Connell Wagner Pty Ltd was commissioned by the Railway Services Authority to undertake a detailed investigation into the structural behaviour and analysis of jack arch bridges, and to provide recommendations for the analysis and load rating of these structures.

Jack arch bridge construction usually comprises two main structural components. Longitudinal RSJ's are the primary spanning members, placed at 900-1500 millimetre centres and generally located parallel to traffic flow. Spanning transversely between the RSJ's as a secondary spanning member, is a brick arch at least two courses thick and with a rise usually half the depth of the RSJ's. The arches spring from a mortar or concrete bed laid on the bottom flanges of the RSJ's.

This paper presents a summary of the investigation and provides recommendations for analysis and rating including methods to deal with load distribution between girders, extent of composite action and allowance for dynamic loading effects.

AUTHOR

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INTRODUCTION

1. The rapid growth of railways in Australia in the late 19th and early 20th centuries led to the development of jack arch bridges. Used primarily for overbridges with clear spans 6.1 - 9.1 metres the majority still in use in the State Rail network were built in the period 1900 - 1935. After this time reinforced concrete and later prestressed concrete had replaced the jack arch concept.

2. In the past, both the load distribution between girders and the extent of composite action of the infill element (arch, concrete infill and roadway material) has not been well understood resulting in methods of analysis long suspected as being too conservative. In addition the extent of corrosion to the girders has been largely unknown as only the soffit of the girder bottom flange is visible. This paper summarises an investigation into these issues involving full scale load testing of three representative bridges, excavating inspection pits in a number of selected bridge decks, reviewing previously demolished jack arch bridges and providing recommendations for load rating and analysis.

DETAILS OF CONSTRUCTION

3. Jack arch construction originated in the United Kingdom early last century. In fact the first recorded jack arch structure was built in 1801 with cast iron girders. In an era before the general use of reinforced concrete, jack arch construction allowed a shallow structural depth for short to medium spans, especially in comparison to the traditional full masonry arch construction.

4. Jack arch bridge construction usually comprises two main structural components. Longitudinal RSJ's are the primary spanning members, placed at 900-1500 millimetre centres and generally located parallel to traffic flow. Spanning transversely between the RSJ's as a secondary spanning member, is a brick arch at least two courses thick and with a rise usually half the depth of the RSJ's. The arches spring from a mortar or concrete bed laid on the bottom flanges of the RSJ's. Details are shown in Figure 1.

5. The horizontal arch action thrust applied by live loading is resisted by steel ties located at mid depth of the RSJ's. Often fill or road pavement is placed above the structural concrete, up to finished road level.

6. Jack arch bridges are usually provided with brick substructures. No special treatment is provided at bearing surfaces and RSJ's usually simply rest on a mortar pad or steel sole plate with no specific provision for expansion or contraction. RSJ's of cast iron have been used extensively in the UK, but none apparently survive in the present State Rail network.

7. The susceptibility of the exposed bottom flange of the RSJ's to corrosion was acknowledged by the early designers and this part of the girder was usually specified to be mortar encased. Nevertheless, this was not found to be successful and has generally broken completely away in all bridges inspected.
8. To date little has been known about the extent of corrosion to the web and top flange of jack arch bridges, since only the bottom flange has been visible. On a number of jack arch structures corrosion to the bottom flange has been considerable. Of particular concern is that the water producing the corrosion has obviously come from above, through the deck, rather than as condensation or exposure to weather, from below.

9. It should be noted that infill concrete is unreinforced and has been placed over the full width of the bridge (usually) without construction joints. Shrinkage cracking of the deck is inevitable. Cracking is most likely to occur in the top surface at the discontinuities formed by the girder flanges. Once shrinkage cracking has occurred it will tend to widen due to flexure of the deck under traffic loading. Given this situation knowledge of the rate and extent of corrosion is essential in order that the bridge structure can be accurately load rated.

10. As part of this investigation test pits have been excavated at three bridge sites, a number of RSA personnel interviewed with reference to first hand knowledge of jack arch bridge corrosion and the remains examined of a number of bridges which have been demolished.

PREVIOUS LOAD RATING ASSUMPTIONS

11. Due to a number of fundamental uncertainties load rating of jack arch overbridges has in the past generally been based on the following generally conservative assumptions.

i) Minimal or no contribution from the concrete deck, ie the girder is assumed to support the load as a non-composite member.

ii) Impact factors adopted as the Australian Bridge Design Code.

iii) Lateral load distribution assuming deck spans simply supported to adjacent girders. Potential cracking of the reinforced concrete infill and brick arch has mitigated against assuming distribution factors for a reinforced concrete deck as per the NASSRA 1976 Bridge Design Specification or similar.

iv) Corrosion loss to the bottom flange as measured or observed by site inspection. No corrosion to web or top flange.

12. These items have been addressed in detail in this investigation. Recommendations are noted at the end of this paper.
LOAD TESTING PROGRAM

13. The three bridges tested and the reasons for their selection are as follows:

**Meeks Road Dive, Marrickville, MMG 6.227**

This is a very wide bridge and hence edge effects on distribution factors can be discounted. Skew crossings can be investigated since the bridge is sufficiently wide to allow vehicle placement both parallel to, and up to a skew of 50° to the main girders. The effects of a skewed trimming edge girder (on the Marrickville end) can also be investigated. The bridge is simply supported single span.

**Canterbury Road, Canterbury, MSB 10.084 UD DG**

This bridge is single span simply supported and is on a 15° skew allowing mild skew effects to be investigated.

**Copeland Road, Beecroft, NSW 26.792**

This is a narrow bridge, thus allowing for edge effects to be modelled. The bridge is on a zero skew and semi continuous (girders joined by web cover plates over piers).

Details of bridges tested are noted in Table 1 below.

<table>
<thead>
<tr>
<th>Bridge Location</th>
<th>Location</th>
<th>Clear Span between front edges of bearings</th>
<th>Square Spacing of girders</th>
<th>Angle of Skew</th>
<th>No. of girders</th>
<th>Girder Depth</th>
<th>Material of Fill</th>
<th>Simply Supported or Continuous</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meeks Road, Marrickville</td>
<td>MMG 6.227</td>
<td>8.540 mm</td>
<td>1,525 mm</td>
<td>Square (Road crosses at 50° skew)</td>
<td>19</td>
<td>610 mm</td>
<td>Mixed Fill</td>
<td>Simply Supported</td>
</tr>
<tr>
<td>Canterbury Road, Canterbury</td>
<td>MSB 10.084 UG, DG</td>
<td>11.360 mm</td>
<td>1,289 mm</td>
<td>15°</td>
<td>17</td>
<td>914 mm</td>
<td>Mixed Fill</td>
<td>Simply Supported</td>
</tr>
<tr>
<td>Copeland Road, Beecroft</td>
<td>NSW 26.792</td>
<td>7.530 mm</td>
<td>1,046 mm</td>
<td>Square</td>
<td>7</td>
<td>508 mm</td>
<td>Highly compacted ballast with clay filling voids</td>
<td>Partially Continuous</td>
</tr>
</tbody>
</table>

Table 1 - Characteristics of vehicles tested
14. Strain gauges were of the electrical resistance three wire type wired to a Peekel CA250 carrier wave amplifier. Installation and monitoring of the strain gauges was carried out by the Railway Services Authority.

15. Two different test vehicles were utilised to load the bridges. A 41.0 tonne triaxle quarry truck was nominated as being suitable, having a similar weight and axle spacing to the AUSTROADS T44 design vehicle. Preliminary calculations however, indicated that this vehicle may over stress the bridges at Marrickville and Beecroft and for those structures a 26.5 tonne dual axle quarry truck was utilised. Details of the trucks and their axle loads are noted in Figures 2 and 3.

16. In order to verify bridge girder sizes assess corrosion levels, and assess the nature of the arch infill, excavation of inspection pits, together with concrete coring and steel sampling was undertaken.

FINDINGS

Distribution factors

17. The load testing confirmed that, if edge effects such as the skew trimmer beam at Marrickville Dive are neglected, the sum of the strains at midspan of all of the girders was constant for any lateral position of the vehicle on the bridge, provided its longitudinal position remained constant. Thus the ratio of the strain of any one girder to the sum of the strains of all girders, the "distribution coefficient", can be used as a measure of the proportion of the total applied load which that girder supports.

18. The notion of distribution coefficients was also investigated in Reference (3) by Chettoe et al (1943) who carried out extensive load testing on 31 bridges using up to five different test vehicles. A summary of the results is shown on Figure 4, plotting girder spacing against "distribution factor". (The "distribution factor" is defined as the ratio of the load carried by the most highly stressed girder to the maximum load that can be applied to any one girder and is also simply the distribution coefficient defined above, multiplied by 2.0).

19. Distribution factors for use on jack arch structures have been prepared by the DOT-UK (1984), Refer Figure 6. These distribution factors are plotted as function of both girder spacing and span (Figure 5) and show a conservative value for each of the three bridges tested, in addition to being also conservative for all of the 31 bridges tested as per Reference (3). It is therefore proposed that the distribution factor method noted on Figure 5 form the basis for rating jack arch bridges.

Composite action effects

20. Results from the testing programme indicate a considerable amount of composite action is provided in reducing live load strains from those calculated on the basis of the bare steel girder only. Work by Chettoe et al (1944) indicates that this composite action can break down at high loads, but not within the usual range of working loads. Load rating
on the basis of ultimate limit State should therefore neglect composite action.

21. For load rating using working loads two approaches appear practical;

Method A: Calculate the composite effect of the brick arch and concrete infill using conventional composite analysis, assuming an effective "flange" width between adjacent arch crowns, neglecting the arch below the neutral axis and assuming an appropriate modular ratio.

Method B: Make an assessment of composite action allowing for the effects of not just the concrete, but also the road base, filling and pavement above. By its nature, given the large variability of material properties above in terms of both stiffness and strength this method has to be empirical. Chettoe et al (1944) proposes that the effective section modulus of the composite beam be taken as equal to that of the bare girder multiplied by a factor \( D/d \) where \( D \) is the overall depth of bridge deck and \( d \) is the depth of the steel girder at midspan.

22. Method A required a relatively complicated and time consuming analysis involving section property calculations of curved sections and multiple analyses to ascertain the location of the neutral axis through interpretation. In addition the nature of the brick arch and concrete infill is that bond to the steelwork is at least unreliable. The reasons for this are;

i) The concrete infill has generally been placed without reinforcement or shrinkage joints. Resultant cracking to the infill has resulted. Some of this cracking is at the interface with the steelwork as evidence by water penetration.

ii) No shear connectors are provided.

23. Results of the field testing show a scatter of impact factors for various speeds and span lengths.

24. This appears to be a common occurrence when this type of structure is tested. 125 impact tests were carried out by Chettoe et al (1944) as part of a Ministry of War bridges assessment programme. A variety of vehicles were used including steam rollers, a 19 tonne truck with pneumatic tyres, a Churchill tank and a low loader carrying a Churchill tank. The results show a scatter between 0.8 and 1.3.

25. The Australian Bridge Design Code requires an impact factor of 1.2 to 1.4. Sufficient work has been done to show this impact factor is conservative. However, there appears to be no scope for a reduction of this factor in rating calculations at this stage.

MATERIAL SECTION PROPERTIES

26. A summary of the findings of the materials tested from three sites, the five sites inspected and the results of the questionnaire are as follows;
a) The exposed bottom flange can be badly corroded due to the inflow of water through the deck and must be inspected prior to any detailed load rating assessment being made.

b) The majority of the bridges inspected showed negligible corrosion to the top flange and web. This is probably due to protective concrete cover in comparison to the exposed bottom flange. Given the contributing effect of the arch and infill to composite section properties a rating assessment carried out assuming zero corrosion to the top flange and web appears reasonable. If there is any doubt inspection tests pits should be dug and the flange inspected.

c) Steel samples taken from structures at Marrickville and Canterbury were tested and yield strengths of approximately 210 MPa indicated. This is considerably below the yield strength range (250-260 MPa) of modern steels (AS 1204). It is therefore recommended that steel samples be taken and tested if an accurate rating assessment is required.

d) A chemical analysis of steel samples tested indicates a high sulphur content (up to 0.081%). Welding to steels with sulphur levels above 0.05% is not recommended due to the probability of cracking in the weld heat affected zone. The effect of existing welding to any girder to which a rating analysis is to be undertaken must be carefully evaluated. This also has implications with regard to fatigue.

RECOMMENDATIONS FOR ANALYSIS AND LOAD RATING

27. The findings of this investigation are such that the following recommendations for load rating can be made;

a) The proportion of load carried by a given girder can be calculated using a distribution factor. It is proposed that the distribution factors in DOT-UK (1984) be used. These distribution factors are a function of both span and girder spacing. Refer Figure 5.

b) If the rating is being carried out based on working stresses then the effect of composite action can be allowed for by taking an effective section modulus as equal to that of the bare girder multiplied by a factor D/d where D is the overall depth of the bridge deck and d is the depth of the girder at midspan. This method allows for the fact that full theoretical composite action is not available.

c) If the rating is carried out based on ultimate strength principles then the effect of composite action should be neglected. Strength calculations should be based on the plastic modulus of the steel section only.

d) The Australian Bridge Design Code should be used for the calculation of impact. This is known to be conservative but there is insufficient data to warrant a reduction.
e) Before any detailed "as is" rating calculation is undertaken, a site inspection is essential to determine:

i) Extent of corrosion to the bottom flange. This directly affects the rating and should be measured with Vernier callipers or similar rather than estimating loss based on the visual assessment.

ii) Extent of corrosion to the top flange and web. This does not have such an impact on the rating as bottom flange corrosion and for the majority of bridges is expected to be very minor. If it is suspected that significant corrosion to the top flange has occurred (as evidenced by cracks, spalling etc to the deck) then test pits should be excavated to allow extent of corrosion to be measured.

iii) That the jack arch between girders is inherently stable and consistent with the proportions of other jack arch structures.

iv) That adequate tie bars are in place, particularly in the vicinity of the outer girders.

v) To assess the effect of utility services.

vi) To confirm that welding to the steel girder has not been carried out. If welding has in fact been undertaken a chemical analysis of the steelwork should be done to establish weldability. A high sulphur content for example could give rise to cracking in the weld heat affected zone. This also has implications with regard to fatigue.

f) Steel samples at two bridges tested indicate a lower yield strength (210 MPa) than current steels to AS1204. Steel sampling and testing should therefore be carried out to accurately determine the yield strength. This is directly proportional to the rating strength of the girder.
REFERENCES

Failure Test of a Jack Arch Bridge Research Report No. 110, David B Beal, 1984

Load Capacity of a Jack Arch Bridge Research Report No. 129, David B Beal, 1985

Paper No. 5418 "The Strength of Cast Iron Girder Bridges", Chettoe et al, 1944

Tests on Road Bridges Research Paper No. 16, Ministry of Transport, 1958

Paper No. 5438 A Laboratory Investigation of Some Bridge Deck Systems, Thomas and Short, 1952

Load Test on Jack Arch Bridge with Cast Iron Girders Research Report No. 310, A F Daly and S J Raggett, 1991

The Assessment of Highway Bridges and Structures Advice Notice BA16/84, Department of Transport, 1984

The Assessment of Highway Bridges and Structures Departmental Standard BD 21/84, Department of Transport, 1984
BITUMINOUS SEAL
ROAD PAVEMENT (VARYING DEPTH)
INFLF CONCRETE DECK
BRICK ARCH
24'x7.5''x100lbs GIRDERS TYP
1525

FIG 1 - TYPICAL CROSS SECTION
FIG 2 - 41t TRI AXLE QUARRY TRUCK

FIG 3 - 26.5t DUAL AXLE QUARRY TRUCK
FIGURE 4
NOTE
FOR ANGLES OF
SKEW GREATER THAN
35° MULTIPLY
FACTOR BY 1.15.

DOUBLE LANE LOADING

REFERENCE:-
DEPT OF TRANSPORT UK - "THE ASSESSMENT
OF HIGHWAY BRIDGES AND STRUCTURES."
ADVICE NOTE BA16/84.

FIG 5 - PROPORTION FACTORS FOR INTERNAL
LONGITUDINAL GIRDERS