BEHAVIOUR OF STANWELL PARK VIADUCT, A MULTISPAN BRICK MASONRY ARCH SYSTEM ON TALL PIERS

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ABSTRACT
Stanwell Park Viaduct is an eight-span brick masonry arch structure on tall piers. Located in the South Coast of New South Wales (NSW), it was constructed in 1920 and it has since then suffered significant deterioration, which caused the viaduct to be closed temporarily whilst Span 6 was replaced. The crack propagation prior to the replacement of Arch 6 by a steel-precast concrete span, is summarised chronologically. With the aid of finite element plane stress analysis results, the reasons for the failure of Span 6 and the subsequent damage to Span 4 are discussed.

INTRODUCTION - HISTORIC BACKGROUND
Stanwell Park Viaduct was constructed in 1920 and located between the cities of Sydney and Wollongong in the South Coast of NSW. It is a continuous brick masonry arch structure on tall piers with eight 13.1m diameter spans, totalling 120.2m. The tallest pier is 34m above the ground. The structure was built on a curved line with radius of 240m. Details are given in Figs. 1 and 2. On 12 December 1985, just 3 days before the first scheduled electric train was to cross the viaduct on its way south to Wollongong, a drastic decision was made to postpone the opening schedule so as to allow repair work to be carried out. Arch 6 was eventually replaced by a steel-precast concrete span. It breaks the continuum into two separate portions. Unfortunately, that was not the end of the problem. Subsequent observations have shown that Arch 4 has since
developed structural cracks which show no sign of stopping. The N.S.W. State Rail Authority (SRA) has made plans to "underpin" Span 4 by constructing a supporting concrete arch under the existing arch.

According to the observations made over the years at the viaduct site, there have been large ground movements due principally to the nearby underground coal mining activities. Such movements have contributed to the deterioration of the viaduct. In order to determine the causes for such local damage to the viaduct, detailed structural analysis of the viaduct is carried out with the assistance of the SRA.

A summary of the crack propagation with time is given herein. Behaviour of the viaduct before and after the replacement of Span 6 is discussed. Finally, some comments are given on the restoration scheme.

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Fig. 1. Overall view of the viaduct (pre 1985, downside face).
CRACK PROPAGATION AND REMEDY

The propagation of cracks prior to 12 December 1985 when the viaduct was closed to traffic is summarised in Table 1 chronologically. It should be read in conjunction with Figs. 1 to 10.

<table>
<thead>
<tr>
<th>Date</th>
<th>Crack No.</th>
<th>Details</th>
<th>(Ref. [1])</th>
</tr>
</thead>
<tbody>
<tr>
<td>14/8</td>
<td>#1</td>
<td>Ran longitudinally through Arch 3. The crack opened up to 169mm wide at the soffit (Fig. 3).</td>
<td></td>
</tr>
<tr>
<td>28/8</td>
<td>#2</td>
<td>Extended from Abutment No.1 to the top of Pier 3 (Fig. 2).</td>
<td></td>
</tr>
<tr>
<td>28/8</td>
<td>#3</td>
<td>Ran from Abutment No.2 to the top of Pier 7 on the upside face (Fig. 2).</td>
<td></td>
</tr>
<tr>
<td>10/9</td>
<td>#4</td>
<td>Extended across the spandrel on the upside face above Piers 3 and 4 (Fig. 2).</td>
<td></td>
</tr>
<tr>
<td>10/9</td>
<td>#5</td>
<td>Spalling had started to affect the downside face of Arch 7; an open transverse crack was noted at the soffit of Arch 7 (Fig. 3).</td>
<td></td>
</tr>
<tr>
<td>10/9</td>
<td>#6</td>
<td>Occurred in the spandrel above Arch 7 and opened up the horizontal mortar lines (Figs. 4 and 5).</td>
<td></td>
</tr>
<tr>
<td>10/9</td>
<td>#7</td>
<td>Occurred in the spandrel above Pier 6 (Fig. 4). Cracks #6 and #7 proved to be the precursors of major failure of Span 6. Humping of the parapets has also been observed over Spans 6, 7 (Fig. 7).</td>
<td></td>
</tr>
<tr>
<td>30/10</td>
<td>#3</td>
<td>Extended towards Sydney over Arch 7 and ended on the top of Pier 6 (Fig. 2).</td>
<td></td>
</tr>
<tr>
<td>30/10</td>
<td>#8</td>
<td>Four major transverse and one longitudinal cracks had developed at the soffit of Span 6, the surface spalling indicated the buildup of high compressive stresses. Crack #8 had begun to hinge allowing the arch to buckle upwards (Fig. 3).</td>
<td></td>
</tr>
<tr>
<td>30/10</td>
<td>#9</td>
<td>Two transverse cracks had occurred at the soffit of Span 4 (Fig. 3).</td>
<td></td>
</tr>
<tr>
<td>15/11</td>
<td>#7</td>
<td>Cracking of Arch 6 had become more severe with some crushing of bricks and further opening of the horizontal cracks (#7). (Fig. 6)</td>
<td></td>
</tr>
<tr>
<td>7/12</td>
<td>#3</td>
<td>Fully sheared upwards into the joint (Fig. 8) above the 'dogtooth course' at Wollongong end.</td>
<td></td>
</tr>
<tr>
<td>9/12</td>
<td>#7</td>
<td>Propagated to a dangerous extent with formation of hinges, change of shape and loss of areas of brickwork from the soffit (Fig. 9). It broke through the arch ring of Span 6 which led to the belief that its failure was imminent (Fig. 10).</td>
<td></td>
</tr>
</tbody>
</table>
Because of the continuing deterioration around Span 6, the SRA decided to close the viaduct in order to undertake restorative work days before the official opening of the viaduct to the newly introduced electric train service. Consequently, Span 6 was replaced by a steel-precast concrete span. The replacement work was completed on 4th February 1986. Simultaneous to the work on Span 6, prestressed tie bars were applied at the springing level of each of the remaining arches.

Fig. 2a. Plan view of the viaduct.

Fig. 2b. Elevation and cracks on the upside face.

Fig. 3. Bottom view of the viaduct.
Fig. 4. A conceptual mechanism of the failure of Span 6.

Fig. 5. Shear through spandrel from dogtooth to top of Span 7 (10/9/85, downside).

Fig. 6. Shear failure through Span 6 (10/9/85, downside).

Fig. 7. Humping of Spans 6 and 7 and cracking in spandrel (15/11/85, upside).

Fig. 8. Shear through spandrel from dogtooth to top of Span 8 (7/12/85, downside).
Fig. 9. Overall view of shear across Pier 6 and through Arch 6 (9/12/85, upside).

Fig. 10. Shear through Span 6 (9/12/85, upside).

Fig. 11. Progressive opening of cracks on Span 4 (downside).

Around 20 February 1986, just after the replacement of Span 6 and the application of tie bars, an inclined crack in the parapet, which had existed since August 1985 began to open dramatically at the crown of Arch 4. Details are given in Fig. 11. This led to plan an "underpinning" concrete arch.
The structural behaviour of a multispans brick masonry arch viaduct is far more complicated than that of a single span arch which is normally assumed in design. The effect of the infill to the arch section can be considered as a series of horizontal struts taking compression only (Fig. 12). The horizontal thrusts caused by these struts will lead to a stress distribution in the arch section different from that of a single span arch. The flexibility of the tall piers can also cause movements of the arches at the springings.

In order to identify the reasons for the failure of Span 6, finite element analyses were carried out assuming the entire standard viaduct to be a 2-D wall system under various loading conditions. From information given by the SRA [2], it was noted that the Wollongong abutment (No.2) moved 90mm towards the Sydney abutment (No.1). It was estimated that this relative movement accumulated over a period of two years prior to the failure of Span 6. In the plane stress finite element analysis, in addition to self-weight and the fill, 90mm prescribed displacement is applied to Abutment No.2. The deformed structure is illustrated in Fig. 13. It may be seen that the relative movement between the two springings of Arch 6 is the largest. High compressive stresses were also found to have generated in Spans 6, 7 and 8. According to the SRA report [2], the brickwork of Span 6 is weaker than in other spans. Quite possibly, the combination of high stresses with the low strength of Span 6 caused its failure.

From the ground movements surveyed by the SRA (Fig. 14), it is apparent that the relative vertical movement at the two piers of Span 4 is the largest. This is a factor for the continuing deterioration of Span 4. The results from a finite element analysis have indicated that stress distribution is far more affected by vertical ground movements than by horizontal ones. Obviously, this is because the horizontal movements can be more readily absorbed by the tall piers whereas the vertical ones could only be accommodated by cracking of the arch.

Fig. 12. Effect of infill on the arches.
Fig. 13. Structural deformation (broken line - deformed shape, solid line - original shape).

Fig. 14. Ground vertical movements and structural movements surveyed by SRA as at 23/6/87. (downside)

COMMENTS

Recent survey results (Fig. 14) show that the two portions of the viaduct separated by Span 6 are moving towards each other. This is partly because Span 6 being on simple supports can no longer resist the horizontal movements of the neighbouring arches. Since the construction of the new Span 6, the two supports have moved about 80mm relative to each other. Tie bars which are used widely in Britain for similar rescue work, cannot limit the relative movement of the two portions of the viaduct. However, the movements within individual arches appear to have been prevented. As a result, the cracks have settled down for the time being.

In the design of the support system for the new Span 6, accommodation was provided for a maximum of 600mm relative movement between the two portions of the viaduct. In view of the diminishing rate of relative movement, the rescue plan is considered justified.
REFERENCES


3. Yan Yang, Structural analysis of Stanwell Park Viaduct, Internal Report, Department of Civil and Mining Engineering, University of Wollongong, NSW, October 1987.

ACKNOWLEDGEMENTS

The authors are grateful to the New South Wales State Rail Authority for the access to relevant documents. Financial support to the first author in the form of a postgraduate scholarship is also greatly appreciated. The computing work was carried out at the Department of Civil and Mining Engineering, and at the Computer Centre, University of Wollongong.