# Heavy-Vehicle Loading of Arch Structures of Corrugated Metal and Soil

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**Results of measurements of both the direct and the residual effects of heavy-vehicle loading of an arch with a 12-m (39-ft) span of corrugated metal and soil constructed as an overpass are presented. Both in-plane stress and vertical deflection of the arch are reported for 70-tonne (77-ton) and 290-tonne (319-ton) trucks positioned at various locations over the arch. The residual effects are shown in terms of a graph of cumulative deflection versus construction and service history. The heavy vehicles caused considerably more permanent deflection of the arch crown than did the 2.1 m (6.9 ft) of soil cover above crown level.**

Overpasses and bridges associated with the mining industry are important applications for arch structures of corrugated metal and compacted soil. Low-cost fill materials are generally available and these, in conjunction with the low cost of supply and transportation for the metal components, make the economics extremely attractive. However, an unusual feature associated with the design for many mining applications is the heavy-vehicle loading. This was the case for a corrugated-steel overpass constructed at Leigh Creek, South Australia, in conjunction with an open-cut coal-mining operation for the Electricity Trust of South Australia.

The structure was designed by Coffey and Partners Proprietary, Ltd., for continuous use by 70-tonne (77-ton) and occasional use by 290-tonne (319-ton) coal haulage trucks (gross weights). Detailed measurements of stresses and deformations were taken by the University of Adelaide at two sections of the arch during the construction period and for some time after. In addition, measurements were taken of the static live-loading effects of both vehicles. The influence of the intermittent heavy-vehicle loading on the long-term deflection response appears to be considerable. The live-loading effects are the subject of this paper.

## BRIEF REVIEW OF PROJECT DETAILS

Details of the construction, the instrumentation, and some results of measurements have been published previously (1). In brief, the structure as shown in Figure 1 had a span of 11.8 m (39 ft), a rise of 4.7 m (15.4 ft), and a fill height above the crown of 2.1 m (6.9 ft). The initial shape consisted of circular arcs that approximated an ellipse. The steel was supported on either side by retaining walls 3 m (9.8 ft) high tied back to anchorages in the side fills. The plate was 7 mm (0.28 in) thick and had 150x50-mm (6x2-in) corrugations. Steel strains and deflections relative to the retaining walls were measured at two sections (the one-third points) with 150x50-mm corrugations along the length of the 26.7-m (87.6-ft) long steel arch. The spacing of the retaining walls was measured, but no relative horizontal movements were detected.

Fill construction above the springline level began about November 1, 1978, and reached 1.9 m (6.2 ft) above the crown on November 25, 1978. However, completion of the road base to 2.1 m above the crown level was not accomplished until January 3, 1979. Use of the structure for haulage by the 70-tonne loaded haulage trucks began a few days prior to that time.

Measurements of live load response associated with the 70-tonne loaded trucks were conducted on January 3, 1979. After continuous use of the structure for haulage by these same vehicles, the first use of the structure by a 290-tonne loaded truck was monitored on July 25, 1979.

## 70-TONNE TRUCK LOADING

The relative locations of the 70-tonne loaded truck for the live-load tests are shown in plan in Figure 2. The truck facing west was stopped at four positions on a line such that one set of wheels was always immediately over the southernmost instrumented section. On each occasion strain readings were taken at seven locations around the steel arch. Strain values were converted to stress based on a Young's modulus value for steel of 200 GPa ($29\times10^6$ psi) and results are plotted in Figure 3. Values shown represent average stress levels across the section in that gauges were located near the neutral axis on both sides of the steel and were connected in series. They are related to zero stress levels at all stations both before and after the truck movement on the bridge. No readings were registered on the gauges at the northern end of the structure.

Deflection measurement was attempted by the conventional level-surveying procedures, but no valuable readings were obtained. Deflections occurred but were apparently less than 1 mm (0.04 in). A second series of measurements was taken with the truck on the same line but in the opposite direction. The resulting stresses are plotted in Figure 4. No residual stresses were observed in either series.

## 290-TONNE TRUCK LOADING

Dimensions associated with the 290-tonne loaded-truck tests are indicated in Figure 5. In view of the earlier inadequacies of surveying methods, vertical deflections were measured in the later truckload tests by electrical displacement transducers fixed to vertically aligned telescopic rods. Steel stresses associated with the various positions are shown in Figure 6 and measured vertical deflections in Figure 7. A repeat test was done at one location only, with the truck facing the same direction (east), and these additional results are shown in Figure 6d and 7d, respectively. In the case of the first test a permanent deflection of 0.5 mm (0.02 in) remained at the crown. No residual deflection could be detected for the repeat test. Likewise no residual stresses were observed.

## LONG-TERM DEFLECTION OBSERVATIONS

A summary of permanent deflections versus time is given in Figure 8. All permanent deflections measured after the fill reached the crown level, including effects of truck loading, are shown in a cumulative fashion.

Of major interest is the fact that only 30-40 percent of the total permanent deflection appears to have been associated with placement and compaction of the fill. The major part of the permanent deflection was caused by the heavy-vehicle loading of

Figure 1. Cross section through structure.

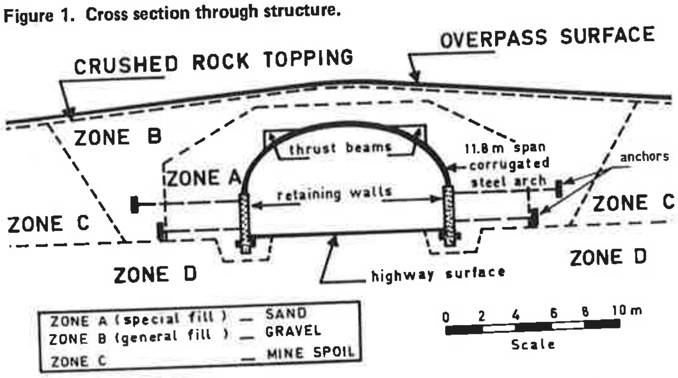



Figure 2. Relative locations of 70-tonne loaded truck.

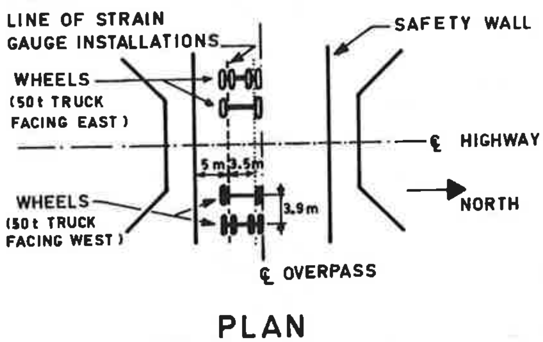
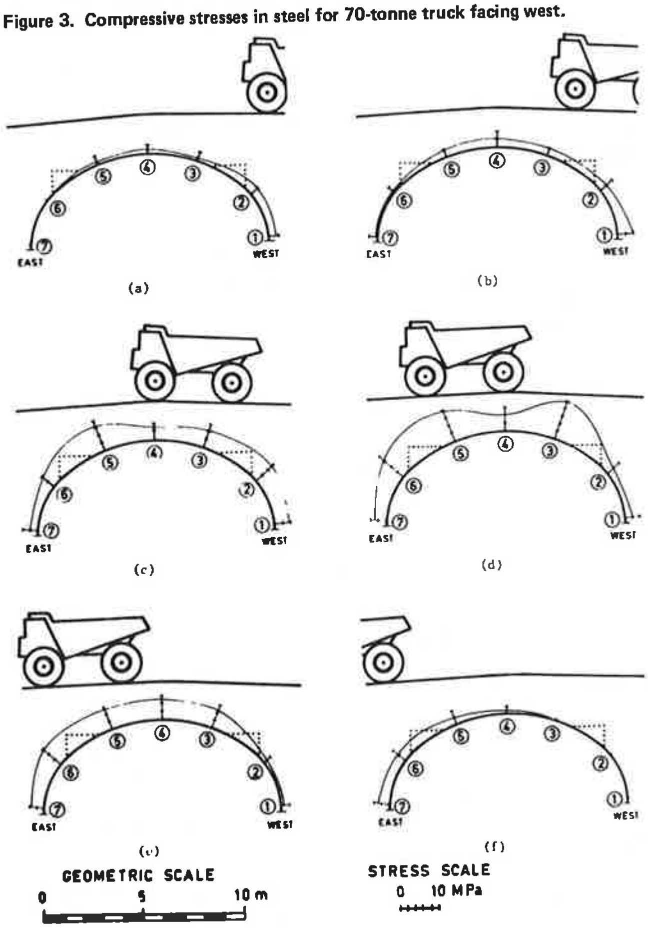



Figure 3. Compressive stresses in steel for 70-tonne truck facing west.




the 70-tonne trucks. Although it is possible that little further deflection may be caused by these vehicles, the effect of the first loading by the 290-tonne truck in producing a total deflection of 5 mm (0.2 in) and a residual deflection of 0.5 mm leaves little doubt that further permanent deflection would be caused by the bigger trucks.

When the weight of the vehicles that pass over the in-service structure is many times that of the equipment used in the compaction process, it is not surprising that additional settlement occurs. Further compaction of the soil in the upper meter is likely. However, the major cause of the additional permanent deflection of the steel arch is probably associated with the concentrated loading applied to the crown area. The profiles of stress shown in Figures 3, 4, and 6 and the deflections shown in Figure 7 appear to bear this out. When the vehicle rear wheels are over the crown, the stress levels at stations 3 and 5 are considerably higher than stress levels at stations 2 and 6. Also, the temporary vertical deflection at the crown is very much greater than that at the thrust-beam locations. The vertical load at the crown apparently causes primarily horizontal movement at the thrust beam and part of this movement associated with initial load

Figure 4. Compressive stresses in steel for 70-tonne truck facing east.

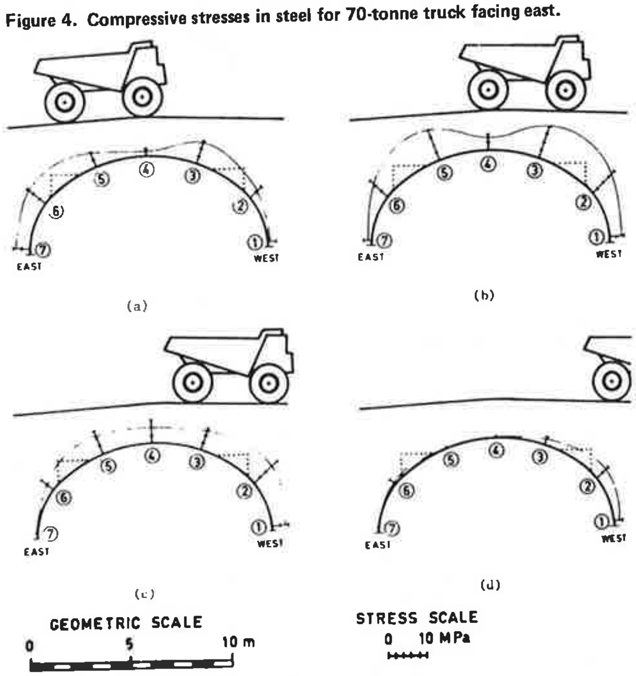



Figure 5. Relative location of 290-tonne loaded truck.

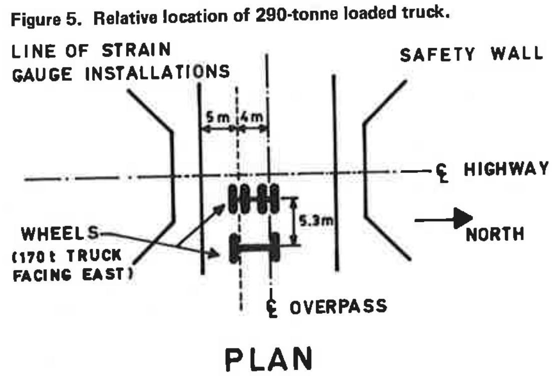


Figure 6. Compressive stresses in steel for 290-tonne loaded truck.

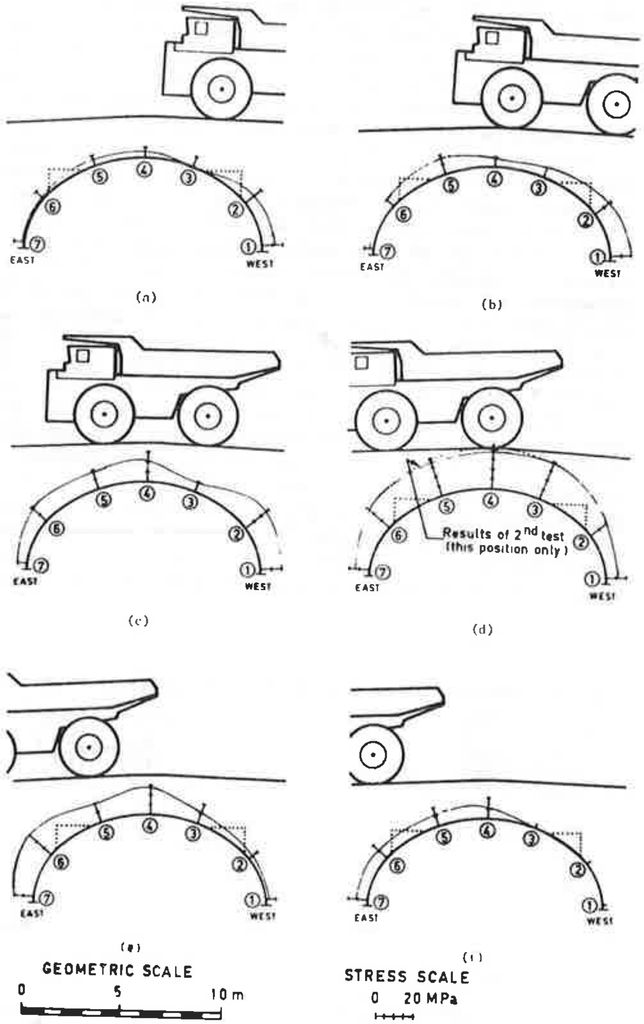
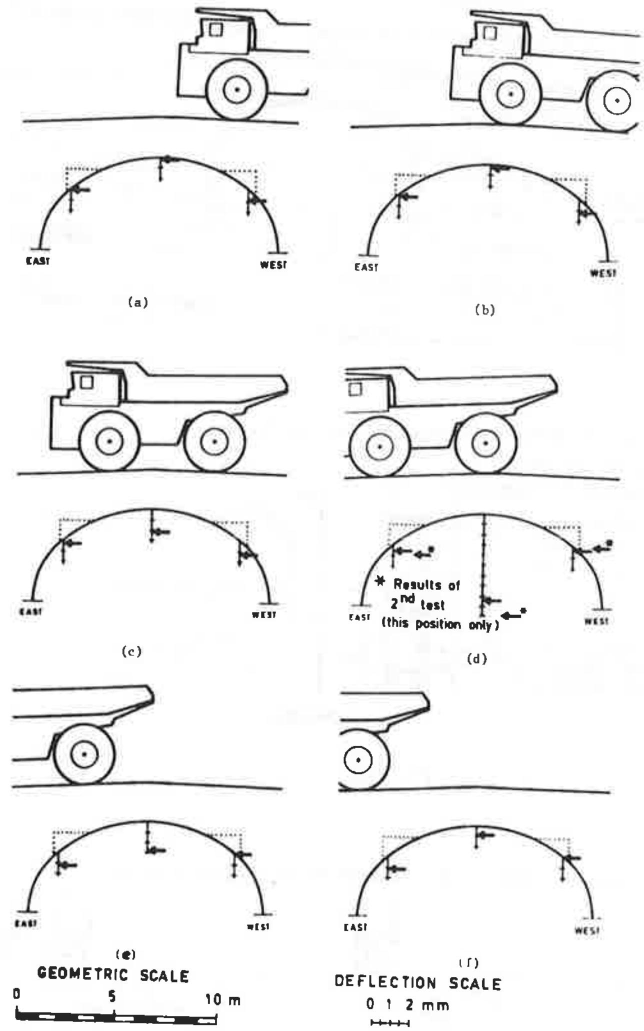



Figure 7. Vertical deflection of steel arch for 290-tonne loaded truck.

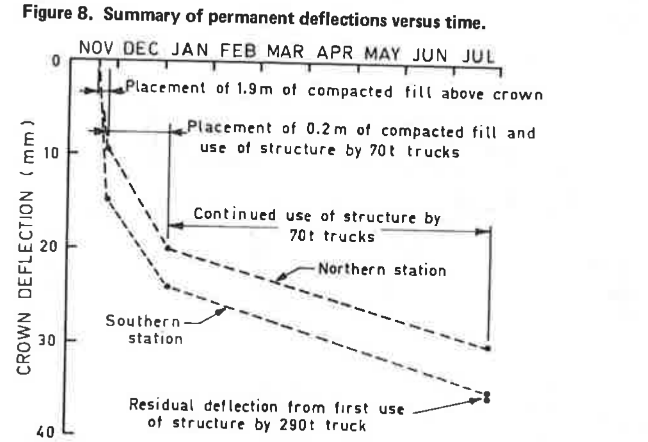



cycles remains. Two factors that contribute to the permanent horizontal movement component of the thrust beam are compaction-stress history and overburden pressure.

During the compaction process, recycling of stresses at a given level tends to produce a soil that will respond relatively elastically provided certain stress levels are not exceeded. It is possible that, locally, stress levels adjacent to the thrust beam exceed the compaction-stress levels when truck loads are applied. Also, the low overburden pressure on the soil adjacent to the thrust beam provides an unfavorable stress state compared with that for a uniform fill loading. Although the vertical stress above the crown increases, the vertical stress outside of the thrust beam is little affected. This condition leads to higher elastic deflection, higher plastic deflection, and a lower level of ultimate load capacity.

Many engineering structures undergo considerable deformation after they are placed in use. Frequently such deformation is unrecognized because it causes little difficulty and it remains unobserved because no measurements are taken. The additional deformation appears to have little significance as far as the Leigh Creek structure is concerned. However, it has important implications in relation to attempts to develop design procedures for such structures. Whereas it appears to be possible to predict the

Figure 8. Summary of permanent deflections versus time.




response of the structure to fill loads, as discussed by Flint and Kay in another paper in this Record, the problem becomes much more complex when live loads are considerably higher than those associated with compaction equipment. If some progress is to be made, it is extremely important to gather and report field data on such projects. Measurement of deflection is probably of greatest importance and this is a relatively simple matter. Gathering of such data will assist with development of criteria for design.

CONCLUSIONS

Details of measured results of stresses and deflections associated with extremely heavy live loading of large-span corrugated-metal arches have been presented. In addition, the permanent effects of these loads on the deflection response have been demonstrated and discussed. Further research in this area will be necessary to enable proper consideration of heavy live loading in the design phase.

REFERENCE

1. J.N. Kay, D.L. Avalle, R.C.L. Flint, and C.F.R. Fitzhardinge. Instrumentation of a Corrugated Steel-Soil Arch Overpass at Leigh Creek, South Australia. Proc., 10th Conference of Australian Road Research Board, Vol. 10, No. 3, 1980, pp. 57-70.

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# Response of Corrugated-Metal Arches to Soil Loads

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**To enable prediction of the response of systems with corrugated-metal arches and compacted soil bridges to soil loading above crown level, graphs were developed from a large number of finite-element analyses. The graphs are based on analysis of a flexible arch in the form of a half ellipse supported on either side by typical footings. They facilitate prediction of response parameters as the fill is placed and compacted in layers above the crown level. The material surrounding the arch is considered homogenous in terms of elastic modulus and Poisson's ratio. Although the graphs are based on linear analysis, important nonlinear effects may be taken into account by application of the graphs in a stepwise manner. Comparison of predicted response with that from a specific finite-element analysis of a field structure suggests that the errors introduced by idealizations associated with the graphical approach may be insignificant for many projects. In addition, comparisons of predictions with field measurements show encouraging results.**

Large-span corrugated-metal arch structures are becoming more common as an alternative to conventional bridges for rail or highway overpasses spanning minor roads or small waterways. Current installations use spans up to about 15 m (50 ft), and several with spans around 12 m (40 ft) have been constructed in Australia. The corrugated plate used is typically 3-7 mm (1/8 to 5/16 in) thick, depending on the size of the arch, with 150x50-mm (6x2-in) corrugations.

The response of these structures to loading is governed by an interaction between a flexible membrane (corrugated metal) and a relatively compressible surrounding medium (compacted soil fill). Analysis of such systems is difficult due to the complex interaction mechanisms involved. No closed-form solution can adequately approximate the true behavior. Most manufacturers use design methods based on formulas that assume a grossly simplified system but at the same time have the backing of considerable experience. Others have developed empirical methods based on small-scale model studies. Some discussion of these methods has been provided by Selig and others (1).

The finite-element method has now been developed to the extent where models for soil-structure interaction problems may be formulated to provide an adequate means for analysis of these structures under working loads. A number of large-span corrugated-metal arch systems have been individually analyzed by this method (2-4), and the results appear to show acceptable correlation with field measurements. However, the cost associated with such analysis is high, due largely to the time involved in formulating and checking the input data for finite-element programs as well as the actual computing costs. Access to a large computer is also required and is not always available to the design engineer.

This paper presents a method whereby the designer may reap the benefits of the finite-element method without the need to resort to the costly detailed analysis of individual structures. Design graphs based on numerous finite-element analyses are presented that may be used to predict the essential response parameters required for design of the wide range of structural configurations likely to be encountered in practice.

To demonstrate the validity and versatility of the graphs, response predictions based on the graphs are compared both with measured results from a field structure and with results obtained from a specific finite-element analysis of the field structure by using mesh data to more accurately fit the specific field conditions. Subject to the uncertainty associated with determination of the appropriate soil modulus, the results are promising.

CONFIGURATION AND CONSTRUCTION CONSIDERATIONS

Although many full-ellipse structures have been completed, the trend in Australia in recent years has been toward construction of half-ellipse and low-profile arch structures. These appear to be more practical and more economical, particularly for overpass applications. Consequently, the half-ellipse profile has been selected as the basic geometry around which the graphs are developed.

The arch is constructed from curved corrugated-plate sections approximately 2x1 m (6x3 ft), which are bolted together on site. Profiles close to elliptical are formed from circles of two different radii for the top and side arch sections. The junction of the two circles occurs at the point where the two radii have a common angle of 50 degrees to the horizontal.

It is convenient to subdivide the construction operation into two stages--stage 1, where the arch is erected and the fill is placed and compacted to crown level, and stage 2, where the fill above the crown is placed and compacted to finished level. The response of the structure to loading during stage 1 is largely dependent on the construction techniques employed. Up to this time, engineering