for the slight lateral bend in the west half, but the amount was indeed very small and not at all disconcerting. In examining the sketch it must be borne in mind that the situation was not identical on the two arms when these relationships existed. The following differences will be noted—the tilt of the end verticals 0-1, upward set of the truss due to camber, and the erection stresses. Taking the last cause first, it will be understood that the west trusses carried no floor stringers, no service tracks, and no traveller, while on the east half the traveller and its three panels of track were in place on panels 17-19-21-23, together with the necessary material track and other erection equipment. The greater erection stresses produced in the east half would give those trusses a relative downward deflection, and the smaller backward tilt would operate in the same direction, while the greater upward set would work to the opposite end. Saturday the 17th was cool and wet, and on Sunday the temperature fell to 53°, the atmosphere remaining very moist. Measurements taken on the south truss showed  $1\frac{3}{8}$  inch minimum opening between adjacent ends of chord members at point 24, which same dimension had been read as  $1\frac{1}{4}$  inch on the previous morning at

slightly over  $60^{\circ}$  F. The next morning was again cool and windy. The elevations were observed and the situation shown in the sketch was almost exactly obtained, the only difference being in centre line deviations. The records of this date give the west end half an inch north of true line, and the east end as  $\frac{1}{8}$  inch south of the correct centre making a relative spread of  $\frac{3}{8}$  inch. The centre pin on the south side was entered on this morning, and the necessary drifting

Fig. 7.



together of the trusses was very easily accomplished by the use of a small jack at point 24 working between the top flange of the chord on the low side, and the bottom flange of the chord on the high side. The following day, the 20th, was a better working day, and the north-side pin was entered during the afternoon. The forcing together vertically of points 24 south-east and 24 south-west on the preceding day had increased the distance between 24 north-west and 24 north-east, as the sketch will make clear,

and the main jack in the toggle system at the north-west end was let down sufficiently to bring the two half pin-holes close enough to enter the pin. The actual amount by which the north-west jack was lowered is recorded as  $1\frac{1}{8}$  inch, which would let 24 north-west down about  $1\frac{1}{2}$  inch. Both pins being now in place, the two trusses were drawn into line by means of jacks and turnbuckles, the nuts of the pins being screwed tight as the correct positions were obtained. The last panel of the lateral bracing on the east side was then placed and rigidly bolted up. At this stage the pins were, of course, not actually in bearing on the bored surfaces of the holes, and were not transmitting any horizontal load, the two halves of the bridge still being cantilevers. Actually there was just  $\frac{1}{4}$  inch clear between the adjacent edges of chords meeting on either pin. Levels were now taken of the two pins and the agreement was practically absolute, the south being a trifle high but not more than  $\frac{1}{8}$  inch at the existing temperature. The main toggle was then slacked down on all four corners and the span gradually swung as a three-hinged arch. Elevation and line were again taken, the former showing the south side  $1\frac{1}{8}$  inch higher than the north, and the latter showing all the points 23 as being nearly  $\frac{1}{2}$  inch north of the true axis. A few days later, on 24th October, a very interesting and informative set of levels was obtained, which has already been referred to, and which form the basis of Fig. 6 a.

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