ASSESSMENT AND LOAD RATING OF MASONRY ARCH BRIDGES

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ABSTRACT

Masonry arch bridges exist in large numbers in the transport network of many countries. The majority of these bridges were built/designed for carrying loads far less than they are carrying today. Different methods of assessment of these bridges exist. The Finite Element Method (FEM) is extensively used these days in all fields of civil engineering. The use of the FEM in analyzing complex structures like masonry arch bridges has been found to provide good assessments. A two-dimensional Finite Element analysis of masonry arch bridges ignores the transverse effects but the behaviour in the span direction can be effectively modeled. In view of this a masonry arch bridge has been analyzed using a two-dimensional non-linear finite element method computer program that has been developed. A three dimensional nonlinear finite element analysis of the same bridge has been carried out using commercially available general-purpose finite element analysis software, using the inbuilt material models and failure functions. Comparison of the two sets of results indicates the suitability of the two-dimensional analysis for the purpose of load rating. The displacements along the span obtained through a shift in the position of the load in the transverse direction do not significantly vary the magnitudes of the maximum displacements at different locations along the span. A significant variation in the transverse direction was indeed observed. The variation in the transverse direction can seriously affect the carrying capacity of the bridge, if the load is placed on the edge, due to cracking and separation of spandrel walls. The displacements observed under service loads are a lot less than those under collapse loads. Hence for overall assessment, a three-dimensional finite element analysis cannot be ignored, but for a quick load rating a two dimensional finite element analysis is sufficient. The details of the investigation are reported here in the paper.

KEYWORDS: bridge, load carrying capacity, non-linear FEA.

INTRODUCTION

Experiments on masonry arch bridges both in the laboratory as well as in field tests give a clear indication of failure taking place due to the formation of a mechanism in the longitudinal direction. Hence, the load carrying capacity of masonry arch bridges is examined in the longitudinal direction by existing assessment methods such as the MEXE method [1], the mechanism method [2] and Castigliano's method [3]. In order to understand the complete behaviour of complex structures like masonry arch bridges, a three-dimensional finite element analysis is an obvious choice where the transverse effects may play an important role. The issues related to the modelling of masonry arch bridges can be handled in the perspective of the type of response under study. In general, finite element modeling is more appropriate for studying the behaviour under the following two categories.

- To determine the collapse load and
- To study the service load response;

From existing literature [4, 5] and the experience of the investigators it is clear that modeling to determine the collapse loads must be three-dimensional. The summary of load tests to collapse on series of ten masonry arch bridges [6] clearly indicates different modes of failures. Hinge formation was clearly noticed in the majority of cases along with the other mode of failure. The separation of the spandrel walls and a significant stiffening effect due to their presence is apparent from the difference in deflections measured at the crown over central and edge locations. However, the spandrel walls usually split from the main arch ring before the ultimate load is reached. Hence, neglecting three-dimensional effects would have little effect on the ultimate strength of the bridges.

On the other hand, under service loads [7], the response of masonry arch bridges is almost linear and these produce unusually small displacements. The intensity of the live load in comparison to the dead weight of the structure is small, so the additional stresses induced by the live loads are not excessive, especially in cases where the depth of the fill on the crown is large. Hence for the purpose of studying the service load response, as well as for rating purposes, three-dimensional effects may be ignored. This adds to the conservatism in the analysis.

In the present study, for comparison purposes two and three dimensional finite element analyses of one of the masonry arch bridges tested in the field (hence, full scale) were performed. The paper presents a comparison of the two-dimensional and three-dimensional modeling results for the analyses of the masonry arch bridge. The two-dimensional as well as the three-dimensional formulations take into account the material non-linearity due to cracking of the masonry and due to the non-linear stress-strain relationship of the masonry. Brief details of the two-dimensional and three-dimensional modeling are also presented.

THREE - DIMENSIONAL MODELLING

The three-dimensional non-linear modeling of the masonry arch bridges was performed using the commercially available finite element analysis package ANSYS 7.1 [8]. Each part of the bridge arch barrel, spandrel walls and fill was modeled separately using inbuilt elements available. The Solid65 element was used, a three dimensional eight noded isoparametric brick element having three degrees of freedom at each node - translations in the nodal x, y and z directions. The most important aspect of this element is the treatment of nonlinear material properties and the ability to model cracking. The material non-linear stress-strain relationship is input directly by providing the magnitudes of the stresses corresponding to the strain levels. The linear elastic properties of the materials are directly input associated with the type of element. This element allows for the formation of cracks perpendicular to the direction of principal stresses that exceed the tensile strength of the material. A smeared crack approach is utilized. The masonry is modelled as a continuum with appropriate compressive and tensile strengths assigned, based on the results of experiments on prisms of the masonry. Cracking in the material is allowed in all three directions perpendicular to the principal stress directions. The presence of a crack at an integration point is represented through modification of the stress strain relation by introducing a plane of weakness in the direction normal to the crack face. A shear transfer coefficient is also introduced which represent a shear strength reduction factor for those subsequent loads, which induce sliding across a crack face. Crushing at an integration point is said to take place if the compressive stress at any integration point exceeds the uniaxial, biaxial or triaxial strength of the material. This is defined by complete deterioration of the structural integrity of the material. Under such circumstances the material strength is assumed to have degraded to such an extent that the contribution of the stiffness of that element at the integration point in question is completely ignored. The fill can be modelled using the same elements or solid 45 elements. A Drucker-Prager material law is used for the fill material.

TWO - DIMENSIONAL MODELLING

For the two dimensional modeling, a non-linear finite element analysis program has been developed in Fortran. An eight noded isoparametric element with two degrees of freedom at each node has been used in the present formulation. Since in the analysis units and mortar are not modeled separately, average masonry properties have been used. Thus the representative stress strain or constitutive relationship obtained from the experimental results [9] under uniaxial compression has been used in the analysis.

$$\frac{\sigma}{\sigma_m} = 2.1599\left(\frac{\varepsilon}{\varepsilon_m}\right) - 1.4935\left(\frac{\varepsilon}{\varepsilon_m}\right)^2 + 0.3241\left(\frac{\varepsilon}{\varepsilon_m}\right)^3$$

Equation 1

where $\sigma_m = 5.841$ MPa and $\varepsilon_m = 0.0030$

Under uniaxial states of stress, failure criteria in tension as well as compression on the basis of principal stresses have been adopted. Shear failure [10] in the masonry has been adopted as per the Mohr- Coulomb theory.

$$\tau_m = V_{bs} + \mu\sigma_y$$

Equation 2

where τ_m is the shear strength of the masonry, V_{bs} is the shear bond strength at zero pre-compression, μ is the coefficient of friction between the units and mortar and σ_y is the normal pre-compression in the masonry.

For biaxial compression-compression [11], a Von-Mises failure criterion has adopted.

$$\left[\frac{\sigma_1}{\sigma_c}\right]^2 - \frac{\sigma_1\sigma_2}{\sigma_c^2} + \left[\frac{\sigma_2}{\sigma_c}\right]^2 \geq 1$$

Equation 3

Similarly, in tension-compression [12] a simplified failure surface approximated by a straight line has been used in the two-dimensional finite element model.

$$\frac{|\sigma_1|}{\sigma_{t\theta}} + \frac{|\sigma_2|}{\sigma_c} \geq 1$$

Equation 4

where $|\sigma_1|$ and $|\sigma_2|$ are numerical values of the principal tensile and compressive stresses respectively, σ_1 and σ_2 are the two principal compressive stresses, $\sigma_{t\theta}$ is the tensile strength at an angle θ with the bed joint and σ_c is the uniaxial compressive strength of the masonry.
Modifying the stiffness in the direction perpendicular to the cracks by suitable reduction factors modifies the constitutive matrix following any of these failure criteria [9]. The convergence criterion on the norm of residual nodal forces in vertical directions is compared with the norm of the applied load in the same direction.

GEOMETRY OF THE BRIDGE

Analysis of a masonry arch bridge with the dimensions given below in Table 1 has been carried out using the developed two-dimensional non-linear finite element computer program, NLAM. For two-dimensional analyses, loading from the wheels of the front and rear axles were dispersed widthwise as per the IRC (Indian Roads Congress) code of practice [11] and a distributed load corresponding to a unit width of the arch barrel was applied. The load so computed was then dispersed longitudinally and applied as a pressure loading on the relevant nodes.

In addition, three-dimensional analysis of the bridge with the same geometry and the material properties was also undertaken using the general-purpose software ANSYS 7.1. The bridge was analysed under standard IRC [13] class AA wheeled loading as shown in Figure 1. For three-dimensional analysis the front axle was positioned at the crown and the rear axle at a distance 1.2 m from the crown. The analysis has been carried out for two load positions; (i) by placing the load symmetric with respect to the centreline of the arch barrel as shown in Figure 2 and (ii) by placing the load at the maximum transverse eccentricity as per the IRC code of Practice as shown in Figure 3. The discretization of the bridge and the placement of the loads are shown in Figure 4.

Table 1 - Principal Dimensions of the Bridge

Description	Dimension
Span (Square) mm	9425
Rise at midspan (mm)	2990
Arch thickness at crown (mm)	600
Arch thickness at springing (mm)	600
Arch Shape	Segmental
Arch Material	Stone Masonry
Spandrel Wall Thickness (mm)	450
Total Width (mm)	5810
Fill Depth At Crown (mm)	410

MATERIAL PROPERTIES USED IN THE ANALYSIS

The material for the arch barrel was stone masonry. The properties of the stone masonry used in the analysis are given in Table 2. Fill over the arch barrel in practice varies from being well-graded granular material to compacted clay. For the present case, fill was assumed to consist of hard clay. The properties used in the analysis are shown in Table 2.

Table 2 - Material Properties used in the Analysis

Property	Arch Barrel	Fill
Young's Modulus (N/mm^2)	5000	50
Poisson Ratio	02	02
Density (N/mm^3)	22×10^{-6}	20×10^{-6}
Compressive strength (N/mm^2)	15	-
Tensile Strength (N/mm^2)	1.5	-
Cohesion (N/mm^2)	-	0.5
Friction Angle (N/mm^2)	-	32

All dimensions in mm

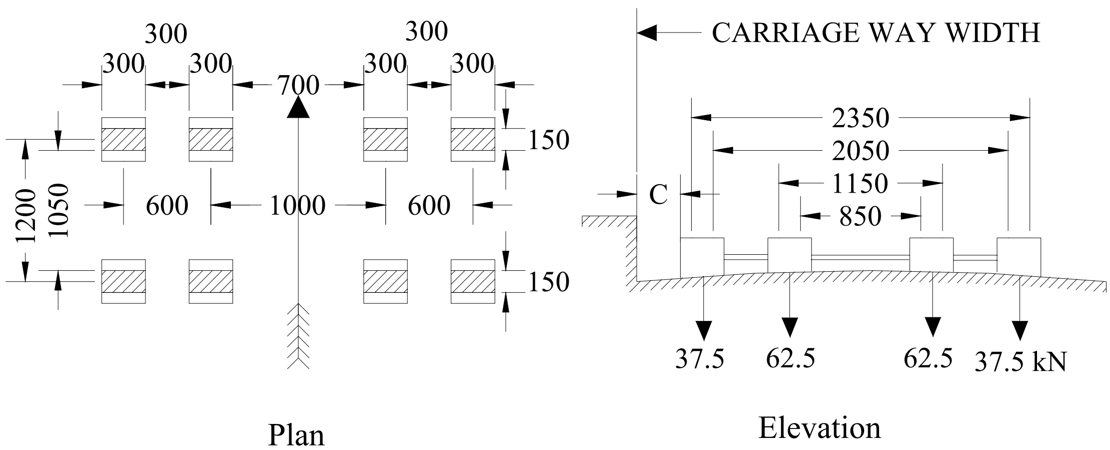

Figure 1 – IRC Class AA Wheeled Vehicle Loading

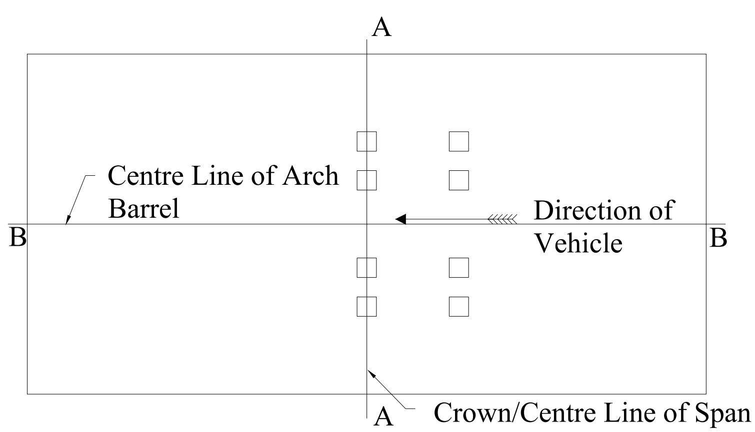

Figure 2 – Central Placement of IRC Wheeled Vehicle with Front Axle on Crown

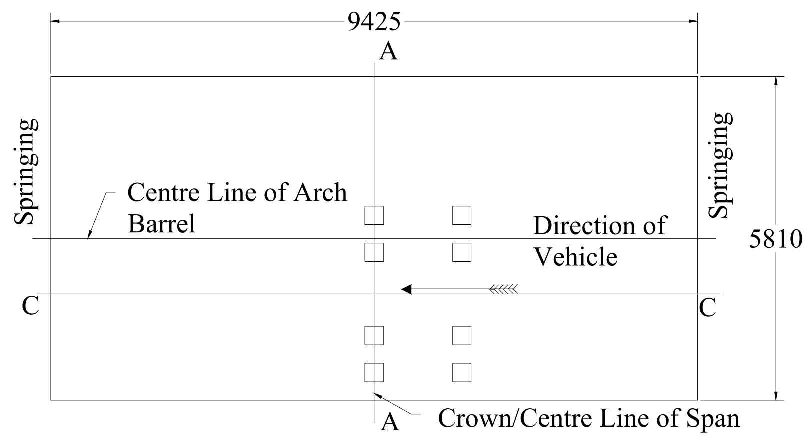

Figure 3 – Eccentric Placement of IRC Wheeled Vehicle with Front Axle on Crown

COMPARISION OF TWO-DIMENSIONAL AND THREE-DIMENSIONAL ANALYSIS

The contour plots showing the variation of the stress σ_x under IRC class AA wheeled vehicle obtained from the three-dimensional analysis are shown in Figure 5. The localized stresses are induced in the fill under the concentrated loads applied over the nodes on the fill. The stresses induced in the arch barrel are tensile on the intrados under the crown and compressive in the remaining portions of the arch barrel.

The variations of the vertical deflections at the midspan section along the line AA on the intrados of the arch barrel due to the two placements of the IRC load considered, obtained from the three-dimensional analysis are plotted in Figure 6. The results indicate that the difference in maximum deflections is only marginal but there is a shift in the position of the maximum deflection in the transverse direction. The peak deflection under the centrally placed load is 1.32 mm whereas the maximum deflection under the eccentrically placed load is 1.35 mm.

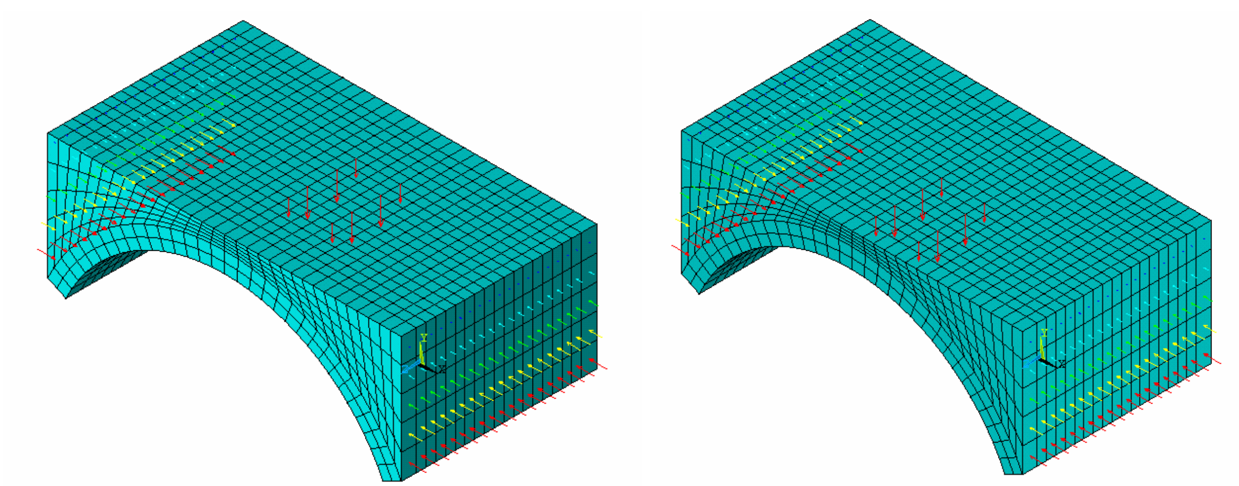

Figure 4 - Placement of IRC Class AA Loading Centrally and Eccentrically on the three-dimensional Model

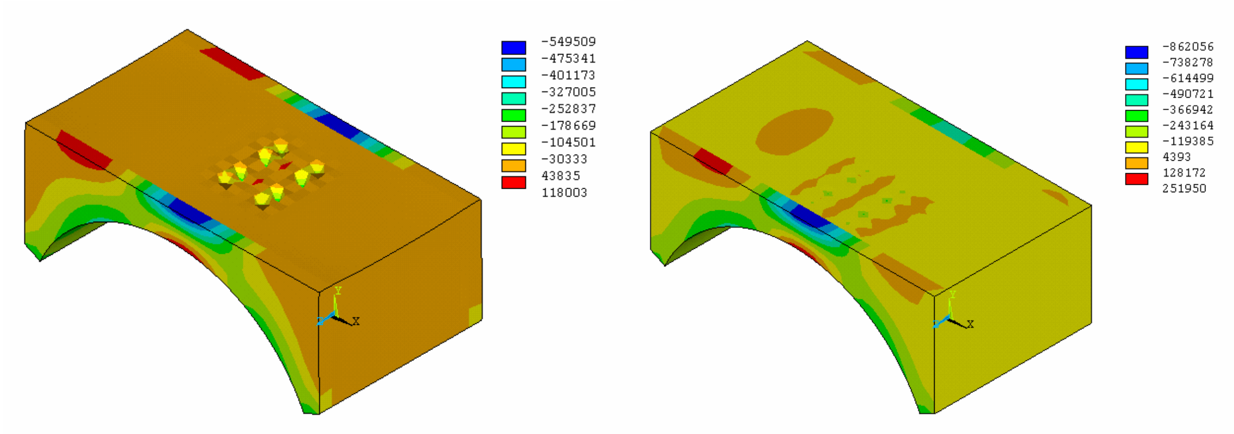

Figure 5 - Stress in X-direction under Centrally and Eccentrically Placed Load

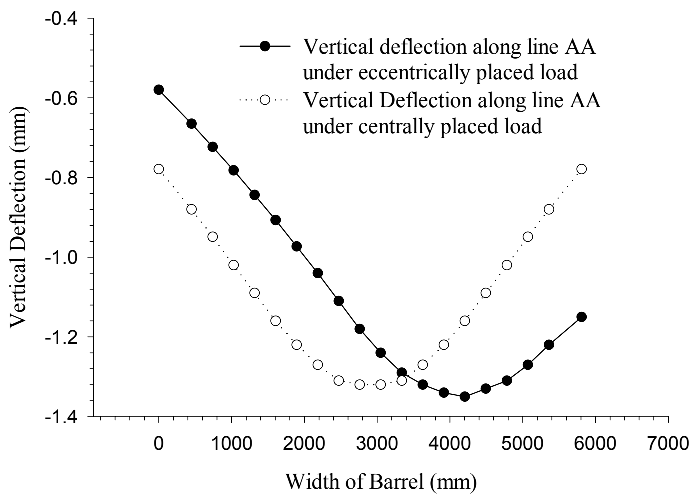

Figure 6 - Comparison of the Vertical Deflection Across the Arch Barrel at the Midspan under Centrally and Eccentrically Placed Loads

The plot of the Cartesian stress σ_x across the width of the arch barrel at midspan (along the line AA) is presented in Figure 7. Under the centre of the centrally-placed vehicle, σ_x is -0.023 MPa, but the stress is +0.012 MPa when the load is placed eccentrically. There is significant variation in σ_x along line AA. The maximum stress developed is 0.094 MPa under the centrally placed load and 0.21 MPa under the eccentric load, on the edge of the arch barrel as can be seen in Figure 7. Thus, these variations indicate the magnitude of the transverse effect induced.

In Figure 8 the vertical deflections along lines BB under the centrally placed vehicle and along line CC under the eccentrically placed vehicle are plotted. The variations along the span are similar as can be seen.

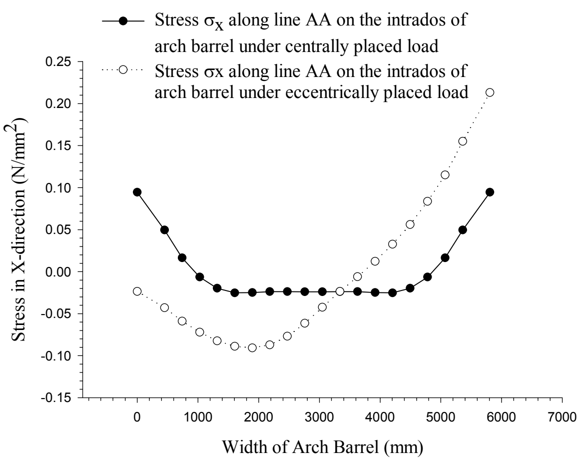

Figure 7 - Comparison of Stresses in the X direction along the width of the arch barrel under centrally and eccentrically placed load.

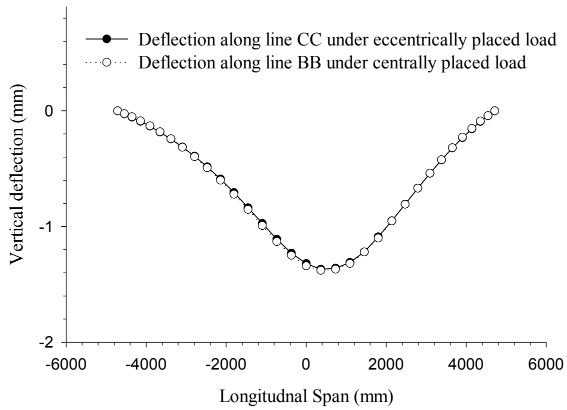

Figure 8 - Comparison of the Vertical Deflection under Centrally and Eccentrically Placed Loads along the Span of the Arch Barrel

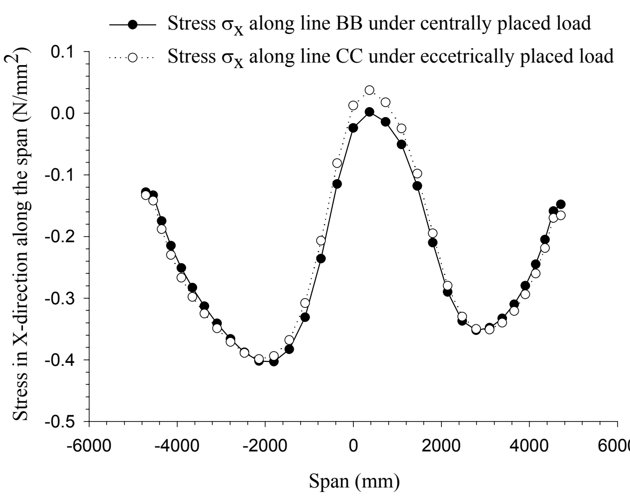

Figure 9 - Comparison of Stresses in the X-Direction Along the Span of the Arch under Centrally and Eccentrically Placed Load

Figure 9 shows the plot of σ_x along the span of arch barrel under the two locations of the vehicles.

From the plot showing the variation of deflections (Figures 6 and 8), it is evident that the vertical deflections produced are of almost same magnitude, along the span of the arch, irrespective of the placement of the loads. It is evident that there is merely a shift in the position of the peak deflections along the width of arch barrel. Also, since, the magnitudes of stresses produced are small, for the purpose of comparison of the two-dimensional and three-dimensional analyses, only the central placement of the loads has been considered. The vertical deflections obtained from the two- and three-dimensional analyses are plotted along the span for the arch bridge with the same geometry and the material properties in Figure 10. It is observed that the deflections obtained through the two-dimensional analysis are of almost same magnitude as those obtained from the three-dimensional analysis.

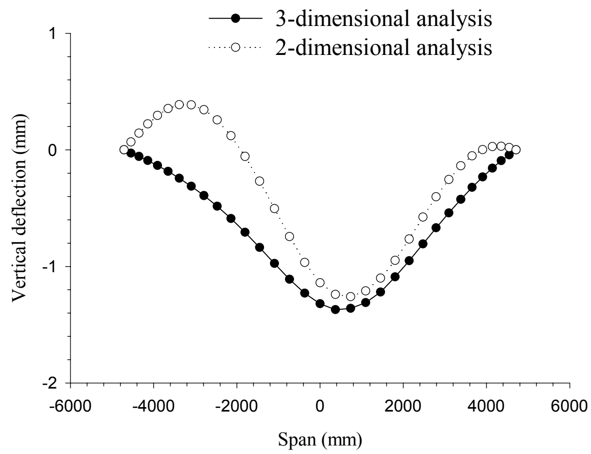

Figure 10 - Comparison of Vertical Deflection of the Intrados along the Span for a Centrally Placed IRC Wheeled Vehicle

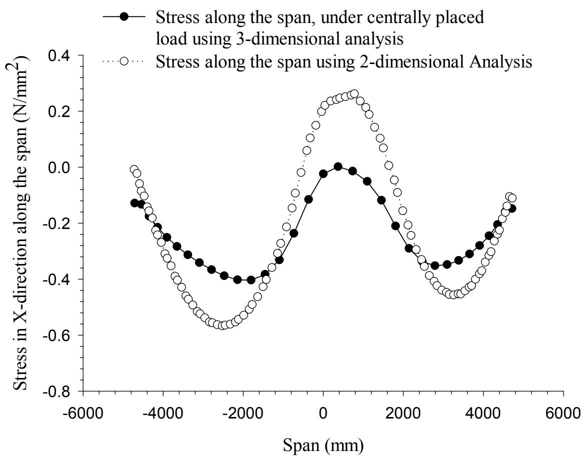

Figure 11 - Comparison of Stress in the X-direction along the Span using three-dimensional and two-dimensional Analyses

The maximum peak deflection under the centre of the vehicle obtained through two-dimensional analysis is 1.26 mm and from three-dimensional analysis is 1.38 mm. Similarly, on the quarter point opposite to the loaded side, the peak deflection obtained using two-dimensional analysis is 0.38 mm against a value of –0.23 mm obtained through three-dimensional analysis. A number of arch bridges tested to destruction as well as under service conditions also depict a similar type of deflected shape of the arch barrel as is obtained using a two dimensional finite element analysis, NLAM.

Similarly, Cartesian stresses in the X-direction along the span are plotted in Figure 11 to compare the stresses obtained from the two- and three-dimensional analyses. The stresses obtained using two-dimensional analysis are within an acceptable range compared to those from the three-dimensional analysis as the magnitude of the stresses developed is small.

Since the vertical deflection and the stress in the span direction are more important for the load rating of masonry arch bridges, only these two parameters have been compared. On the basis of comparison of the deflections and the stresses, and to reduce the complexity of the problem, without compromising the accuracy of the results, a two-dimensional analysis can be recommended for the purpose of load rating and studying the response under the service loads.

CONCLUSIONS

The non-linear two-dimensional finite element computer program, NLAM [10] developed the material non-linearity, which is quite important for the load rating analysis of the masonry arches. The proposed formulation takes into account the non-linearity due to stress-strain behaviour of the material as well as the cracking and crushing of the material.

- On the basis of the comparison of the analyses conducted on a sample arch bridge, the necessity of conducting two-dimensional or three-dimensional analysis has been investigated. It is obvious that the transverse effects are neglected in two-dimensional analysis. The magnitudes of the deflections obtained through the two-dimensional analysis are quite comparable to those obtained from the three-dimensional analysis. Further, the magnitudes of deflections observed in two-dimensional as well as three-dimensional analysis are very small in magnitude as compared to those available in the literature, from full scale tests on masonry arch bridges.
- It has been observed that under service loads, the deflections under the load points while placing the loads centrally and eccentrically on the arch barrel along the width merely shifts the position of the peak deflections and the peak stresses along with the placement of the load. Under centrally and eccentrically placed loads there is an insignificant difference in the peak deflections and the stresses developed, indicating a similar stiffness rendered by the fill in comparison to that rendered by the spandrel walls under the service loads.
- The overall response exhibited through two-dimensional and three-dimensional analysis is similar but the stress variation obtained from two-dimensional analysis is conservative in comparison to those of three-dimensional analysis. The peak stress at the crown from two-dimensional analyses is 0.26 MPa against 0.037 MPa obtained from three-dimensional analyses. Similarly, the peak stresses between quarter point and left abutment obtained from two-dimensional analyses is 0.56 MPa against 0.39 MPa obtained from three-dimensional analyses. The peak stresses from the three-dimensional analysis are probably lower due to the dispersion of the loads in the transverse and longitudinal directions.
- The non-linearity in the material models is undoubtedly significant in the evaluation of the capacity of masonry arch bridges. The results of the tests conducted on full-scale models indicate the initiation of non-linearity in even the lower ranges of the loads. These may be induced by the cracking and crushing of the units and the mortar. Hence, in the context of the masonry, due to its very low tensile strength, the inclusion of non-linearity in material behaviour in the analysis is relevant.

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