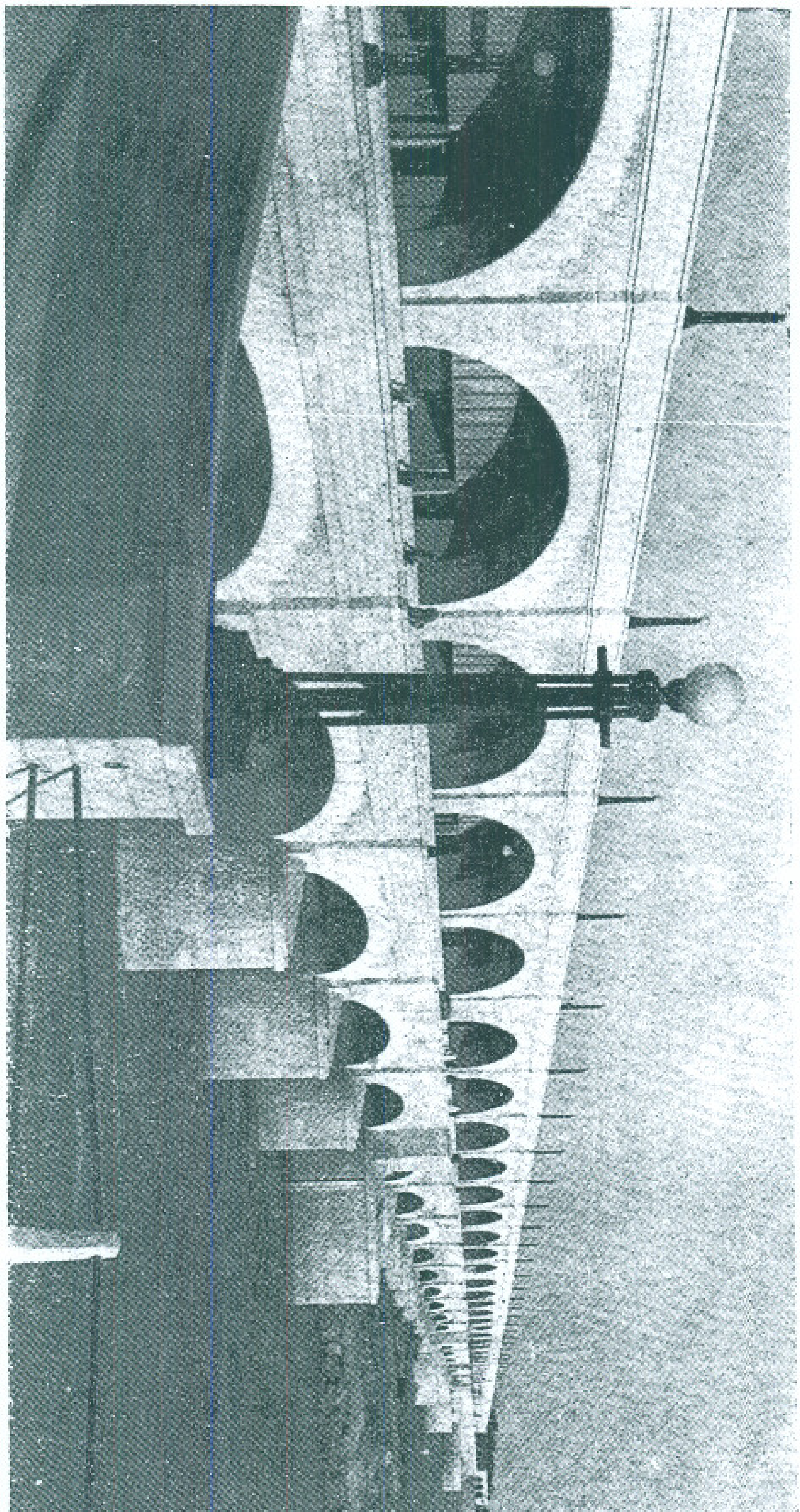


SUKKUR BARRAGE

and its offtaking irrigation channels hold a strategic position in the rural economy of southern West Pakistan, and prosperity and progress of this tract is almost entirely dependent upon this project.

The barrage comprises 66 bays, each of 60′ clear span, and is divided into 3 sections : The right undersluices, the weir portion, and the left undersluices. The right and the left undersluices have 5 and 7 bays respectively and are separated from the main weir by the right and the left divide wall. The middle weir portion of 54 spans is divided into 6 compartments of 9 spans each. These compartments are separated from one another and from the scouring undersluices by 20' wide abutment piers. Ordinary piers between the spans are 10′ wide.

The piers support two separate bridges. The one on the upstream side of the piers is the Gate Bridge or the Regulating Bridge and is at a higher level than the Road Bridge on the downstream side. The gate bridge consists of two separate cement concrete arches, one 8′-3″ wide on the upstream, and the other 5′-3″ wide on the downstream, with a gap of 13′ clear between them. The gates and the counter weights of the barrage are suspended in the gap by wire ropes. The operating gear etc., for the gates is mounted on this bridge which is not suitable for use by the public or for traffic except that of the operational staff, the trash crane and a light weight trolly. The arches have a springing level of R. L. 219.0 and a rise of 15′. The top of the bridge is at R. L. 238.5. The low level bridge on the downstream side of the piers, *i.e.*, the Road Bridge is also supported on reinforced cement concrete arches. These arches are 25′-3″ in width and their springing level is at R. L. 201.0, slightly above the highest estimated flood level of R. L. 200.1. The arches have a rise of 10' and the top of the road-way is at R. L. 215.25. Originally this bridge had a 16′ wide road-way for vehicular traffic with two side walks each 3′-6″ wide. One side walk was subsequently removed to widen the road-way to 18′-6″ as it was too narrow for the heavy traffic developing on the bridge. Immediately after the coming into the operation of the barrage, serious silt trouble was experienced on the right bank canals. After model experiments at Poona, it was considered necessary to construct certain river training works which resulted in closure of certain weir bays. This reduced the flood capacity of the barrage from 15 lac cusecs to about 9 lac cusecs.

In the first design of the barrage prepared in 1919, stone masonry voussoir arches were proposed for the road bridge as well as the gate bridge for the river and the canals. Clear span for the river arches was 60 feet and for the canal regulator arches 25 feet. The barrage road bridge arches were proposed to be elliptical, with 10 feet rise, springing at R. L. 201.0 and with thickness at the crown as 2′-9″ and at the springing 4′-6″. The voussoirs were to be of single stones and full 25′-3″ length. The barrage regulating bridge arches were also elliptical, with a rise of 15′, springing at R. L. 219.0, thickness at the crown as 2′-6″ and at the springings 4′-0″. Two series of separate stone arches 8′ wide on the upstream and 5′ wide on the downstream with a clear space between arches of 16 feet for gates and counter weights were envisaged. The canal regulator road bridge arches were elliptical, 8′-4″ rise, 25′-3″ wide, thickness at the crown 1′-6″ and at springing 2′-6″. Regulator gate bridge arches were two separate semi-circular voussoir masonry arches of 25 feet span each, springing at R. L. 202.0, thickness at the crown 1′-6″ and at springing 2′-0″. The upstream and downstream arches were 4′-3″ and 4′-0″ wide respectively.

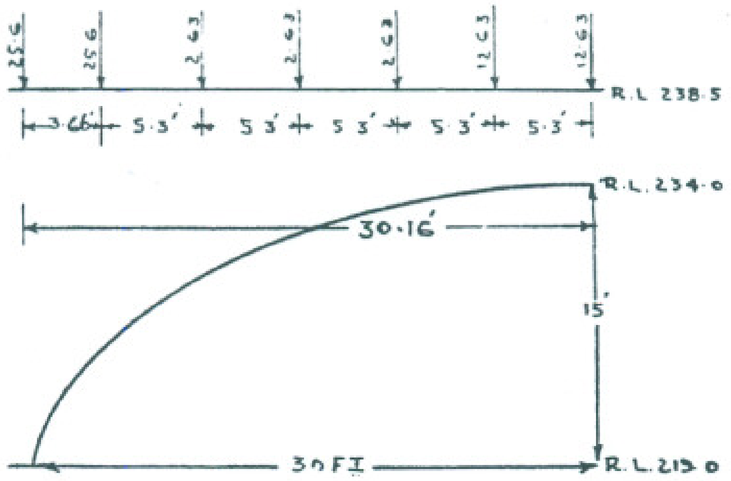
The design for the river arches of 60′ spans was changed in 1928 just before their actual construction, and reinforced cement concerte replaced stone as material for construction. The design of arches for the head-regulators of the seven canals was however not changed as their span of 25′ was rather short, and they were constructed in stone masonry as masonry voussoir arches. For the river arches of longer span, reinforced concrete was considered stronger, more consistent, more economical simpler and quicker in construction. As a result of fresh laboratory tests, stone was considered unsuitable and weak for such an important structure. Originally compression tests for stone were carried out on 2″ size cubes. In 1928 when 6″ size cubes were tested, the average crushing strength (2464 *psi*) was indicated to be less than half of that for 2″ cubes (4963 *psi*). For larger specimens a still smaller strength was feared due to stratified nature of local limestone rock and possibility of weaker sandwiched layers. It was also considered difficult that the heavily stratified local rock would yield well dressed voussoir stones 25′ long, 4′ high and about 2′ thick. Reinforced concrete was considered to be easy to manufacture and handle, and work could be continued even during the flood days.

Steel arch centering to stone arch profile had been manufactured when decision to change over to concrete was taken. Consequently the intrados arch profile for the road bridge as well as the gate bridge was adopted without any change to avoid delay in construction. The extrados profiles were made into 5 centered, segmental arches. The modified thickness at crown was taken as 2′-0″ for the road bridge and 2′-6″ for each of the gate bridge arches. The arches were given stone voussoir facing on the sides visible from the river or the road way so as to get a stone architectural effect. The modified thicknesses very nearly conformed to the original thinking for the stone voussoir arches.

In the 1919 project, the following loads were accepted over the road bridge and the gate bridges :

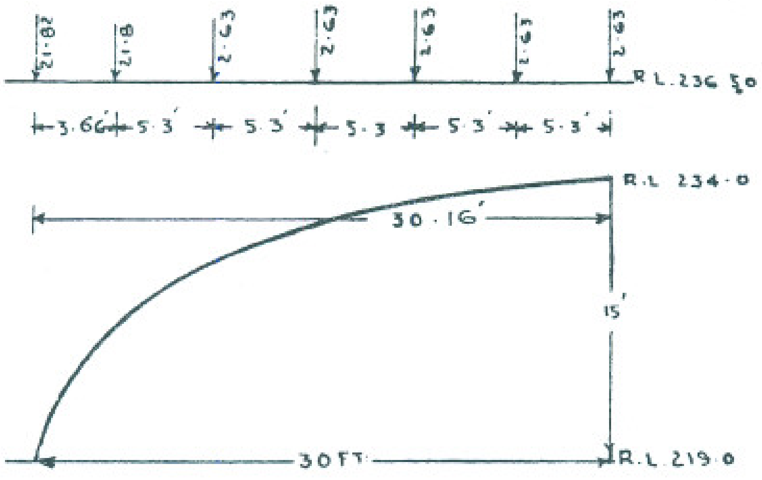
(i) *Barrage Road Bridge* : A live load of 100 lbs. per sq. ft. with an impact factor of 13% was taken over the road-way and the footpaths, in addition to the computed dead load at any section. To this load was added (without any deduction) the weight of a steam roller weighing 15 tons, with 9 tons on the rear axle at the centre of the bridge and 6 tons on the front axle 11′ from the crown. No impact was allowed for the steam roller load which was assumed to be distributed over the full 25′ width of the bridge arch. This system of live loads gave moments and thrusts equivalent to those generated by an evenly distributed load of 200 lbs. per sq. ft. over the whole area. The arch was analysed graphically for dead and live loads placed symmetrically about the crown, and with thrust at the crown as horizontal. It yielded a maximum compressive stress in masonry of 316 *psi* at crown and 311 *psi* at joint of rupture.

(ii) *Barrage gate bridge* : In addition to dead weight, a live foot traffic load of 45 lbs. per sq. ft. with 13% impact factor was considered over the full width of bridge floor between parapets. The arches also carried reactions from cross girders supporting the gates and counter weights. The upstream arch also catered for the full weight of a 3-ton travelling crane with a laden weight of 15 tons. Impact of 33% over the total crane weight was considered. The following were the design load diagrams for the two gate bridge arches, in addition to the dead weight of the structure :



UPSTREAM G.B. ARCH LOADS

FIG. 1



DOWNSTREAM G.B. ARCH LOADS

FIG. 2

Under this combined system of loading, maximum compressive stresses in upstream arch was 315 *psi* at crown and 313 *psi* at joint of rupture. The corresponding values for the downstream arch were 316 *psi* and 313 *psi*.

During review of the design in 1928, the project loads were considered in-adequate. Live load over the road bridge was changed to a succession of 16 ton lorries per 10 feet traffic lane as in figure 3.

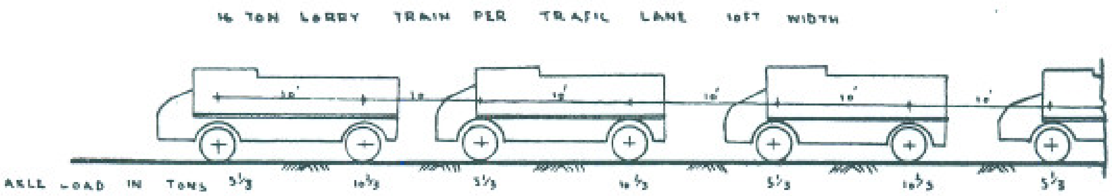


FIG. 3

The lorry axles were uniformly spaced at 10 feet, with 2/3rd load on the rear axle and the remaining 1/3rd on the front axle. Impact factor was taken as as 50%. Crown load of 100 lbs. per s. ft. was not changed, but it was not considered acting under the lorries. Point loads were dispersed at 45° up to the extrados crown level. The dispersion was however taken only across the arch to get point load per foot width, but was not taken along the arch. The worst condition of loading for moments and stresses was assumed as when the arch was fully loaded, with the heavier axle at the crown.

For the gate bridge arches, various alternative loads likely to be experienced during operation or during construction were analysed. The worst loading accepted for design was a static load of 40-ton gate standing on 4 bearers (total load 44 tons) and a rolling load of a crane carrying 7 tons passing over the arches. The crane laden weight was 34 tons and its rolling load came to

22 tons on the upstream and 12 tons on the downstream arch. The point loads under the bearers were taken as in Fig. 4.

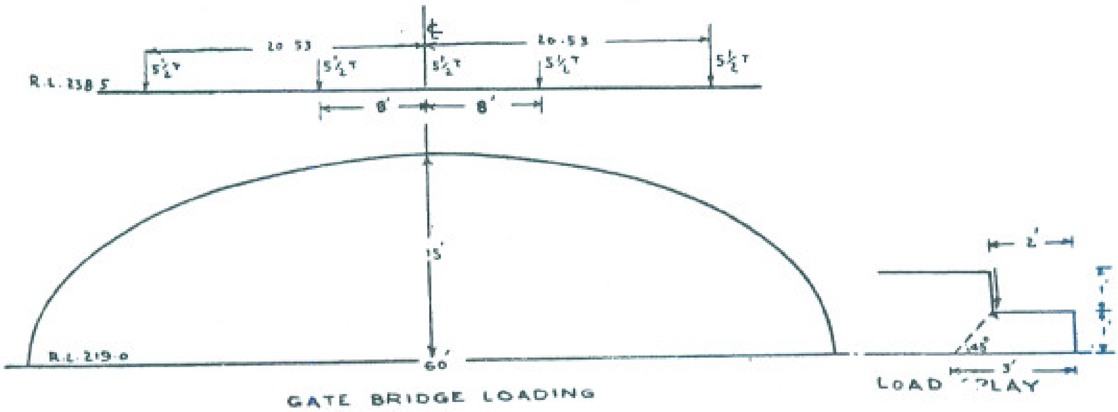


FIG. 4

The angle of splay was 45°. Crane load was presumed to be carried on a bogie with closely spaced wheels over a length of 13′-6″. Loading on the upstream arch was heavier than on the downstream arch. Design calculations were run for the heavier loaded arch only and identical concrete section and reinforcement steel was provided in the other arch for uniformity in construction.

In addition to live and dead loads, temperature effect was also considered in the design of concrete arches. At Sukkur, the widest temperature variations recorded were 17°F and 175°F in the sun. Reinforcement steel in concrete arches was provided with a cover of 22″ and at the level of reinforcement this temperature variation was expected to be much lower. A total variation of ±30°F was considered in design and was considered adequate on the assumption that the arches would be constructed during mild weather in September and October or March and April. Self-recording thermometer fixed in the arches during construction showed a maximum of 115°F in June and July and a minimum of 63°F in January at the level of reinforcement, indicating a total variation of 52°F.

Summation equations for a symmetrically loaded encastre arch were used in the design of concrete arches to compute crown moments, thrust, and shear for loads and temperature effect. Live loads were placed symmetrically about the crown for worst effect and one half of the arch was considered for analysis. Certain arbitrary thickness of section and area of steel was presumed and checked for stresses for the computed moments. No alternatives for shape and size were examined for an economical design of the arch.

Work on construction of concrete arches was taken up late in 1928 and was completed in 1931. The work of concreting was continued in all seasons.

Specifications for Reinforced Cement Concrete Arches

Cement concrete in arches consisted of 1 part of cement, 4 parts of crushed local lime stone and $1\frac{1}{2}$ parts of over size stone metal. No sand was used. Batching was done by volume and mixing was done in concrete mixers. Concrete was placed in position either by tip wagons or as head load, after which it was tamped by iron bars. No vibrators were used. Wet curing was done for one month. No tests were carried out in the laboratory or in the field to find the strength of the concrete. No slump test or compression test was performed or specified.

Fat lime concrete filling in the haunches and over the arches consisted of 4 parts of stone metal, 2 parts of sand, and 1 part of fat lime, mixed by volume.

Need for Structural Analysis

Cracks were first reported in the Barrage piers in 1949. As Sukkur Barrage is an extremely important structure, reports of damage created considerable alarm. The damage was inspected by a large number of experts, including Sir Arnold Musto, the original designer of the structure and Mr. Mohsin Ali. The piers were repaired by grouting under pressure, and by providing reinforced cement concrete protective covers to the upstream noses. Cracks were first reported in the cement concrete arches in December, 1950. It was decided to repair these arches by guniting through Cementation Company, a British Firm specializing in the trade. The work of guniting the soffits of all the road and gate bridge river arches was assigned to this firm and was completed during 1953-56. Soon after these repairs, cracks, were noticed in a number of repaired arches in March, 1956. The Cementation Company was requested for advice, but it made no serious attempt and advanced reasons which were not convincing. The high level Technical Committee constituted in 1964 felt that after the unhappy experience of 1950-56, a thorough thrashing of the problem was essential. Structural safety of the arches with their visible state of deterioration needed immediate assessment. This called for extensive field and laboratory investigations to find out the extent and causes of damage, and detailed structural analysis to evaluate the factors of safety. Field and laboratory investigations were extended to determine the degree of separation of cover concrete in various arches, the extent of rusting of bars, the depth of cover concrete, strength and properties of concrete and old gunite, the properties of construction materials, mapping of all cracks in the arches and measurement of temperatures in the body of the arch. These investigations do not form a subject of the present paper. The structural analysis of the road bridge and gate bridge arches was undertaken for different loadings and for different assumptions regarding the soundness of structure. The design assumptions originally made were checked, and shortfalls in original design procedures were taken care of. Various check calculations were run as suggested by field investigations. The results obtained are of considerable interest and are being presented in this paper.

Analysis of a Fixed Arch

A true arch is defined as a structure which depends in a considerable degree for its ability to support applied vertical loads on the development of horizontal reaction components acting towards the centre of the arch span at the two end supports. Structurally arches may be classified as three hinged, tow hinged or hingless. Three hinged arch is statically determinate, while the two hinged arch has one redundant factor. The hingeless arch, also known as fixed or encastre arch has a degree of redundancy of three, and three additional equations are required, apart from the usual three equations obtained from moments and reactions, to solve the structure. Reinforced concrete ribs are almost always hingeless. Fixed masonary arches may be further divided into thick or thin arches. Only thin arches are being considered for the present. A thin arch may be further divided into a short span or a long span arch. In the long span arch, elastic deflection of the arch axis introduces significant stresses and has to be taken into account. Short span arch may be analysed as an elastic structure, ignoring deflection and neglecting the effects of the change in shape of the arch axis due to elastic strains. The arch may be symmetrical or unsymmetrical. A symmetrical arch has its half spans right and left of the crown as identical, and has the two supports at the same level. Our interest at present is confined to a thin, elastic, symmetrical, hingeless short span arch.

The shape of an arch is not the shape of its intrados or extrados, but the shape of its profile, or the neutral axis drawn through the centroid of its cross sections. The span of the arch is the span of the profile. Concrete arches at Sukkur have their intrados as true ellipse but the extrados is a multi-centred curve. The neutral axis or the arch profile does not follow a curve which can be represented by a simple mathematical expression. It is thus not true to call the Sukkur Barrage arches as elliptical arches.

A reinforced concrete arch differs from a masonary arch in that it is designed to resist any tension developing at the faces, and the line of thrust is not confined to the middle third. In a voussoirs arch, the arch ring is invariably in compression and no tension is indicated at any point of any section. The difference may be compared to that of a reinforced concrete retaining wall and a gravity stone masonry wall. The line of thrust is no longer the linear arch or funicular polygon due to external live and dead loads. Secondary forces of temperature and shrinkage come into play due to continuity of the structure and have a profound modifying influence on design. For the same loads, at any section the position of line of thrust and the value of thrust will vary with rise or fall of temperature. It is therefore not the practice to plot thrust lines for such arches. Much thinner sections are possible with reinforced concrete for the same span. Secondary moments have to be carefully balanced against the load moments to design the best section and consideration is not confined to the line of pressure due to external loads.

Individual arch sections are designed like eccentrically loaded short columns after envelop of maxima positive and negative moments has been determined. The envelop is determined by the worst possible combination of moments due to dead load, live loads, concrete shrinkage, rise and fall of temperature and rib-shortening stresses. The arch is generally symmetrically reinforced, with equal top and bottom steel. Calculations of arch stresses are generally long and laborious and in practical design extensive use is made of tables and graphs covering a number of geometrical shapes. Arch geometry is generally selected such as can be represented by a simple second or third degree equation, making simple integration possible. Choosing of any complicated random shape would call for a mass of laborious figure work during the structural analysis.

Sukkur concrete arches do not conform to any regular shape. Consequently for each check, long calculations are required by using the summation equations. The job becomes further complicated when different degrees of soundness of reinforcement is accepted, and the arch section is no longer symmetrically reinforced.

Theory of a thin Elastic Arch

It is not the intention here to discuss the fundamental theoretical equations for a thin elastic arch. These equations may be found in any standard text book. Each author has his own liking for a particular shape or procedure. The designer generally picks up a certain set of equations and applied them to his design, but unless he is firmly grounded in theory, he is likely to lose sight of the basic assumptions and end up with an erroneous design. This unfortunately has happened in 1927-28 at the time of the original design of the Sukkur Barrage concrete arches. As the subject may be new to many of us, a very brief account is given below for a better understanding of the subsequent discussion.

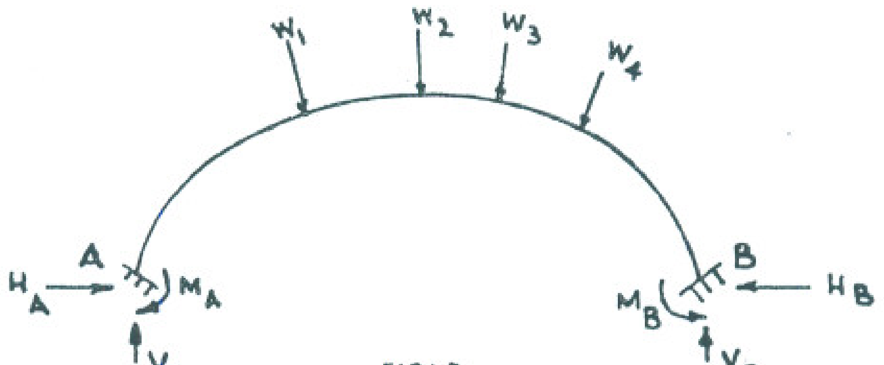


FIG: 5

Fig. 5 shows a thin elastic arch, carrying any system of loads and fixed at the ends. There are 6 unknowns :

(*i*) Horizontal thrusts H_A and H_B at two ends.

(*ii*) Fixed end Moments M_A and M_B

(*iii*) Vertical reactions V_A and V_B

Six independent equations are required to solve these six un-knowns. Three standard equations of equilibrium are obtained by resolving the forces in two directions normal to each other, and by taking moments about a point. Three more equations essentially required can be derived either from Strain principle or from Elastic Strain principle.

In case of a rigid elstic arch there is no rotation and translation of one end with respect to the other. Applying the Elastic Strain principle, the three summation equations for angular movement, vertical movement and horizontal movement will be equal to 0. These three equations can be written as :—

$$\int \frac{M}{EI}. \ ds=0 \quad \text{(no rotation)}$$

$$\int \frac{Mx}{EI}. \ ds=0 \quad \text{(no vertical movement)}$$

$$\int \frac{My}{EI}. \ ds=0 \quad \text{(no horizontal movement)}$$

These three equations are the same as obtained by Strain Energy method, and can be solved by direct integration or by graphical methods.

Sukkur Barrage arches follow a complex shape and cannot be analysed by using Integral equations. A solution can be found by using the three basic summation equations :—

$$\Sigma \frac{M}{EI} \cdot \delta s \quad =0 \qquad \text{— (1) for no end to end rotation)}$$

$$\Sigma \frac{Mx}{EI} \cdot \delta s \quad =0 \qquad \text{— (2) (for no end to end vertical movement)}$$

$$\Sigma \frac{My}{EI} \cdot \delta s \quad =0 \qquad \text{— (3) (for no end to end horizontal movement)}$$

Simplifying assumptions

The equations can be simplified by considering E as constant. If Moment of Inertia I of arch Sections is not constant, the arch may be divided into such suitable number of 'n' segments that δs, the length of any segment along the profile curve bears a constant ratio to the average I of the Section, and $\frac{\delta s}{I}$ = Constant. In that case E, and $\frac{\delta s}{I}$ can both be omitted from the summation equations No. 1 to 3 which reduce to

$$\Sigma M = 0 \qquad (4)$$

$$\Sigma M.x = 0 \qquad (5)$$

$$\Sigma M.y = 0 \qquad (6)$$

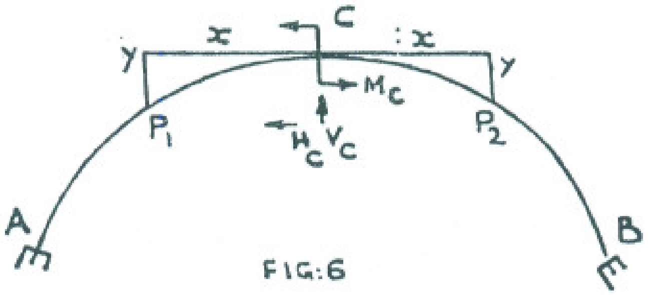


FIG: 6

Refer Fig. 6, take the origin at Crown C (0, 0), y is measured positive downwards, x is positive to the right as well as the left. Each half of the arch may be considered separately. Under load, the rotation or horizontal movement of C with respect to the two ends A and B must be equal and opposite, while the vertical movement should be equal and alike. This would yield :

$$\sum_C^A M = -\sum_C^B M \qquad -(7) \qquad \text{(rotation).}$$

$$\sum_C^A M.x = \sum_C^B M.x \qquad -(8) \qquad \text{(vertical movement).}$$

$$\sum_C^A M.y = -\sum_C^B M.y \qquad -(9) \qquad \text{(horizontal movement)}$$

For any point P_1 (x, y) to left of C, let M_L be the moment of all external loads between P_1 and C. Similarly for P_2 (x, y) on right, let M_R be the value. Also assume M_c, H_c, V_c as the values of moment, horizontal thrust and vertical reaction for equilibrium if arch is cut at the crown. This will yield another three equations :

$$\sum_{C}^{A} M \quad = \quad n.\, M_c + H_c \Sigma y + V_c \Sigma x - \Sigma M_L \qquad — (10)$$

$$\sum_{C}^{B} M \quad = \quad n.\, M_c + H_c \Sigma y - V_c \Sigma x - \Sigma M_R \qquad — (11)$$

$$2n.M_c + 2H_c \Sigma y - \Sigma M_L \quad - \Sigma M_R = 0 \qquad — (12)$$

Solution of equations 7 to 12 will yield the three redundant terms M_c, H_c and V_c. However it has to be clearly remembered that Summation is strictly confined to n terms for each half of the arch and to the n segments into which each half has been so divided as to give $\frac{\delta s}{I}$ constant. The resultant solution is:

$$H_c = \frac{n\Sigma M_R\, y + n\Sigma M_L\, y - \Sigma M_R\, \Sigma y - \Sigma M_L\, \Sigma y}{2\,[n\,\Sigma\, y^2 - (\Sigma\, y)^2]} \qquad — (13)$$

$$M_c = \frac{\Sigma M_R + \Sigma M_L - 2\, H_c \Sigma\, y}{2n} \qquad — (14)$$

$$V_c = \frac{\Sigma M_L\, x - \Sigma M_R\, x}{2\Sigma x^2} \qquad — (15)$$

Having computed M_c, H_c, and V_c, Moment M value at any point on the arch can be readily calculated. For any point P in P_1 position lying anywhere between A and C :

$$M = M_c + H_c\, y + V_c x - M_L \qquad — (16)$$

And for P in P_2 position right of C,

$$M = M_c + H_c\, y - V_c x - M_R \qquad — (17)$$

If the arch carries a loading which is symmetrical about the central axis through the crown, further simplification of equations 13 to 17 is obtained. For such loading, $M_L = M_R$ for conjugate points. The arch may then be analysed for one half only, and these equations reduce to :

$$H_c = \frac{n\Sigma M_L\, y - \Sigma M_L\, \Sigma\, y}{n\Sigma\, y^2 - (\Sigma\, y)^2} \qquad — (18)$$

$$M_c = \frac{\Sigma M_L - H_C\, \Sigma\, y}{n} \qquad — (19)$$

$$V_c = 0 \qquad — (20)$$

$$M = M_c + H_c\, y - M_L \qquad — (21)$$

Temperature, Shrinkage and Rib-shortening Effect

From the equations already discussed, suitable equations to yield temperature and shrinkage moments can be readily derived. For temperature,

$$H_{c_t} = \frac{I}{\delta s} \cdot \frac{n.E.\phi.t.L}{2[n\Sigma y^2 - (\Sigma y)^2]} \qquad — (22)$$

$$M_{c_t} = -\frac{H_{c_t} . \Sigma y}{n} \qquad — (23)$$

$$M \quad = M_c + H_c . y \qquad — (24)$$

where ϕ = Coefficient of expansion of concrete $=6 \times 10^{-6}$ per 1°F
E = Modules of Elasticity of concrete $=2 \times 10^6$ *psi.*
t = Variation of temperature. (+ for rise, — for fall).
L = Span of the arch profile.

For shrinkage, it is usual to presume an effect equivalent to 15°F fall. Shrinkage values can therefore be found by taking $t=15°$ in equations 22 to 24.

For rib-shortening, the elastic shortening of the arch under load may be viewed as a fall of temperature effect. Thrust and the resultant compressive stress f_c varies along the arch, and an average value may be accepted for computing elastic strain $e_c = \frac{f_c}{E_c}$.

In equation No. 22, e_c would replace temperature strain $\phi\ t$.

$$\therefore \quad \phi t = e_c \quad = \frac{f_c}{E_c} \quad \text{or} \quad f_c \quad = \phi . E_c . t$$

If $t=1°$F, $f_c = (6 \times 10^{-6}) \times (2 \times 10^6) \times 1 = 12$ *psi.*

Thus for every 12 *psi* compressive stress on arch ring, the equivalent strain effect can be considered equivalent to 1°F fall of temperature. This is a convenient conversion factor for computing Rib-Shortening M and T values.

It may be seen that in all the three cases, M and H values are directly proportional to I, the moment of inertia of the arch section which in turn is proportional to d^3, where d is depth of the section. Consequently secondary stresses due to temperature and shrinkage sharply mount with thickening of the arch, and have to be balanced carefully against the load values.

In the foregoing discussion great stress has been laid on the accepted simplifying procedures and various design assumptions to ensure that the following observations and check calculations are appreciated properly.

After the moment and thrust values have been computed for various arch sections, the design procedure is comparatively simple. Individual sections are designed like eccentrically loaded R. C. Columns. If sections are unsymmetrically reinforced, the designer cannot make recourse to standard tables and has to proceed from first principles. Different practices exist with respect to loadings, load spread, impact, temperature, initial setting shrinkage and load placement for maxima stresses. The design has first been checked for design conditions to compare with the original computed values. Available factors of safety have subsequently been computed for different degrees of assumed damage.

Original Design Procedure

A typed copy of design calculations for concrete arches at Sukkur could be located with great difficulty. These calculations show a marked difference in the degree of thoroughness as compared to the design calculations for other parts of the barrage structure in the printed project volumes. Probably the job has been done in a hurry after the decision was taken at a late stage to switch over to reinforced cement concrete as material of construction for the river arches. Concrete arch geometry was kept pretty close to that of original stone arch, and the design at best, shows a rough project approach. No study was made to investigate the relative influence of dead and live loads and secondary forces on the design. Alternative shapes and profiles for economy were not considered.

Plate I and II show the reinforcement details for the Road Bridge Arch and the Gate Bridge Arch.

Design Check procedure

In the first instance live loads have been taken the same as in original design calculations. As much heavier traffic loads are being experienced on highway bridges now, the Road Bridge has been checked further for present-day requirements. For the Regulating Bridge only erection loads had been considered in design. During the check, worst operational loads have also been taken into account. The regulating bridge has also been analysed for emergency loadings.

In the initial design an impact factor of 50% had been adopted according to the British Ministry of Transport practice at that time. While exercise has been done with this impact factor also, fresh valves have been worked out for a realistic value of impact in the light of the latest research on the subject. All point loads have been dispersed at 45° to the vertical in both X and Y plans. As all design calculations are with respect to the elastic axis of the arch, the splay lines have been carried up to this level. Load intensities over the area of splay are considered as uniform. Where two load splays intersect, the loads are evenly distributed within the extreme limits.

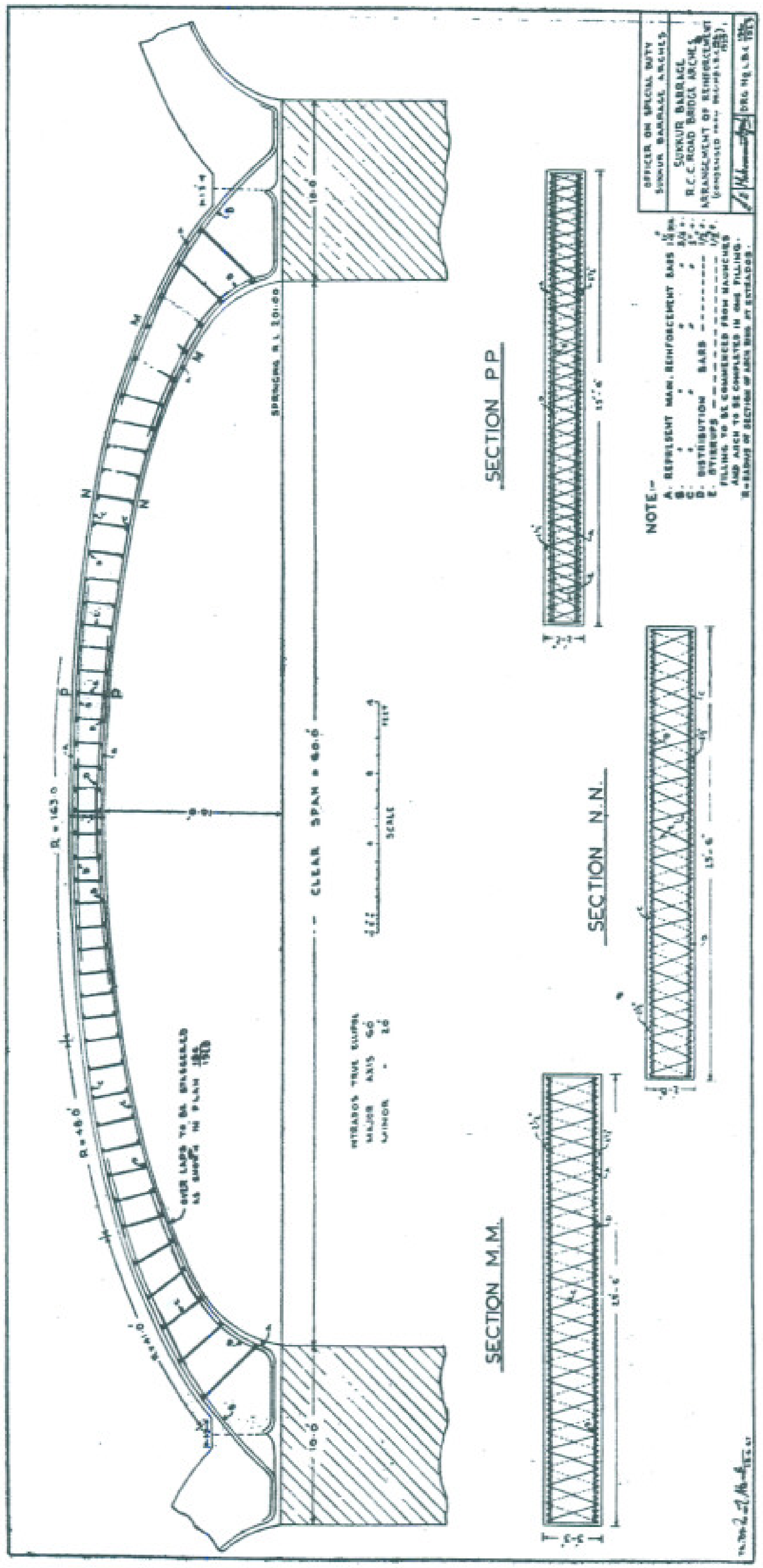

SECTION M.M.
SECTION N.N.
SECTION P.P.
CLEAR SPAN = 60'-0"
SCALE
NOTE:-
SUKKUR BARRAGE
R.C.C. ROAD BRIDGE ARCHES

Plate I

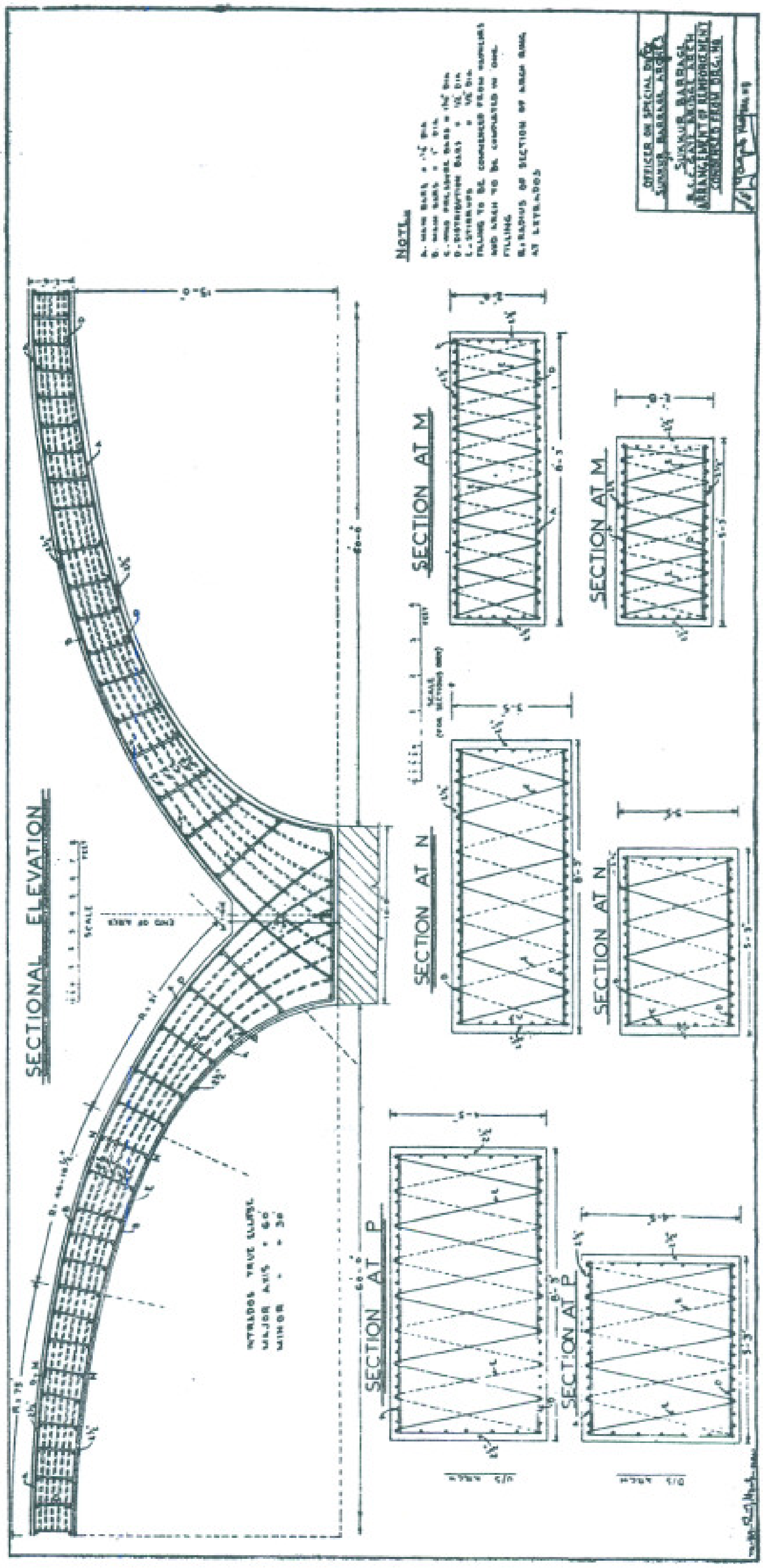

SECTIONAL ELEVATION
SECTION AT P
SECTION AT P
SECTION AT N
SECTION AT N
SECTION AT M
SECTION AT M
NOTE

Plate II

Moment and Thrust at any section of the arch is not maximum at the same time. Moments cause tension on one face and maxima moments are accepted in design practice as the basis of working out stresses. The value of corresponding thrust at the section which may not be the maxima value at the section is used. The arches are quite stiff and resistant to local irregular deflections. After splay through thick decking it may not be inaccurate to even out small differences in spot loadings. Live load system normally yields a fairly uniformly distributed load pattern.

In an arch, maxima live load moments do not occur like a beam when the whole span is loaded. The maxima positive or negative conditions at various sections, except the crown, occur when $\frac{4}{10}$ to $\frac{6}{10}$ of the span from one end is under load. At the crown, maxima conditions occur generally when the middle $\frac{3}{10}$ to $\frac{4}{10}$ of the arch is under load, but this section does not give the absolute maxima moment value for design. It is fairly accurate to assume for design that maxima $\pm M$ values at all sections occur when live load lies on one half span. This has been accepted as the basis for check calculations.

The truth behind this half-span load placement is better understood when influence lines for the road bridge are plotted. Influence lines for road bridge arch Fig 15 have been plotted for Bending Moment and Thrust. Due to heavy top fill and thickness of arch, splay of a point load on the bridge is considerable and is of varying dimensions due to varying depths of the arch profile below the road level. When referred to the influence line diagram, instead of measuring only ordinates under the load, the area of the diagram in the splay-limit zone has to be calculated. This makes the use of the influence line diagrams for moment and thrust computation fairly involved. Consequently these diagrams have been used only for heavier loadings (Class 70) on the roadway. Live load contribution to total maxima moment values is small, as the secondary moments are extraordinarily high. Any approximation in load placement, therefore, does not have much effect on the maxima stress values.

Moment and Thrust values have been worked out separately for each load component to appreciate their relative effect on design. These values have been worked out separately for dead load, live load, temperature and shrinkage. Rib-shortening correction is made for each loading. Live loads are generally converted into equivalent uniformly distributed loads and placed on one half span of the arch. Any special loads on the arch are treated separately.

For facility of calculation, M and T values for a unit UDL over the whole arch, and over one half span were analysed in the first instance. Corresponding values for different load intensities could then be worked out easily. Influence lines were used for check in case of heavy loading on the road bridge.

For temperature, no allowance has been made during check calculations for individual arch pour temperature characteristics, and original design assumption of a variation $\pm 30°F$ has been adopted. As the arches at Sukkur are very thick, temperature makes a major contribution towards the maxima design moments. To verify this assumption, thermometers have been embedded at different depths in one road bridge arch and one gate bridge arch and field observations are in hand. No firm values are available so far, but the field data is expected to yield valuable information for the future.

Shrinkage effect has been taken as equivalent to a fall of 15°F. It had been ignored in the initial design of the arch.

Normal and Reduced Arch Section Condition.

The design geometry of concrete arches at any section is accepted as the Normal condition. The design depth of concrete and quantity of steel are taken without any change.

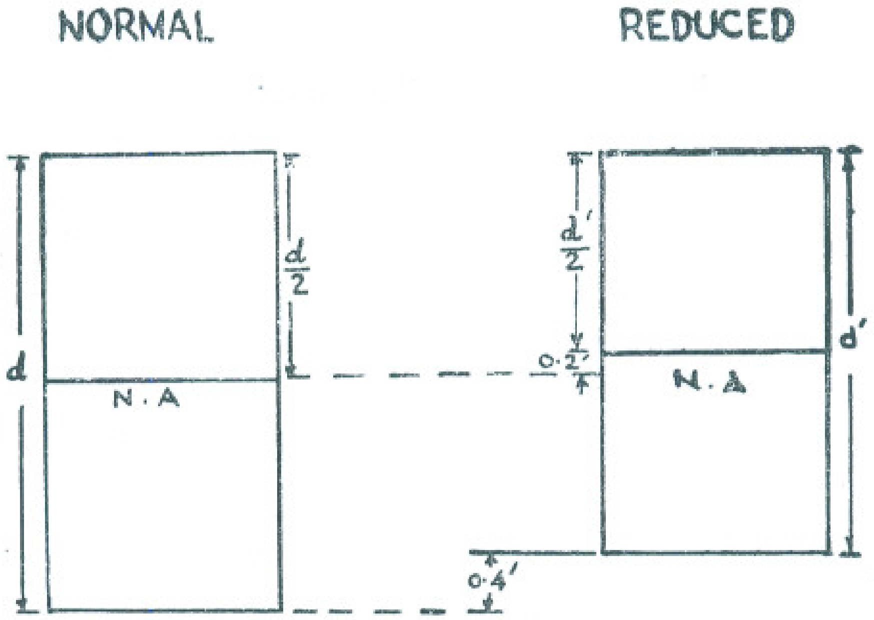


FIG. 7.

Field investigations show that the bottom most layer of steel in the arches is rusted, the rust intensity varying from arch to arch, and within an arch from point to point. The rust scale may exist all round the periphery of bar at any point or may be only on a part of the circumference. The rust varies from a thin, tenacious coating to heavy, laminated, friable incrustation up to 5/8" thickness. There is no sign of any harmful scaling of steel at the extrados, or in the second row of bars above the lower one at the intrados of road bridge arches. The penultimate condition of rusting is, therefore, taken when the bottom most intrados steel is so heavily rusted as to lose complete bond with concrete. The concrete is also supposed to have been burst by the expanding rust up to the top of the rusting steel and rendered structurally ineffective. This condition is accepted as Reduced Condition of a section. The clear cover of bottom steel is $2\frac{1}{2}''$ and the bars are $1\frac{1}{4}''$ dia. The effective concrete depth is therefore reduced by 0.4 ft.

In road bridge arches in certain sections of the haunches, and in the gate bridge arches, there is only one layer of intrados steel. Refer to Fig. 8-A,

REDUCED SECTION CONDITION

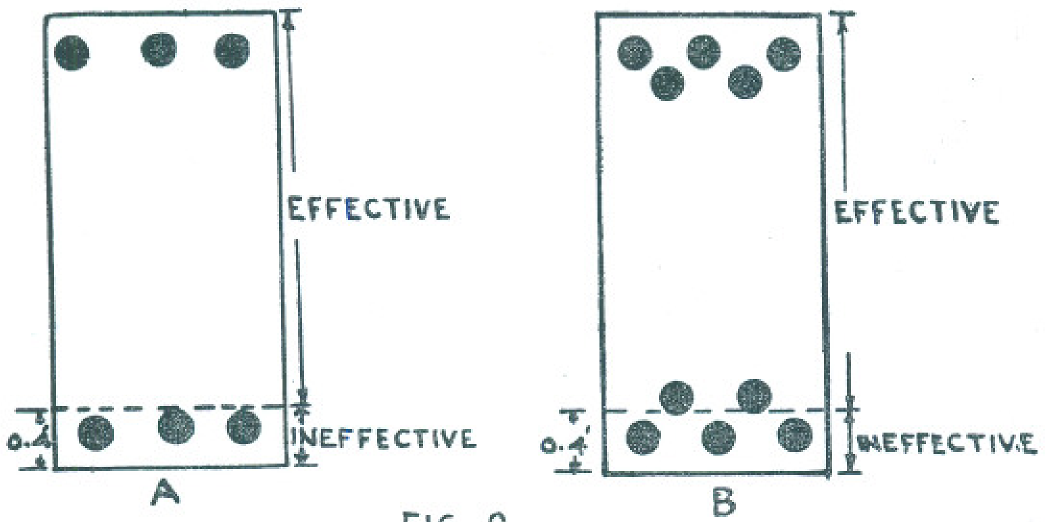


FIG. 8.

with this assumption of a reduced section, no steel is structurally available at the lower face. At other sections of crown and springings of road bridge with two layers of steel at the bottom, the upper layer of intrados steel will continue to be structurally effective as in Fig 8-B.

For checking the arches in the Reduced condition, fresh values of composite section areas and second moments of area are worked out. The second moments of area is worked out on the assumption that the neutral axis of the reduced section moves up by 0.2 ft, as in Fig 7. This is not strictly true, as neutral axis may lie farther up due to more steel at top. In that case, distances from neutral axis of upper and lower extreme fibres would also be different. This would complicate calculations considerably. As assumptions regarding Reduced Section are arbitrary, a simplifying assumption of the new neutral axis has been made, without causing any serious error.

At any section, value of I for Reduced condition is lower than values of I for Normal condition. Secondary moments due to temperature and rib-shortening will therefore have lower values when the arch is considered fully deteriorated. This, in a way, is a welcome relief, as secondary moments are very high. As the arch loses its strength, Maxima critical moments will also get reduced by the design assumptions and the critical sections may not be facing the brunt of full Normal moments. Computed stresses would therefore be lower than if tested for Normal maxima moments.

The assumptions for Reduced conditions are arbitrary, but may represent the worst foreseeable deterioration of the structure. The present state of damage to arches is that of a transition, and a considerable percentage of lower most steel at intrados face is effective. This only shows that present condition is better than the penultimate worst, and it may be worth-while to arrest further deterioration if it can be attempted economically.

While analysing the arches under Reduced Condition, shrinkage stresses are ignored. It will be too severe to allow for full deterioration of steel as well as for full shrinkage at this stage. Initial shrinkage strain by now may have been offset by plastic strain. No further shrinkage is likely to occur when the structure is already 35 years old.

An exercise has also been run presuming the arch in its transitional stage of damage, when half of the outermost bottom steel also continues to be structurally effective at any section. This steel was considered fully ineffective in Reduced Condition. All load and secondary moments are assumed to be the same as for the Normal Condition, and have been used for working out concrete and steel stresses for the transitional state of damage when steel at the intrados face is less than Normal. As the arch section in this case is unsymmetrically reinforced, with more steel at the top than at the bottom, stress calculations have to be made from first principles and require a lot of labour. In the Reduced condition also, concrete sections are unsymmetrically reinforced, and considerable labour is involved in computing concrete and steel stresseses.

Values of maxima positive or negative moments and corresponding thrust values for each component load are worked out for the segments of the arch, and the results are tabulated. The worst possible combination of various contributing factors is then considered to find the section maxima $\pm$ M value. Thrust corresponding to this value, which may not be the maxima thrust value at the section, is worked out by algebraic addition of components. Mathematically speaking, various thrusts should be compounded by vector analysis by resolving in two normal components to represent pure compression and pure shear. In actual fact, this refinement is not of much practical significance as the angle of resolution is small. Shear or thrust component parallel to section is small and shear stress is not considered in arch design. Normal component is almost equal to the total thrust value. While, therefore, vector addition to thrust components would be more accurate, this labour is not of much practical value.

For computing finally the $\pm$ M maxima value at any section, component values due to dead load and shrinkage effect have to figure. Live loads, and temperature rise or fall, generally give two separate values for positive or negative M, and correct component values have to be chosen for maxima section $\pm$ M or— M. There may, however, be a condition when a particular section may develop maxima value without live load and/or temperature, in which case that particular component may be ignored. Rib shortening correction is applied to each component value in the first instance, and no final correction is therefore required.

Individual arch sections are analysed as eccentrically loaded columns for concrete and steel stresses. For normal condition, arch sections can be analysed for f_c or f_s by resorting to standard charts and tables as steel is symmetrically placed. For Reduced condition, or with other assumptions for structural effectiveness of bottom steel, reinforcement is unsymmetrical and resort to first principles becomes necessary. If eccentricity $e=\frac{M}{T}$ is low, no tension develops on any face. If $\frac{e}{d}$ value is very high the case resembles more closely pure beam bending and analysis for stresses is done like a beam section in flexture after transferring T to centre of steel.

Line of Thrust or Linear Arch has some significance so long as only dead and live loads are considered. The funicular polygon in that case represents pure beam bending moment diagram. When load moments are compound with secondary moments due to temperature and shrinkage, the picture gets distorted. The diagram is no longer the beam B.M. diagram and has no physical significance. In reinforced arches when tension is allowed at the face, and value of T fluctuates with rise or fall of temperature resulting in change in e/d for the same loads, the line of thrust diagram ceases to have any significance or importance.

Before some of the results are analysed, it will be appropriate to discuss some of the important omissions and errors in initial design. It is not the intention here to cast any adverse reflections on the designer at this stage. As remarked earlier the job was done in a hurry. Even today, without a Central Design Office, and with the so-called 'oversafe' and hit-and-miss type of practices getting current for lack of centralisation and specialisation, the same mistakes are likely to be repeated by an unguarded, amateurish approach. The following are briefly the points which came to light during the present study:—

(*i*) The design calculations lack in thoroughness and precision, and many unqualified assumptions have been made, most probably out of ignorance.

(*ii*) Live load for maximum effect was placed on the arch, on the beam concept of maximum moments, over the entire span. Heaviest point loads were placed in mid-span. In actual fact, it was moı appropriate to load only one half of the span. By the adopte procedure while thrust values were maxima, Moment value were low. This gave low $e=\frac{M}{T}$ values.

(*iii*) Point loads were splayed up to arch extrados crown level, irrespective of load position. Splay effect should have been carried up to elastic axis.

(*iv*) Summation equations were used for computing the M and T values without any appreciation of their basis. Division of arch into n segments was not done to have $\frac{\delta S}{I}$ as constant, δ S was taken as $\frac{S}{n}$, and $\frac{1}{I}$ as arthmetic means of various section I values. In addition to n Segmental values, intermediate values of M and T were taken under point loads. This completely upset the very basis of the equations and the computed M and T values for loads and for temperature were incorrect.

(*v*) No consideration was given in design to heavy shrinkage moments and the rib-shortening effect. It was a piece of good fortune that the two omissions were complementary.

(*vi*) While computing maxima M and T values, maxima T values were computed independently, and used in design. In fact T value corresponding to maxima M value was required.

(*vii*) No analysis was made to evaluate relative influence on design of various load components and secondary moments. Live load contributed 7 to 12% while temperature influence was more than 70% on the critical Moment Values. By making the arch unnecessarily thick, secondary moments which vary as d^3 were greatly increased. True economy in changing over from stone to concrete as material of construction was lost and the arch had a reduced useful load carrying capacity.

Arch Profiles

1. *Road Bridge Arch.* The intrados is a true ellipse with major and minor axes as 60′ and 20′. The extrados is multi-segmental curve. With crown as the origin, radii of curvature are R=163′ from x=0 to x=12.5.′ R=48′ from x=12.5′ to x=23.5′, and R=41′ from x=23.5′ to x=34.25′. The arch profile has been so located as to bisect at right angles various section

ROAD BRIDGE ARCH
(GEOMETRICAL DETAILS)

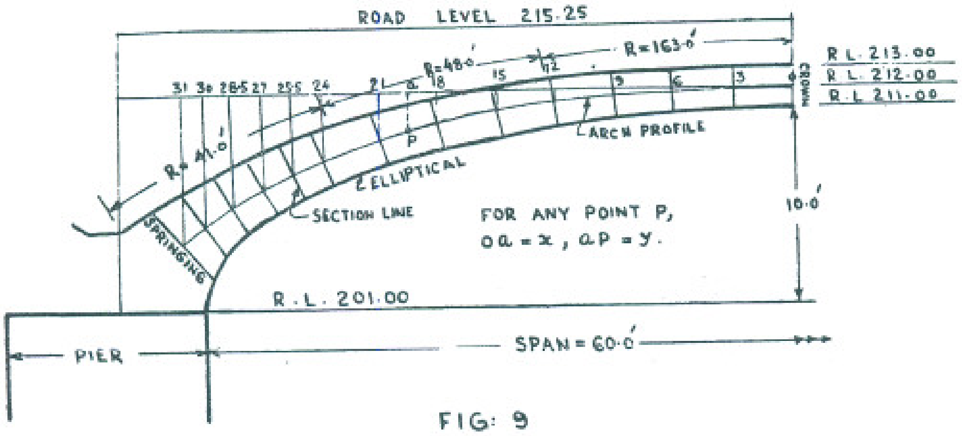


FIG: 9

1. INTRADOS IS A TRUE ELLIPSE. MAJOR AXIS = 60.0',
MINOR AXIS = 20.0'

2. EXTRADOS IS A FIVE CENTERED ARCH.
R = 163.0' FOR x = 0 TO x = 12.50',
R = 48.0' FOR x = 12.50' TO x = 23.50',
R = 41.0' FOR x = 23.50' TO x = 34.25'

lines (Fig. 9) and could be drawn by trial and error. Its coordinates were scaled from the diagram. Measuring x and y with profile crown as the origin (o, o,), the essential section values are :—

TABLE 1

Units are feet.

Section at x	0 Crown	3	6	9	12	15	18	21	24	25.5	27	28.5	30	31 Springing
y	0.00	0.05	0.17	0.36	0.63	1.07	1.65	2.40	3.40	4.00	4.75	5.60	6.65	7.50
Thickness of Section, d	2.00	2.00	2.05	2.20	2.35	2.53	2.70	2.80	2.90	3.00	3.22	3.46	3.90	4.40

The arch span is therefore $2 \times 31 = 62$ ft.

2. *Gate Bridge Arch.* The intrados is a true ellipse with major and minor axes as 60′ and 30′, extrados is multi-segmental curve. With crown as the origin, radii of curvature are R=75′ from x=0 to x=13.82, R=46.88′ from x=13.82 to x=23.25, and R=31′ from x=23.25 to x=34.25.

The arch profile is located similar to the case of road bridge arch (Fig. 10) and has the following characteristics, the arch span being 62 ft.

GATE BRIDGE ARCH
(GEOMETRICAL DETAILS)

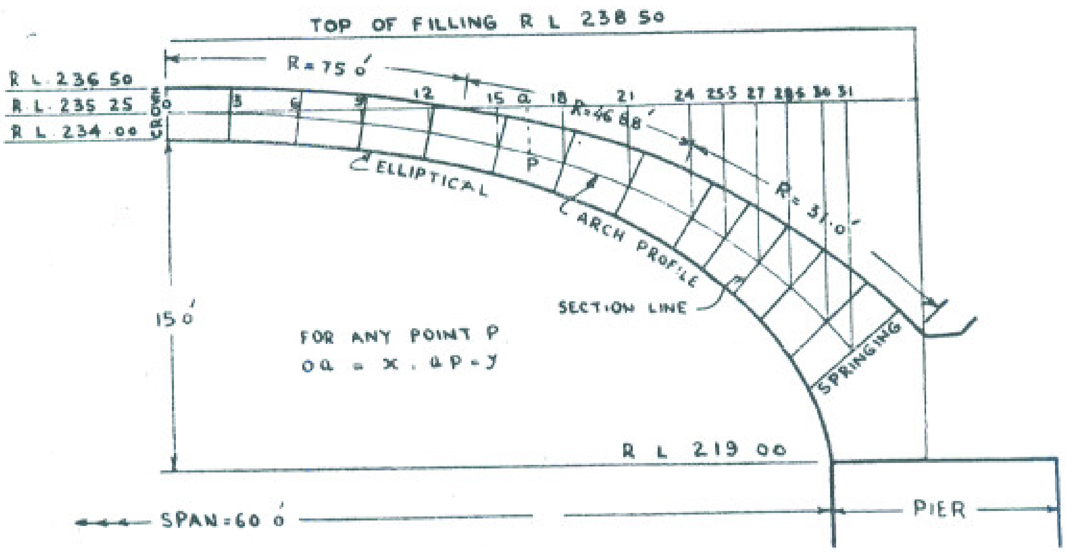


FIG 10

1 INTRADOS IS A TRUE ELLIPSE, MAJOR AXIS = 60', MINOR AXIS = 30'

2 INTRADOS IS A FIVE CENTRED SEGMENTAL ARCH
R = 75 0' FOR x = 0 TO x = 13 82',
R = 46 88' FOR x = 13 82' TO x = 23 75',
R = 31 00' FOR x = 23 75' TO x = 34.25'

TABLE 2

Units are feet

Section at x	0 Crown	3	6	9	12	15	18	21	24	25.5	27	28.5	30	31 Springing
y	0.00	0.10	0.27	0.62	1.10	1.75	2.65	3.70	5.15	6.00	7.00	8.20	9.70	10.85
Thickness of section d	2.50	2.50	2.55	2.60	2.70	2.85	3.00	3.20	3.60	3.80	3.95	4.25	4.70	5.10

The two arch profiles were checked to see if they could be fitted into a simple $y=f(x)$ relationship, in which case integral equations could be used to save considerable labour in calculations. None of the two profiles could be fitted into $y=cx^2$ (parabola), $\frac{x^2}{a^2}+\frac{y^2}{b^2}=1$ (ellipse) or $y=cx^2+dx^3$. Recourse was therefore necessary to Summation equations already discussed at some length.

Length of profile between Sections was measured by rotometer. Moment of Inertia I value for each section was computed by the usual method for composite concrete sections $I=I_c+(m-1)\ I_s$. Separate values were computed for Normal and Reduced conditions. S-I diagram was drawn for each with S along X-axis and $\frac{1}{2}$ I above and below the base line as ordinates. S-I diagram was divided by trial and error into desired number of segments such that $\frac{\delta S}{I}$ for each case is constant.

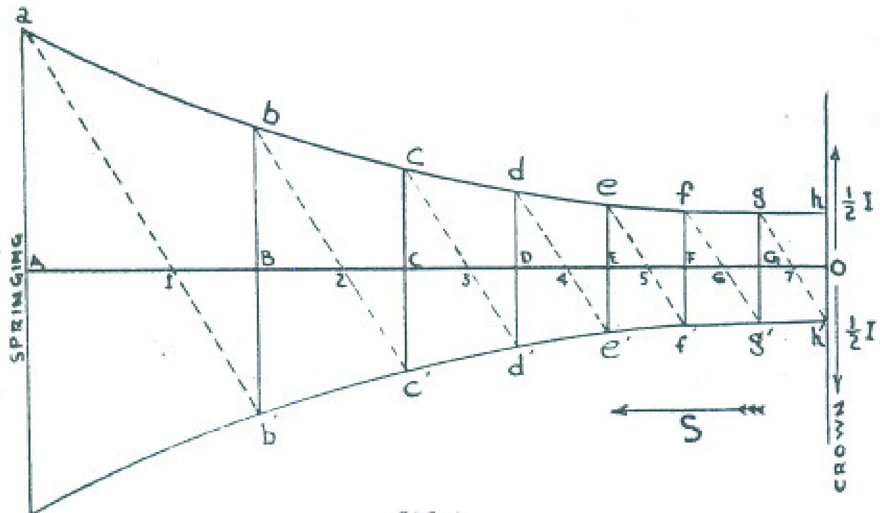


FIG: II

x and y values of the archprofile for mid-point of each segment were measured for use in the summation equations.

The road bridge arch profile was divided into 15 segments in each half, while the number of divisions was reduced to 10 for the gate bridge arch due to fewer loads. As I values are lower for Reduced condition as compared to I values for Normal condition, the exercise had to be repeated for the two cases. For the Road Bridge with $n=15$, $\frac{\delta S}{I}$ was 1.027 (Normal), and 1.95 (Reduced). In case of the gate bridge arch with $n=10$, the values were $\frac{\delta S}{I}=1.085$ (Normal) and 1.958 (Reduced).

Calculations of dead weight, and Resultant Moments and Thrust for only one case are given here to illustrate the procedure adopted. The case selected is the first exercise for Dead weight up to extrados crown level (RL 213.0) or road bridge arch (Normal Condition). Dead weight over each arch segment was computed, and this load was presumed to act at the centre of the Segment for calculating Moments. Density of stone masonry, reinforced concrete and lime concrete spandrel filling was accepted as 150 lbs per cft. (Reference Table 3)

TABLE 3

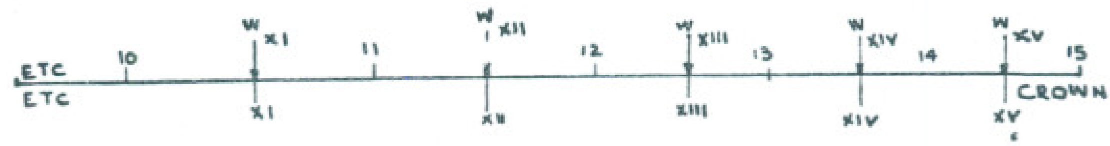
1. *Road Bridge Arch.* 2. *Linear units are feet.* 3. *Normal condition.*

Section	15 Crown	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
x	0.00	1.27	2.52	3.81	5.11	6.41	7.77	9.20	10.71	12.34	14.15	16.27	18.52	21.14	24.30	31.00
Ordinate Intrados to R.L. 213.00	2.00	2.00	2.05	2.10	2.15	2.23	2.36	2.50	2.70	2.80	3.20	3.55	4.10	4.90	6.10	8.50
Av : Ord	2.00	2.02	2.07	2.12	2.19	2.29	2.43	2.60	2.75	3.00	3.37	3.82	4.50	5.50	7.30	
δx	1.27	1.25	1.29	1.30	1.30	1.36	1.43	1.51	1.63	1.81	2.12	2.25	2.62	3.16	6.70	
Area	2.54	2.53	2.67	2.76	2.84	3.12	3.48	3.92	4.48	5.43	7.15	8.60	11.80	17.38	48.90	
W Segment (Kips)	0.381	0.380	0.400	0.414	0.426	0.468	0.522	0.587	0.673	0.815	1.072	1.290	1.77	2.60	7.33	
W running total	0.381	0.761	1.161	1.575	2.001	2.469	2.991	3.578	4.251	5.066	6.138	7.428	9.198	11.80	19.13	
Section	XV	XIV	XIII	XII	XI	X	IX	VIII	VII	VI	V	IV	III	II	I	
x at mid-point	0.64	1.90	3.16	4.46	5.76	7.09	8.49	9.96	11.53	13.25	15.21	17.40	19.83	22.72	27.65	
y at mid-point	0.005	0.02	0.06	0.11	0.14	0.22	0.33	0.45	0.60	0.82	1.10	1.53	2.11	2.96	5.15	

Average value of $\frac{\delta S}{I}=1.027$, n=no. of divisions=15.

As left and right half of the arch is symmetrically loaded, only one half of the arch is considered for analysis. Various M_L values for each segment are worked out as below :—

MOMENTS AND THRUSTS FOR NORMAL CONDITIONS



M_{xv}	$= 0$			$= 0.00$ K.ft.
M_{xiv}	$= M_{xv}$	$+0.381 \times 1.26$	$= 0 + 0.480$	$= 0.48$
M_{xiii}	$= M_{xiv}$	$+0.761 \times 1.26$	$= 0.48 + 0.96$	$= 1.44$
M_{xii}	$= M_{xiii}$	$+1.161 \times 1.30$	$= 1.44 + 1.51$	$= 2.95$
M_{xi}	$= M_{xii}$	$+1.575 \times 1.30$	$= 2.95 + 2.05$	$= 5.00$
M_{x}	$= M_{xi}$	$+2.001 \times 1.33$	$= 5.00 + 2.66$	$= 7.66$
M_{ix}	$= M_{x}$	$+2.469 \times 1.40$	$= 7.66 + 3.46$	$= 11.12$
M_{viii}	$= M_{ix}$	$+2.291 \times 1.47$	$= 11.12 + 4.40$	$= 15.52$
M_{vii}	$= M_{viii}$	$+3.578 \times 1.57$	$= 15.52 + 5.60$	$= 21.12$
M_{vi}	$= M_{vii}$	$+4.251 \times 1.72$	$= 21.12 + 7.30$	$= 28.42$
M_{v}	$= M_{vi}$	$+5.066 \times 1.96$	$= 28.42 + 9.94$	$= 38.36$
M_{iv}	$= M_{v}$	$+6.138 \times 2.19$	$= 38.36 + 13.44$	$= 51.80$
M_{iii}	$= M_{iv}$	$+7.428 \times 2.43$	$= 51.80 + 18.04$	$= 69.84$
M_{ii}	$= M_{iii}$	$+9.198 \times 2.89$	$= 69.84 + 26.58$	$= 96.42$
M_{i}	$= M_{ii}$	$+11.80 \times 4.93$	$= 96.42 + 58.18$	$= 154.60$
M_{o}	$= M_{i}$	$+19.13 \times 3.35$	$= 154.60 + 64.08$	$= 218.68$

From this, other Summation Values are worked out in the following table :

TABLE 4

Units are feet and kips.

Section	X	Y	X^2	Y^2	M_L	$M_L x$	$M_L y$
XV	0.64	0.005	0.41	0.000	0	..	..
XIV	1.90	0.02	3.61	0.000	0.48	0.91	0.01
XIII	3.16	0.06	9.99	0.004	1.44	4.55	0.09
XII	4.46	0.11	19.89	0.012	2.95	13.16	0.32
XI	5.76	0.14	33.18	0.020	5.00	28.80	0.70
X	7.09	0.22	50.27	0.048	7.66	54.31	1.68
IX	8.49	0.33	72.08	0.109	11.12	94.41	3.67
VIII	9.96	0.45	99.20	0.203	15.52	154.59	6.99
VII	11.53	0.60	132.94	0.360	21.12	243.51	12.67
VI	13.25	0.82	175.56	0.672	28.42	376.55	23.30
V	15.21	1.10	231.35	1.210	38.36	583.45	42.20
IV	17.40	1.53	302.76	2.341	51.80	901.33	79.25
III	19.83	2.11	393.21	4.452	69.84	1385.0	147.36
II	22.72	2.96	516.19	8.762	96.42	2190.7	285.40
I	27.65	5.15	764.51	26.523	154.60	4274.7	796.20
Σ	169.05	15.605	2805.15	44.716	504.73	10305.97	1399.84

Summation equations at Serial Nos. **13** to **15** will yield values for H_c, M_c and V_c in a general case. In the case under consideration, loading is symmetrical about the crown, simplified equations at serial Nos. **18** to **20** will apply, and M_R terms can be replaced by M_L term in all expressions.

$$\therefore V_c = 0$$

$$H_c = \frac{n\Sigma M_L y - \Sigma M_L \, \Sigma y}{n..y^2 - (\Sigma y)^2} = \frac{15\times1399.84 - 504.73\times15.605}{15\times44.716 - (15.605)^2} = 30.71 \text{ K}$$

$$M_c = \frac{\Sigma M_L - H_c\Sigma y}{n} = \frac{504.73 - 30.71\times15.605}{15} = 1.70 \text{ K.}$$

Moment and thrust at any point on the elastic axis can be determined readily from equation Nos. 16 and 17

$$M_x = M_c + V_c x + H_c y - M_L \qquad \text{(left of crown)}$$

$$= M_c - V_c x + H_c y - M_R \qquad \text{(right of crown)}$$

$$T_x = \sqrt{H_x^2 + V_x^2}$$

$H_x = H_c$ as there are no horizontal loads. $V_x = V_c + \Sigma_c^X W$

The computed results are shown in the following diagram.

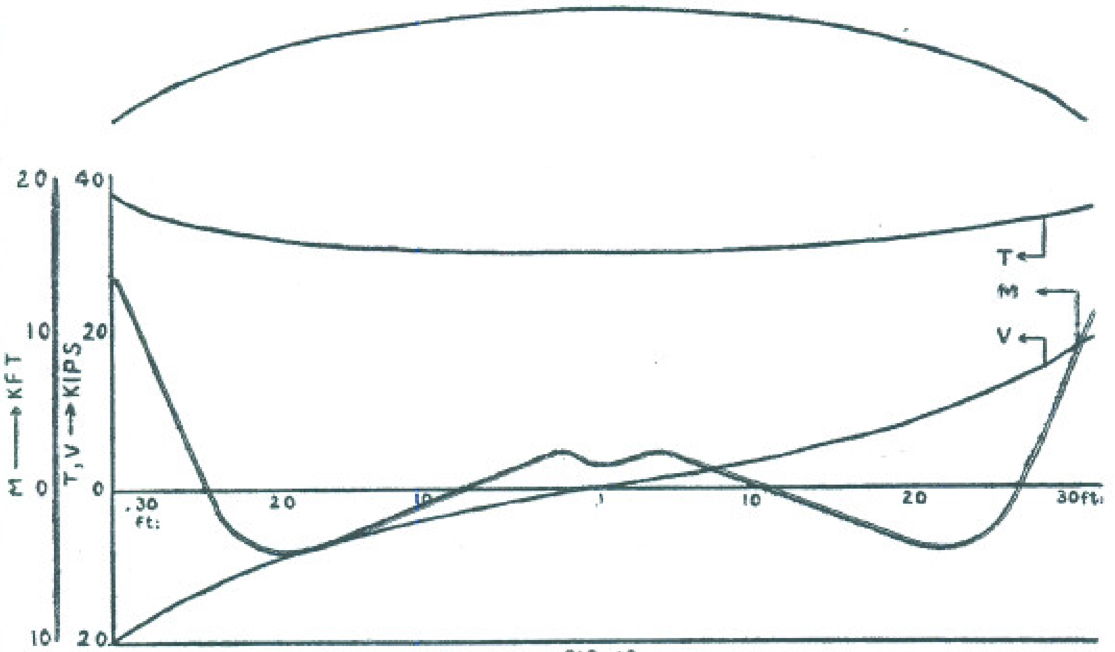


FIG: 12

Similar calculations were done for a UDL of IK per foot run over the entire span, and over left half of the arch span respectively. In the latter case, as loading was not symmetrical $V_c \neq o$, and each half of the arch had to be considered. As right half span had no load, M_R and all terms involving M_R reduce to zero. M_L and other values involving M_L have the same values as for UDL over the whole arch.

Temperature M and T values were computed by using equations No. 22 to 24. For temperature variations in the arch concrete mass :

$$H_{c_t} = \pm \frac{I.}{\delta S} \; \frac{\phi \, t \, L. \, n. \, E}{2[n\Sigma y^2 - (\Sigma y)^2]} \qquad (22)$$

Where t = Variation of temperature $= \pm 30^\circ$F.

ϕ = Coefficient of linear expansion of composite section $= 6 \times 10^{-6}$ per 1°F

n = No. of segments in half arch = 15.

L = Total effective span of the arch = 62′

E = Modulus of elasticity of concrete = 2..10^6 psi = 2.88×10^5 kips per sq. ft.

$\frac{I}{\delta S}$ =Ratio of divisions in S-I diagram.

$= \frac{1}{1.027}$ for Road Bridge (Normal).

$$M_{c_t} = \frac{-H_{c_t} . \Sigma y}{n} \tag{23}$$

$M_{c_{t,}}$ H_{c_t} for $\pm 30°$F.

$$Hc_t = \pm \frac{1}{1.027} \times \frac{6 \times 10^{-6} \times 30 \times 62 \times 15 \times 2.88 \times 10^5}{2 \times 427.13} = \pm 55.0 \text{ K.}$$

$$M_{c_t} = \mp \frac{55.0 \times 15.61}{15} = \mp 57.0 \text{ ft. K.}$$

M and T at other Sections :

M_x $= M_{c_t} + H_{c_t} . y$

T $=$ Constant $= T_c$ $= H_{c_t} = +55.0$K (Rise)

$= -55.0$ K (Fall)

Temperature effect was further corrected for rib-shortening. As has been discussed earlier, average compression of 12 *psi* on arch ring is equivalent to 1°F fall of temperature. Temperature thrust produced correction equivalent to 103 *psi* or 8.6°F, and got modified to (30°—8.6°) =21.4°F or 71% of the initial value. The corrected value was further used to calculate shrinkage values, and rib-shortening effect due to various loads. To complete the previous example, rib-shortening correction for Dead load up to RL 213.0 (Road Bridge, Normal) is worked out below, and is shown in Fig. 13.

ΣT (one half) =351.28 K

ΣA (one half) =40.61 ft.^{2}

$$\therefore f_c = \frac{\Sigma T}{\Sigma A} = \frac{351.28}{40.61} \times 6.95 \, psi = 60.0 \, psi = 5.0°\text{F fall.}$$

TABLE 5

1. *Units are feet and kips.*

Section	C	XIII	XI	IX	VII	V	IV	III	II	I	0
	Load Effect										
M	1.7	2.1	1.0	0.7	—1.0	—2.9	—5.1	—3.3	—3.8	5.3	18.49
T	30.71	30.72	30.76	30.83	30.97	31.21	31.45	31.78	32.45	34.21	36.1
	Rib Shortening Effect										
M	6.8	6.4	5.9	4.6	2.9	—0.4	—3.2	—7.0	—12.6	—25.8	—44.7
T						—5.4					
	Corrected Values										
M	8.5	8.5	5.9	5.3	1.9	—3.3	—6.3	—10.3	—16.4	—20.5	—31.3
T	24.3	24.3	24.4	24.5	24.6	24.8	25.0	25.4	26.0	27.8	29.9

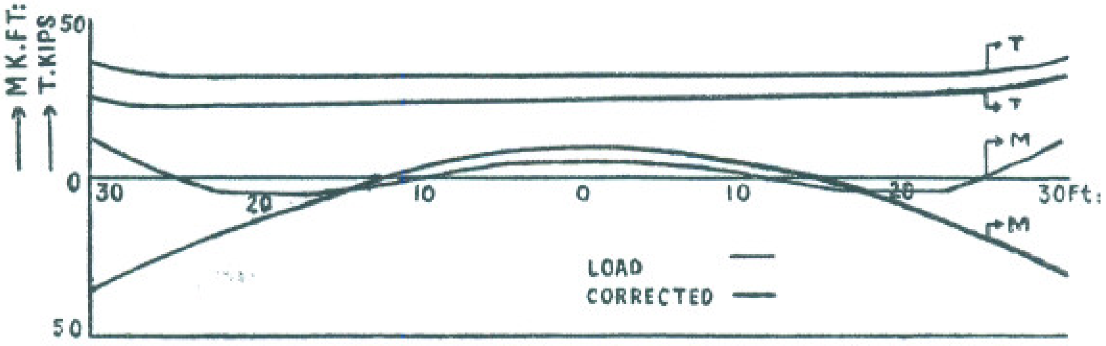


FIG:13

Similar exercises were done for each type of loading for each case of the arch.

Live Load on Road Bridge

16 ton lorry train accepted as design load is equivalent to a UDL of 180 lbs/ft^2. after splaying. Impact factor, corrected for damping due to heavy spandrel fill is 10 to 15%. Equivalent live load comes to $180 \times 1.15 = 207$ say 200 lbs per foot run of each traffic lane.

Summary of Maxima Moment and Thrust values by components and in combination for Road Bridge (Normal Condition) is given in the Table 6 and is represented graphically in Fig. 14 for ready appreciation. Mark the great influence of temperature on Maxima design values.

An appraisal of these results will indicate that :—

1. Dead load net M and T values are severer than Live load values, and are 2 to 6 times of the latter. This is due to heavy spandrel filling. The filling from RL 213.0 to road surface RL 215.25 alone is almost as severe in effect as the Live load.

2. Secondary M values due to temperature are very high and completely eclipse the Live load values. At critical sections (crown and springing), they are 13 to 16 times the live load values. When compared to combined load values (Live + Dead loads), the temperature effect is 2 to 5 times as great. As temperature moments are directly proportional to I or d^3 this indisputably points towards an unusually thick concrete arch, with poor load carrying capacity.

3. Shrinkage M values follow the same pattern as temperature moments, and are quite high.

4. Rib shortening moments which are also a function of arch section are high. They partly relieve the high temperature moments.

5. A thinner arch with the same outline geometry may have been stronger by achieving sharp reduction in secondary moments.

6. With rise of temperature, T is +ive (compression), while M is negative from crown to quarter span, and positive from quarter span to springing. The sign is reversed for fall of temperature, and T is negative (tension).

TABLE. G

SECTION	C	XIII	XI	IX	VII	V	IV	III	II	I	O
X	0.00	3.16	5.76	8.45	11.55	15.21	17.40	19.83	22.72	27.65	31.00
d	2.00	2.00	2.05	2.18	2.33	2.54	2.68	2.76	2.86	3.32	4.40
I	1.24	1.24	1.32	1.43	1.66	2.10	2.33	2.70	3.32	5.30	11.46
A	3.23	3.23	3.28	3.41	3.05	3.26	3.40	3.48	4.05	4.55	5.63
Z	1.24	1.24	1.25	1.31	1.42	1.65	1.74	1.96	2.32	3.20	5.20
TOTAL STEEL p%	4.4	4.4	4.3	4.0	2.2	2.0	1.9	1.9	3.1	2.6	2.0
	MOMENTS										
DEAD LOAD	18.6	18.2	14.3	10.0	1.5	-3.7	-16.3	-23.3	-32.8	-34.7	-32.6
LIVE LOAD RIGHT	3.0	0.3	-1.8	-3.3	-4.3	-5.3	-4.5	-3.8	-1.5	7.0	16.5
LEFT	3.0	5.5	6.3	6.8	4.7	1.5	-0.5	-4.0	-8.2	-13.9	-18.3
TEMPERATURE RISE	-40.7	-38.4	-35.2	-27.7	-17.2	+2.5	15.4	42.0	75.5	154.5	268.0
FALL	40.7	38.4	35.2	27.7	17.2	-2.5	-15.4	-42.0	-75.5	-154.5	-268.0
SHRINKAGE	20.4	19.2	17.6	13.9	4.6	-1.3	-9.7	-21.0	-37.8	-77.3	-134.0
	THRUSTS										
DEAD LOAD	44.7	44.7	44.8	44.9	45.3	45.7	46.1	46.6	47.5	49.8	52.2
LIVE LOAD RIGHT	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1
LEFT	6.1	6.0	6.1	6.1	6.1	6.3	6.4	6.6	6.8	7.3	7.7
TEMPERATURE RISE	39.2	39.2	39.2	39.2	39.2	39.2	39.2	39.2	39.2	39.2	39.2
FALL	-39.2	-39.2	-39.2	-39.2	-39.2	-39.2	-39.2	-39.2	-39.2	-39.2	-39.2
SHRINKAGE	-19.6	-19.6	-19.6	-19.6	-19.6	-19.6	-19.6	-19.6	-19.6	-19.6	-19.6
MAXIMA MOMENTS AND CORRESPONDING THRUSTS											
1. (DEAD + LIVE) LOADS											
MAXIMA +M	21.6	23.2	20.6	16.8	6.6	...	...	...	...	...	...
-M	...	...	...	...	-2.4	-15.0	-21.2	-27.3	-41.0	-45.6	-50.9
CORRESPONDING T FOR +M	50.8	50.7	50.9	51.1	51.4	...	...	...	...	...	...
T FOR -M	...	...	...	...	51.4	51.8	52.2	53.2	54.3	57.1	59.9
CORRESPONDING 'E' FOR +M	0.41	0.46	0.41	0.33	0.13	...	...	...	...	...	...
'E' FOR -M	...	...	...	...	-0.04	0.29	0.41	0.51	0.75	0.80	0.85
2. LOADS + TEMPERATURE											
MAXIMA +M	61.7	61.6	55.8	44.5	23.8	-5.7	3.1	18.7	42.7	129.8	252.3
-M	-22.1	-20.2	-22.7	-21.0	-19.6	-17.5	-40.6	-69.3	-116.5	-200.1	-318.9
CORRESPONDING T FOR +M	11.6	11.5	11.7	11.9	12.2	91.2	85.3	85.8	86.7	95.1	97.5
T FOR -M	83.9	83.9	90.1	90.2	90.6	12.6	13.0	14.0	15.1	17.9	20.7
CORRESPONDING 'E' FOR +M	5.32	5.36	4.77	3.74	1.95	-00.06	0.04	0.22	0.49	1.36	2.55
'E' FOR -M	-0.26	-0.24	-0.25	-0.23	-0.22	-1.39	-3.12	-4.95	-7.71	-11.20	-15.40
CORRESPONDING E/D FOR +M	2.66	2.68	2.33	1.72	0.84	-0.02	0.01	0.08	0.17	0.41	0.59
E/D FOR -M	-0.13	-0.12	-0.12	-0.11	-0.09	-0.55	-1.16	-1.79	-2.70	-3.38	-3.50
3 LOAD + TEMPERATURE + SHRINKAGE											
MAXIMA +M	82.1	80.8	73.4	58.4	33.4	-7.0	-6.7	-2.3	4.9	52.5	118.3
-M	-1.7	-1.0	-5.1	-7.1	-10.4	-18.8	-50.3	-90.3	-154.3	-277.4	-452.9
CORRESPONDING T FOR +M	-8.0	-8.1	-7.9	-7.7	-7.4	71.6	65.7	66.2	67.1	75.5	77.9
T FOR -M	64.3	64.3	70.5	70.6	71.4	-7.8	-6.6	-5.6	-4.5	-1.7	1.1
CORRESPONDING 'E' FOR +M	-10.26	-10.0	-9.35	-7.60	-4.51	-0.10	-0.10	-0.03	-0.70	-0.70	1.52
'E' FOR -M	-0.03	-0.02	-0.07	-0.10	-0.14	2.69	7.61	16.30	34.3	163.0	411.7
CORRESPONDING E/D FOR +M	-5.13	-5.0	-4.56	-3.50	-1.94	-0.04	0.04	0.01	-0.03	-0.21	0.35
E/D FOR -M	0.02	-0.01	-0.03	-0.04	-0.06	1.06	2.84	5.90	12.0	49.1	93.7

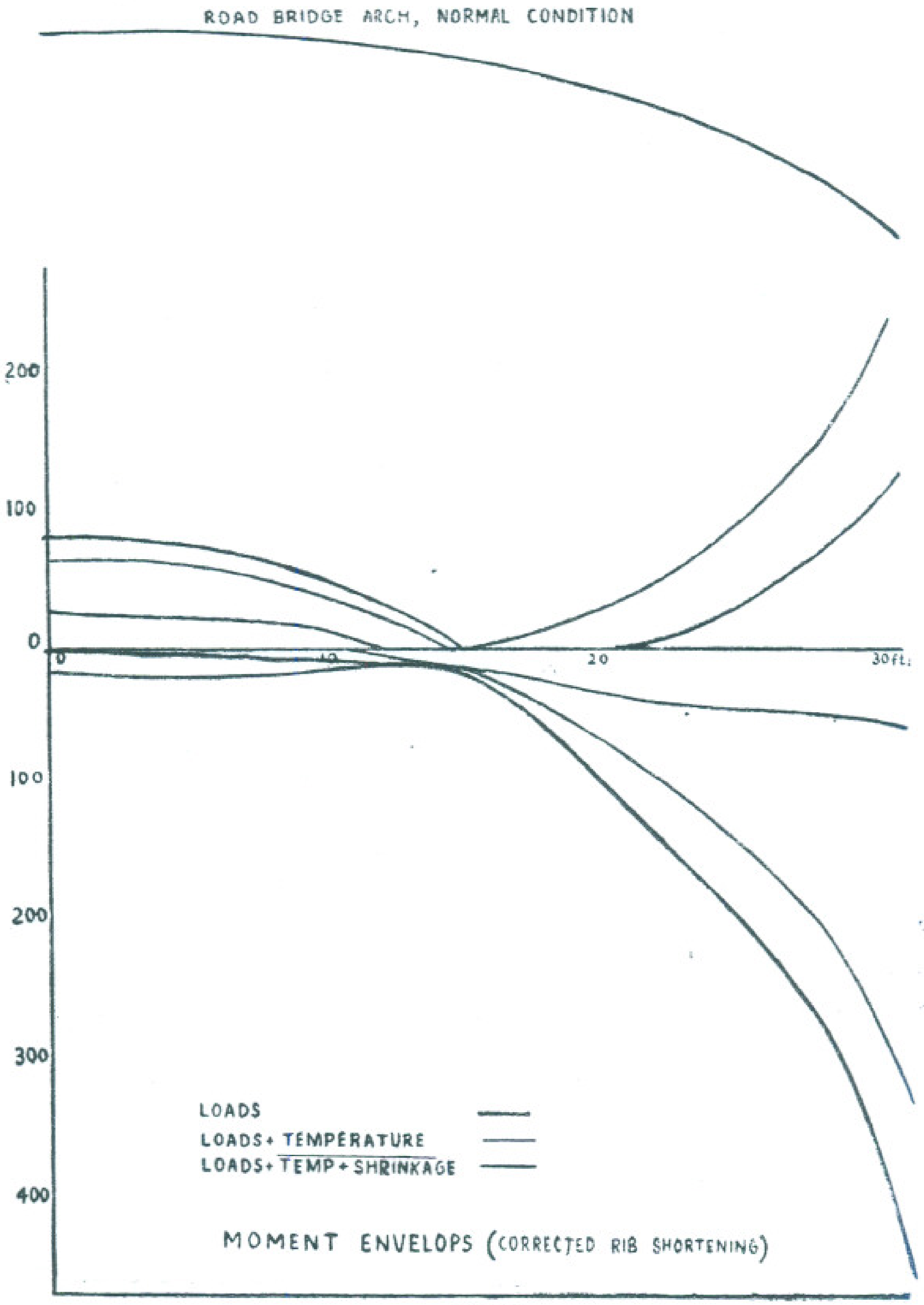

ROAD BRIDGE ARCH, NORMAL CONDITION
200
100
0
100
200
300
400
0
10
20
30 ft.
LOADS
LOADS + TEMPERATURE
LOADS + TEMP + SHRINKAGE
MOMENT ENVELOPS (CORRECTED RIB SHORTENING)

FIG: 14

Shrinkage and Rib shortening M and T values follow the pattern of those due to fall of temperature.

7. Temperature fall, shrinkage and Rib shortening generate tension and reduce thrust at sections. They also increase any net positive moments from crown up to quarter span and reduce any net negative moments in this region. From quarter span up to the springing any net positive moments get reduced, and net negative moments get increased.

Due to fall in net thrust, there is a virtual increase in eccentricity. The eccentricity values will be widely different for rise and fall of temperature.

8. (Dead+Live) load Moments values are different from similar values if the arch were a voussoir arch when no rib shortening corrections are applied.

Similar results were computed for Road bridge arch (Reduced condition), Upstream Gate bridge arch (Normal), Downstream Gate bridge arch (Normal), Upstream Gate bridge arch (Reduced) and Downstream Gate bridge arch (Reduced). There is no space here to elaborate on individual cases, but the foregoing example is indicative of the procedure.

Stress Calculations

Having tabulated Maxima M corresponding T values at critical arch Section with different assumptions regarding the extent of damage, the stage is set for working out steel and concrete stresses. Discussion on finer points of design and on the detailed procedure for various special cases can hardly find any room in this paper. Only a summary of various important results is given below :

TABLE 7

1. *Road Bridge Arch* (*Normal Condition*) 2. *Units are* psi.

Section.	C x=0.00′ Crown	XI 5.76′	IX 8.49′	V 15.21′	II 22.72′	I 27.65′	0 31.00′ Springing
			1. Loads + Temperature				
Maxima f_c intrados	287	292	276	216	415	562	595
extrados	370	318	234	170	266	460	530
Maxima f_s intrados	6070	5000	3360	..	..	2370	5160
extrados	..	..	..	1010	7440	11540	13400
			2. Loads+Temperature+Shrinkage				
Maxima f_c intrados	151	168	170	183	398	608	665
extrados	340	285	208	123	129	230	278
Maxima f_s intrados	8250	6800	5600	..	..	..	1050
extrados	..	..	..	3680	9750	15200	18300

TABLE 8

1. *Road Bridge Arch* (*Reduced Condition*). 2. *Units are* psi. (3) *Only* (*loads* +*Temperature*).

Section		C $x =$ 0.00 Crown	XI 5.59′	VIII 9.52	IV 16.02	II 21.02′ No bottom steel	II 21.02′ bottom steel	I 26.78′	0 31.00 Springing
Maxima f_c	intrados	300	328	372,—133	350	663	465	690	560
	extrados	405	373	290	175	270	240	415	550
Maxima f_s	intrados	7600	7000	Steel	ineffective		..	1560	8250
	extrados	..	..	..	2370	7950	4860	9000	8400

TABLE 9

1. *Gate Bridge Arch* (*Normal Condition*) 2. *Units are* psi.

Section.		C $x=$ 0.00 Crown	VIII 5.45	VI 9.97	IV 14.77	II 20.95	I 26.91	0 31.00 Springing
	1. Upstrram Arch (Loads+Temperature).							
Maxima f_c	intrados	290	280	295	242	455	593	652
	extrados	418	411	330	110	238	422	490
Maxima f_s	intrados	6550	6450	5650	..	..	2580	5360
	extrados	..	..	..	..	8550	13400	16100
	2. Upstream Arch (Loads+Temperature+Shrinkage)							
Maxima f_c	intrados	157	168	195	198	580	790	855
	extrados	530	480	394	88	129	192	250
Maxima f_s	intrados	11400	10300	9350	..	..	..	520
	extrados	..	..	..	..	15000	19500	22200
	3. Downstream Arch (Maximum Loads+Temperature)							
Maxima f_c	intrados	283	260	260	250	465	600	600
	extrados	440	410	328	114	240	390	470
Maxima f_s	intrados	6600	6200	4120	..	..	1670	4100
	extrados	..	..	..	..	8350	12300	13900
	4. Downstream Arch (Operational Maximum Loads+Temperature)							
Maxima f_c	intrados	283	260	260	205	410	580	590
	extrados	400	357	291	100±	240	390	435
Maxima f_s	intrados	7170	6100	4980	..	..	1670	4150
	extrados	..	..	..	..	8800	13000	14600

TABLE 10

1. *Gate Bridge Arch (Reduced Condition)* 2. *Units are* psi.

Section	C $x=0.00$ Crown	VIII 4.91	VI 9.06	III 16.24	II 19.46	I 26.13	0 31.0 springing
UPSTREAM ARCH							
Maxima							
f_c intrados	298,- 246	306-249	288,-162	240	495	666,-94	620,-174
extrados	424	425	334	206	203	390	417
Maxima							
f_s intrados	Steel incapable of taking any stress						
extrados	..	..	..	very low	5400	9150	10300
DOWNSTREAM ARCH							
Maxima f_c							
intrados	280,-256	274,-224	250,-120	256	470	582.-74	600,-142
extrados	421	390	280	256	206	360	380
Maxima f_s							
intrados	Steel incapable of taking any stress						
extrados	..	..	..	very low	5400	7650	9840

Road bridge was further checked for design loads with impact factor of 50% as accepted in initial design. Reduced section was further checked for no live load. A summary of comparative stress values with $i=50\%$ and $i=15\%$ for these cases is given in Tables 11 and 12.

TABLE 11

1. *Normal.* 2. *Units are psi*

Section.			$i=50\%$ f_c	$i=50\%$ f_s	$i=15\%$ f_c	$i=15\%$ f_s
		1. Loads+Temperature				
C $x=0.00$	intra	..	287	6700	287	6070
	extra	..	408	..	370	..
I $x=27.65$	intra	..	572	2420	562	2370
	extra	..	470	11750	460	11540
O $x=31.00$	intra	..	590	5270	595	560
	extra	..	542	12700	530	13400
		2. Loads + Temperature + Shrinkage				
C	intra	..	151	8200	151	8250
	extra	..	353	..	340	..
I	intra	..	630	..	608	..
	extra	..	236	15300	230	15200
O	intra	..	682	1080	665	1050
	extra	..	286	18300	278	18300

TABLE 12

Reduced Section

1. *Reduced.* 2. *Units are* psi.

Section		i=50%		i=15%		No Live Load	
		f_c	f_s	f_c	f_s	f_c	f_s
	Loads + Temperature						
C x=0.00	intra	300	8300	300	7600	300	8100
	extra	440	..	405	..	376	..
VIII x=9.52	intra	390 −143	Steel ineffective	372 −133	Steel ineffective	338 −109	Steel ineffective
	extra	313	..	290	..	224	..
I x=26.78	intra	750	1600	690	1560	530	1440
	extra	425	9800	415	9000	384	7260
O x=31.00	intra	607	8450	560	8250	445	7300
	extra	563	9100	550	8400	510	7300

Road bridge was also checked for heavier present-day loading Class B, Class A, Class AA, Class 40 and Class 70. The Moment and Thrust values were further checked by the use of Influence Lines. Influence lines for Moments and Thrust are in Fig. No. 15 and have been corrected for rib-shortening. Their use was pretty complicated due to load-splay of varying order due to varying depth of the point load to the elastic axis. The accepted criteria of loading one half of the arch for maximum M is pretty satisfactory, except at the crown section. Refer to Fig. 15.

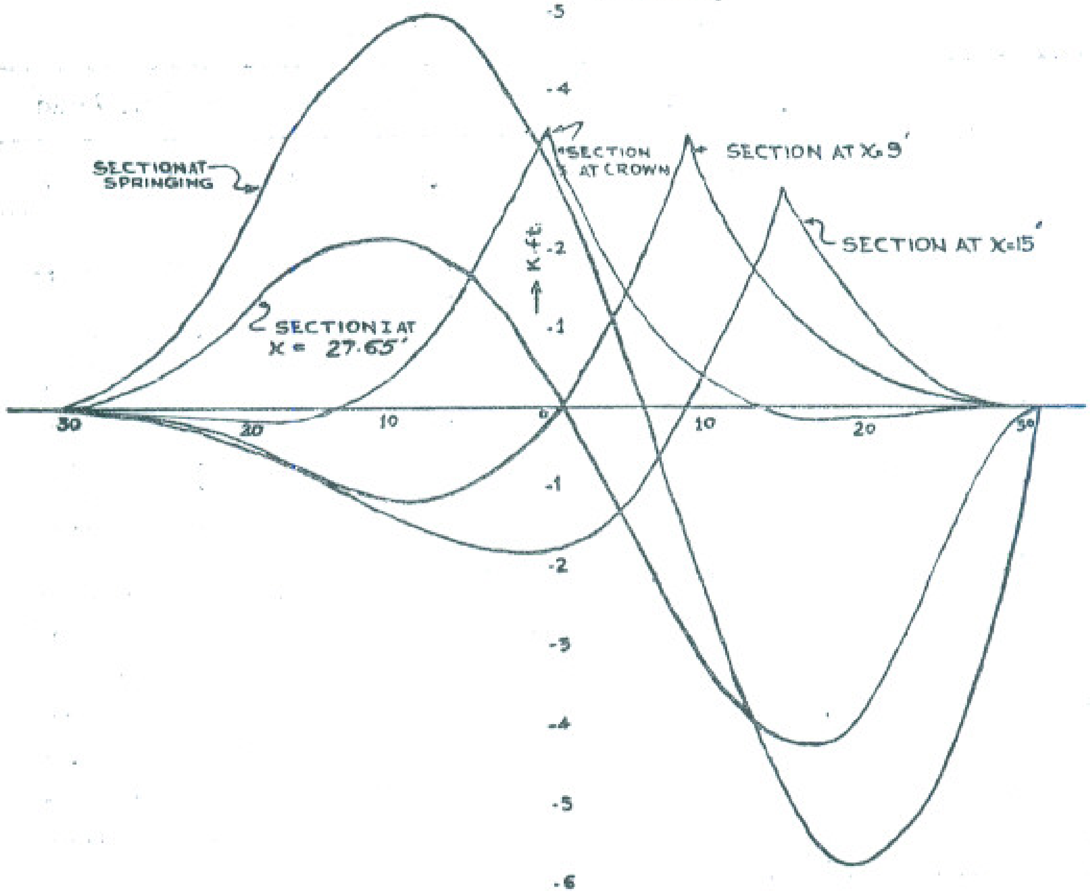
A summary of the stress values for the road bridge are given in Tables 13 and 14.

TABLE 13

1. *Road Bridge Arch.* 2. *Reduced Section.* 3 *Units are* psi. 4. *For (Loads+Temperature) only ignoring Shrinkage.*

Section		Class A or class 40 load on a single Lane.		Class A loading on 10-feet. traffic lane.		Class AA or 70 loading on a single traffic lane.	
		f_c	f_s	f_c	f_s	f_c	f_s
C at x=0	intrados	300	8100	300	7850	300	7400
	extrados	430	..	470	..	495	..
VIII at x=9.52	intrados	385,140	No steel	420,150	No steel	456,159	No steel
	extrados	306	..	340	..	362	..
I at x=26.78	intrados	730	1590	765	1650	780	1700
	extrados	422	9560	435	9800	450	9800
O at x=31.0	intrados	594	8400	625	8700	640	8950
	extrados	560	8900	580	9150	595	9200

ROAD BRIDGE (NORMAL CONDITION)
MOMENT INFLUENCE LINES (WITH RIB SHORTENING EFFECT)



ROAD BRIDGE (NORMAL CONDITION)
THURST INFLUENCE LINES (WITH RIB SHORTENING EFFECT) ANY SEC.

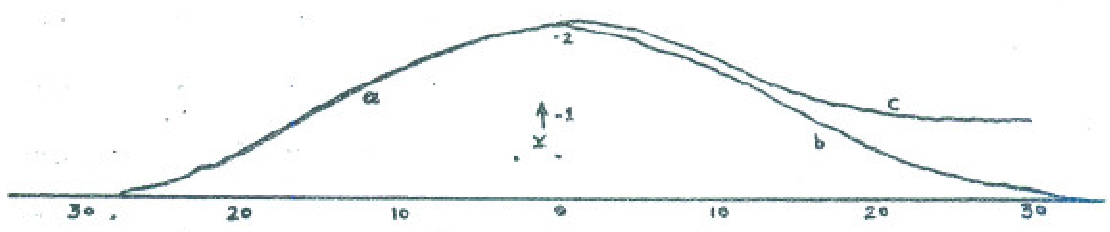


TABLE 14

1. *Road Bridge Arch* 2. *Normal Section* 3—*Units are* psi.

		Class A or Class 40 load on a single traffic lane.				Class A loading on 10-feet traffic lane.				Class AA or 70 loading on a single traffic lane or class 40 loading on 10′ traffic lane.			
		Loads+Temperature		Loads+Temperature+Shrinkage.		Loads+Temperature		Loads+Temperature+Shrinkage.		Loads+Temperature		Loads+Temperature+Shrinkage.	
		f_c	f_s	f_c	f_s	f_c	f_s	f_c	f_s	f_c	f_s	f_c	f_s
C at x=0	intrados	287	6500	151	8250	287	6550	151	7950	287	6370	151	7700
	extrados	400	..	350	..	437	..	390	..	465	..	423	..
I at x=27.65	intrados	570	2400	623	..	604	2850	650	..	637	3280	670	..
	extrados	467	11700	235	15270	505	12100	290	15300	540	12500	342	15300
O at x=31.0	intrados	590	5240	677	1070	600	5670	690	1190	610	6080	705	1300
	extrados	538	13050	283	18300	560	12900	305	18500	580	13100	324	18700

For a better understanding different load systems on road bridge are reduced to the following equivalent Uniformly Distributed Loads :—

1.	Design load of 16-ton lorry train, with impact factor	i=15%=0.20 K/ft.
2.	Design load of 16-ton lorry train, with impact factor	i=50%=0.27 K/ft.
3.	Class B, IRC loading on 10′ traffic lane,	i=15%=0.20 K/ft.
4.	Class A, IRC loading on a single traffic lane,	i=15%=0.24 K/ft.
5.	Class 40 army loading on a single traffic lane,	i=15%=0.25 K/ft.
6.	Class A, IRC loading on a 10′ traffic lane,	i=15%=0.35 K/ft.
7.	Class 40 army loading on a 10′ traffic lane,	i=15%=0.42 K/ft.
8.	Class AA, IRC loading or class 70 army loading on a single traffic lane	i=15%=0.43 K/ft.

In case of Gate bridge, an exercise was undertaken for emergency loading on the bridgeway when a gate got jammed or was damaged by sabotage. It was concluded from the computed moment values that the Normal Operation Loads criteria already used in check calculations was more severe. No fresh stress values were therefore worked out.

Arches under transitional stage of damage

Road bridge Arch No. 7 was opened up on an experimental basis. In the mostly heavily rusted bars, the loss of steel area did not exceed 20%. A point load was, however, expected to splay at least to a 5-ft. wide strip, and average loss of area over the 5-ft. wide strip did not exceed 5%. It was, however, felt that it was not the loss of steel area but the loss of bend between steel and concrete which really mattered. It was arbitrarily assumed that at the worst section in a 5 ft. wide strip, at least half the steel area would be structurally effective. This was rather a conservative assumption as, in actual fact, the state of damage is less severe.

Moment and thrust values were accepted the same as for Normal condition but steel in the outermost bottom layer was reduced to half for taking any stresses. For the road bridge, this exercise was run for design loads only. For the gate bridge, only Upstream arch with heavier loading was checked for the critical operational loads. The following is the summary of stress values at critical sections :—

TABLE 15

1. *Road Bridge.* 2. *Half outer bottom steel effective.* 3. *Units are* psi.

Section	Loads + Temperature				Loads+Temp.+Shrinkage			
	f_c		f_s		f_c		f_s	
	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos
C. crown at $x=0$	325	356	7000	..	156	370	11050	..
Section at $x=9$	366	454	13620	..	226	314	16900	..
I at $x=27.65$	553	467	2700	9900	665	278	..	15200
0 springing at $x=$ 31.0	574	554	6650	11800	710	280	1200	18400

TABLE 16

1. *Gate Bridge Upstream.* 2. 50% *bottom steel effective* 3. *Units are* psi.

Section	Loads + Temperature				Loads + Temp+ Shrinkage			
	f_c		f_s		f_c		f_s	
	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos
C crown at $x=0$	316	540	14000	..	168	690	24200	..
Section at $x=9$	318	470	13500	..	206	560	23200	..
I at $x=27.91$	615	446	3350	12000	870	200	..	21400
O springing $x=$ 31.01	700	540	8100	16500	900	240	..	24400

TABLE 17

Arch Section	Maxima Stress as per Original Design Calculations (lbs per sq. in.)		Maxima Stress as Checked (lbs per sq. in.)											
			Steel Normal Section				Reduced Section				50 % Steel in bottom layer effective			
			Steel		Concrete		Steel		Concrete		Steel		Concrete	
	Steel (tension)	Concrete (compression)	Intrados	Extrados	Intrados	Extrados	Intrados	Extrados	Intrados	Extrados	Intrados	Extrados	Intrados	Extrados
Gate Bridge Upstream Arch														
1. Crown Section ...	4781	480	11400	...	157	530	Steel in effective	...	298, —246	424	24200	...	168	690
2. Section I, about 5 ft. from Springing. ...	10321	573	...	19500	790	192		9150	666, — 94	390	...	21400	870	200
3. Section 0 at Springing ...	Not calculated		520	22200	855	250	Do.	10300	620, —174	417	...	24400	900	240
Road Bridge Arch														
1. Crown Section ...	4381	429	8250	...	151	340	7600	...	300	405	11050	...	156	370
2. Section VIII, about 9 ft. from Crown ...	5133	403	5600	...	170	208	Steel in-effective	...	372, —133	290	16900	...	226	314
3. Section I, about 4 ft. from Springing ...	8218	479	...	15200	608	230	1560	9000	690	415	...	15200	665	278
4. Section 0, at Springing ...	Not calculated		1050	18300	665	278	8250	8400	560	550	1200	18400	710	280

These high stress values may not be taken as alarming as :—

(*i*) The assumption of 50% steel is much too severe.

(*ii*) In case of any spot failure, stress will be reduced due to local redistribution of moments, affording relief to weaker sections.

(*iii*) No reduction has been made in Secondary M and T Values.

(*iv*) For worst combination of various contributing factors, permissible stresses in concrete and steel are 20% higher than normal.

A summary of original design and checked stress values for the Road Bridge and the Gate Bridge Arches is given in Table 17 for ready appreciation.

Ultimate Stress Theory

It was considered at one stage to check the reinforced concrete arches for their strength by the Ultimate Strength Theory or the Plastic Theory. The arches are heavily reinforced and are designed as eccentrically loaded columns. Their strength indicated by the Ultimate Strength Theory was likely to be higher than that determined by the conventional Elastic Theory. In the Elastic Theory for the design of R.C.C. structures, working stress of concrete is determined by dividing the crushing strength of concrete by a factor of safety varying from $\frac{1}{0.33}$ to $\frac{1}{0.45}$. Factor of safety for steel is taken as $\frac{1}{0.4}$. Both steel and concrete are assumed to follow Hook's Law within the working stresses. It is known that concrete is subject to considerable plastic strains under a constantly applied load. In fact elastic and plastic deformations in concrete occur simultaneously. The plastic deformation alters the stress-strain relationship and transfers greater stresses to steel. The plastic flow also relieves concrete of concentrated local stresses, especially in the most highly stressed parts. Under the Ultimate Strength Theory, a more realistic approach to design is made and actual behaviour of both steel and concrete under-loads is taken into account. The structure is first designed for an ultimate load for failure after which working loads are determined by allowing factors of safety different for different cases and consistent with the type of loading. This theory is eminently suited to the design of axially and eccentrically loaded columns, and effected considerable economy in the design of compression members especially when they have to be heavily reinforced.

The arch when tested with Elastic Theory indicated its capacity of taking heavier live loads than originally accepted in the design. Its strength under Plastic Theory would be still higher and would be useful in making out a strong case for the design of a decking for class AA loading on the like-for-like basis if the arches were to be replaced. At the present stage of damage to the steel reinforcement, immediate replacement of the arches is not foreseen. Under the deteriorated condition, when complete loss of bond for the bottom row of steel at the intrados face is accepted, the Plastic Theory could be of doubtful utility as at many sections there was no steel structurally effective to take compression/tension. This theory was therefore not applied in check calculations.

Temperature control of Arches

It was envisaged at one stage to investigate the strength of the arch, presuming that the mass temperature variation was restricted to $\pm 20°F$ as against $\pm 30°F$ assumed in design. Arch mass is quite thick and average mass temperature may indicate less temperature changes than those indicated by an air thermometer or a thermometer embedded close to the surface. Theoretically arch temperature can also be controlled by some type of heat insulation. As temperature has a profound influence on stress values, a substantial relief could be anticipated by reducing the temperature cycle to 2/3 of accepted value when the structure was seriously endangered due to penultimate total loss of bond of bottom steel (Reduced Section Condition).

The results obtained are quite interesting and are summarised as below :—

TABLE 18

1. *Gate Bridge Arch.* 2. *Reduced Section.* 3. *Temperature variation* $\pm 20°F$
4. *Units are* psi.

Section		*fc*				*fs*			
		Extrados		Intrados		Extrados		Intrados	
		$\pm 20°F$	$\pm 30°F$	$\pm 20°F$	$\pm 30°F$	$\pm 20°F$	$\pm 30°F$	$\pm 20°F$	$\pm 30°F$
Up-Stream Arch								Steel ineffective	Steel ineffective
C at	x=0.00	374	424	210,-150	298,-246	..	..		
VIII	4.91	382	425	227,-170	306,-249	..	..		
VI	9.06	315	334	232,-106	288,-162	..	..		
I	26.13	284	390	450,-12	666,-94	4500	9150		
O	31.00	312	417	423,-87	620,-174	5100	10300		
Down-Stream Arch								Steel ineffective	Steel ineffective
C at	x=0.00	348	421	193,-166	280,-256	..	..		
VIII	4.91	344	390	198,-145	274,-224	..	..		
VI	9.06	260	280	193,-63	250,-120	..	..		
I	26.13	258	360	430,-8	582,-74	4100	7650		
O	31.00	276	380	405,-55	600,-142	4650	9840		

TABLE 19

1. *Road Bridge Arch.* 2. *Reduced Section.* 3. *Temperature* variation ±20°F 4. *Units are* psi.

Loading	Equivalent UDL K/Ft.	Section VIII. f_c		Section I. f_c		Section I. f_s	
		—30°F	—20°F	—30°F	—20°F	—30°F	—20°F
16-ton lorry train design load $i = 15\%$	0.20	290 int.	278 ext.	690 int.	560 int.	9000 ext.	5850 ext.
Class B loading, $i = 15\%$		—133 int.	—62 int.				
Class 40 loading, single traffic lane, $i = 15\%$	0.25	306 ext. —140 int.	293 ext. —68 int.	730 int.	590 int.	9500 ext.	6130 ext.
Class A loading, single traffic lane, $i = 15\%$							
16 ton lorry train design load, $i = 50\%$	0.27	313 ext. —143 int.	300 ext. —70 int.	750 int.	600 int.	9800 ext.	6250 ext.
Class A loading on 10′ traffic lane, $i = 15\%$	0.35	325 ext. —79 int.	325 ext.	765 int.	618 int.	9800 ext.	6300 ext.
Class 40 loading, on 10′ traffic lane, $i = 15\%$	0.43	362 ext.	351 ext.	780 int.	635 int.	9800 ext	6350 ext.
Class 70 or class AA, single traffic $i = 15\%$		—159 int.	—88 int.				

The degree of relief afforded is immediately appreciated. It may also be of interest to know that *the Arch in Reduced condition is stronger in summer than in winter, and stronger during the day than during the night.*

Collapse Behaviour of the Arch

No arch has ever failed due to an isolated crack. An encastre voussoir arch may first get converted into a three-pinned arch by spread of abutments and a crack near the crown. It collapses suddenly under increased load when a fourth pin develops at the intrados, converting it into an unstable four-link mechanism from a three-pinned stable shape. The collapse strength indicated in laboratory experiments is many times the load at which the first crack appeared.

For an R.C.C. arch, appearance of an isolated radial crack will only act as a warning and will not be a sign of failure. Theoretically a thorough crack will break the continuity of the arch and will give immediate relief to the secondary effects of temperature and shrinkage. Consequently the resultant Moments will be far lower than when the arch was continuous and uncracked, and the arch may be subject to lower stresses after cracking. A sudden collapse of a reinforced concrete arch cannot therefore be envisaged. The arch has great strength by virtue of its shape.

Thinner Arch

As has been said earlier, road as well as gate bridge arch has been constructed unnecessarily thick, almost conforming to the voussoir arch outline. This unfortunately has introduced heavy secondary stresses and the arch is mostly carrying its own direct and indirect load, leaving only 10% to 15% capacity for the live load. The arch presumably would have been as strong if its thickness was reduced all over to save in materials.

An exercise was done to establish this enigmatic statement for the road as well as the gate bridge arch. The arch profile was not changed, but thickness of each section was reduced to *half* with same percentage of steel reinforcement. This would reduce arch concrete and steel quantities to half of what exist at site. The new arch would have following important features :

1. As depth of section and quantity of steel is half that of Normal Moment of inertia of any section I_1 would be $(\frac{1}{2})^3$ or $\frac{1}{8}$ of I values for a corresponding section of Normal arch.

2. Shape of S-I diagram will not change, but $\frac{\delta s}{I_1}$ would be 8 times the value of $\frac{\delta s}{I}$ as $I_1=\frac{1}{8}$ I.

3. Primary values of M and T for load will be the same, but rib-shortening corrections will be different.
4. Primary values of M and T values for Temperature and Shrinkage which are proportional to I_1 will be $\frac{1}{8}$th of those computed for Normal section, but rib-shortening effect will be different.
5. For the same thrust T, compressive stress in arch ring f_{c_1} will be $2f_c$ as $A_1 = \frac{1}{2}A$. However, rib-shortening effect is proportional to I_1. Consequently the rib-shortening moments for the thinner arch would be roughly $\frac{1}{8}\times 2=\frac{1}{4}$th of the thicker arch.

1. Road Bridge Arch

For temperature,

$$H_{c_{t_1}}=\tfrac{1}{8}H_{c_t}=\pm\tfrac{1}{8}\times 55.0=\pm 6.9 \text{ K.}$$

$$M_{c_{t_1}}=\tfrac{1}{8}M_{c_t}=\mp\tfrac{1}{8}\times 57.0=\mp 7.1 \text{ K ft.}$$

$$\text{Rib shortening effect}: f_{c_1}=\frac{\Sigma T_1}{\Sigma A_1}=\frac{\frac{1}{8}\Sigma T}{\frac{1}{2}\Sigma A}=\tfrac{1}{4}f_c$$

$$=\tfrac{1}{4}\times 8.6^\circ=2.2^\circ\text{F fall of temp.}$$

The value after correction will be $\dfrac{30.0-2.2}{30.0}$ or 92.7%, as against 71% for the thicker arch.

The result for the thinner arch make an interesting reading when compared to the thicker arch values. The section properties of the thinner arch are as below :—

TABLE 20

Section.	x	d_1 Depth	A_1 Area	I_1 M. I.	Z_1 Section Modulus	P_1 %age Steel
C	0.00	1.00	1.62	0.15	.031	4.4
I	27.75	1.66	2.28	0.66	0.80	2.6
O	31.00	2.20	2.82	1.43	1.30	2.0

A summary of Moment and Thrust values is given in Table 21. These values have been corrected for rib-shortening effect.

TABLE 21

Road Bridge Thin Arch. Units are ft. and kips.

Section	Dead load up to RL 213.0		Dead load UDL 0.338K per foot on whole span		Total Dead load.		Live UDL of 0.2K per ft. on left half		Temperature Rise +30°F		Temperature Fall −30°F		Shrinkage.	
	M	T	M	T	M	T	M	T	M	T	M	T	M	T
C at $x=0$;	3.9	28.6	6.2	24.0	10.1	52.6	1.9	7.2	−6.6	6.4	6.6	−6.4	3.3	−3.2
I at $x=27.65$	−3.1	32.1	3.3	25.7	0.2	57.8	R11.4	7.2	25.1	6.4	−25.1	−6.4	−12.6	−3.2
							L−9.6	8.4						
O at $x=31.0$	−1.1	34.1	23.7	26.1	22.6	60.2	R24.5	7.2	43.6	6.4	−43.6	−6.4	−21.8	−3.2
							L10.7	8.8						

Maxima Moments and corresponding thrusts. Road Bridge Arch, Normal and halved sections, under identical loads and temperature conditions :—

TABLE 22

1. *Units are feet and kips.* 2. *Identical Design load and Temperature conditions.*

	(Dead+Live) Loads				Loads+Temperature.				Loads+Temperature+Shrinkage			
Section	M	T	*e*	*e/d*	M	T	*e*	*e/d*	M	T	*e*	*e/d*
C + M	12.0	59.8	0.20	0.20	18.6	53.4	0.35	0.35	21.9	50.2	0.44	0.44
− M	...	..	..	..	..	..	..	..	..	..	..	..
I + M	11.6	65.0	0.18	0.11	36.7	71.4	0.51	0.31	24.1	68.2	0.35	0.21
− M	−9.4	66.2	−0.14	−0.08	−34.5	59.8	−0.58	−0.35	−47.1	56.6	−0.83	−0.50
O + M	47.1	67.4	0.70	0.32	90.7	73.8	1.23	0.56	68.9	70.6	0.98	0.45
− M	...	...	..	..	−31.7	62.6	−0.51	−0.23	−53.5	59.4	0.90	−0.41

TABLE 23

Section	Loads				Loads+Temperature				Loads+Temperature+Shrinkage			
	Normal Section		Halved Section		Normal Section		Halved Section		Normal Section		Halved Section	
	M	T	M	T	M	T	M	T	M	T	M	T
C + M	21.0	50.8	12.0	59.8	61.7	11.6	18.6	53.4	82.1	—8.0	21.9	50.2
— M	..	...	..	..	—22.1	83.9	..	..	—1.7	64.3	..	..
I + M	..	...	11.6	65.0	129.8	95.1	36.7	71.4	52.5	75.5	24.1	68.2
— M	—45.6	57.1	—9.4	66.2	—200.1	17.9	—34.5	59.8	—277.4	—1.7	—47.1	56.6
O + M	...	..	47.1	67.4	252.3	97.5	90.7	73.8	118.3	77.9	68.9	70.6
— M	—50.9	59.9	..	..	—318.9	20.7	—31.7	62.6	—452.9	1.1	—53.5	59.4

Maximum $\pm$M values and corresponding values of T. e, e/d as worked out from Table 19 are given in Table 22.

Comparison of values in Table No. 22 with those given in Table No. 6 brings out strikingly the relief afforded by sharp reduction in secondary forces. The values are tabulated in Table 23 for immediate appreciation of the results.

Computed comparative stress values in the two arches are also tabulated below :—

TABLE 24

Maxima Stresses

Units are psi.

Section		Load + Temperature				Loads + Temperature + Shrinkage			
		Normal Section		Halved Section		Normal Section		Halved Section	
		f_c	f_s	f_c	f_s	f_c	f_s	f_c	f_s
C	Intrados	292	6050	Low	1410	151	7500	Low	2620
	Extrados	370	..	610	..	366	..	655	...
I	Intrados	562	2360	518	1170	608	...	645	..
	Extrados	400	11500	560	1680	230	15200	417	4600
O	Intrados	595	5150	324	7350	665	1050	480	4120
	Extrados	530	13400	770	..	278	18300	608	2770

It may be immediately seen that the thinner road arch, having half the concrete and steel of the thicker arch, is stronger of the two and is developing lower stress, under identical load and temperature conditions.

2. Gate Bridge Arch

The procedure adopted is the same as discussed for the road bridge arch. Details are omitted and only a comparative summary of moments, thrusts and stresses is given in Tables 25 and 26 for comparison.

TABLE 25

Gate Bridge Arch—Maxima Moments and Corresponding Thrusts

1. *Units are kips and feet.* 2. *Identical loads and temperature*

Point	Loads				Loads+Temperature				Loads+Temperature+Shrinkage			
	Existing Section		Halved Section		Existing Section		Halved Section		Existing Section		Halved Section	
	M	T	M	T	M	T	M	T	M	T	M	T
C + M	19.8	49.5	8.2	56.1	71.5	26.1	15.7	51.8	97.0	12.1	19.3	49.7
− M	..	..	..	..	−29.1	77.4	..	..	−4.7	63.4	..	..
I + M	..	..	12.5	67.5	124.0	88.8	35.0	71.8	40.9	75.8	24.3	69.7
− M	−54.4	63.2	−19.1	69.8	−201.8	35.3	−41.6	65.5	−285.5	20.9	−52.3	63.4
O + M	..	..	46.0	74.3	238.5	95.7	83.0	78.6	112.1	81.7	64.5	76.5
− M	69.6	70.2	−9.2	76.8	−322.4	42.3	−46.2	72.5	−448.8	28.3	−64.7	70.4

TABLE 26

Gate Bridge Arch—Maxima Stresses

1. Units are lbs. and inches

Point		Loads + Temperature				Loads+Temperature +Shrinkage			
		Existing Section		Halved Section		Existing Section		Halved Section	
		f_c	f_s	f_c	f_s	f_c	f_s	f_c	f_s
C	Intrados	290	6550	Low	..	157	11400	Low	1150
	Extrados	418	..	495	..	530	..	550	..
I	Intrados	593	2580	575	Low	790	..	690	..
	Extrados	422	13400	506	2480	192	19500	405	5150
O	Intrados	652	5360	415	5600	855	Low	550	2860
	Extrados	490	16100	680	Low	250	22200	555	3530

The results are too eloquent to need any further explanation. A *thinner arch having half the thickness of section and half the quantity of steel of the existing arch is superior and stronger.* Steel can even be reduced further as stresses indicated are quite low.

Conclusion

Structural investigations of Sukkur Barrage Concrete Arches have been undertaken at some length and many interesting results have come to light. Due to lack of space, important results have been discussed only briefly in this paper. The arches do not stand in immediate need of replacement and may be rendered safe by suitable repairs. It may, however, be appreciated that the investigations point forcefully to the need for specialisation in design and the necessity of a Centralised Design Office. The mistakes and omissions in the 1928 design of Sukkur Barrage Arches are principally due to the fact that it was an individual effort without expert direction and review. Towards the end of the execution of the Sukkur Barrage and Canals Project, the need for standardisation of works on various canals was acutely felt and an executive engineer was attached with the Superintending Engineer for design and planning. When the Sutlej Valley Project was launched in the former Punjab, a Central Design Office was created at Lahore to standardise design and planning and to achieve a high level of efficiency consistent with economy and sound engineering. This institution steadily grew and became the training centre and brain trust of the Irrigation Departments. During re-organisation of the department in 1962, this office was unfortunately abolished and design offices have been created in the regional offices of chief engineers. This step to liquidate the Central Design Office at Lahore after more than 30 years of its unmatched performance was putting the clock back in this age of specialisation and specialised consultants. Many mistakes out of ignorance and lack of thoroughness may be anticipated from the poorly-staffed and poorly-directed regional design offices where desk assignments are not given any recognition. The drift to engage foreign consultants and to let them make our policies and sensitive technical decisions is fraught with danger. It may paralyse our mental capabilities and distort our professional growth.

It is also essential that suitable attractions be offered to promising brains in the Department to handle difficult and exacting desk assignments. There is no dearth of talent in our country and we can decidedly match the best in the world. What is generally lacking is encouragement and direction, trust and recognition, confidence and foresight, and this is causing a steady drain and paralysis of the best we have. The Congress may do well to take an immediate stock of the worsening situation.

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