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Strengthening of Brinsworth Road Bridge, Rotherham

Renforcement du pont de Brinsworth Road à Rotherham

Verstärkung der Brinsworth Road Bridge in Rotherham

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SUMMARY

The paper describes the strengthening of a precast, prestressed hollow-box beam skew bridge deck by means of steel plates bonded transversely to the soffit. During the passage of a 456 tons load deflection and strain measurements were undertaken and the transverse profiles compared with those obtained by theory.

RESUME

La contribution décrit le renforcement d'un pont biais précontraint et préfabriqué à l'aide de poutres à caisson, effectué à l'aide de plaques d'acier collées transversalement. Lors du passage d'une charge de 456 to, les mesures ont été comparées avec les valeurs calculées.

ZUSAMMENFASSUNG

Der Beitrag beschreibt die Verstärkung der Fahrbahnplatte einer vorgefertigten, vorgespannten schiefen Hohlkastenbrücke mit quer zur Brückenaxe aufgebrachten Stahlplatten. Durchbiegungs- und Dehnungsmessungen wurden während des Verschiebens einer Last von 456 Tonnen vorgenommen und mit den theoretischen Werten verglichen.



1. DESCRIPTION OF BRIDGE

Brinsworth Road Bridge was constructed in 1965 to carry the M1 London - Leeds Motorway over a side road. The existing road alignment dictated a 45° skew crossing with a clear span between abutment faces of 16.3m.

The deck supporting each carriageway was formed from 17 no. 965mm. wide x 1117mm. deep hollow prestressed concrete box beams placed at close centres. Following concreting of the interbeam gap transverse post-tensioning was applied on the line of internal diaphragms in the boxes. An external fascia beam was placed to carry the parapets. A plan on the bridge deck is shown in figure 1.

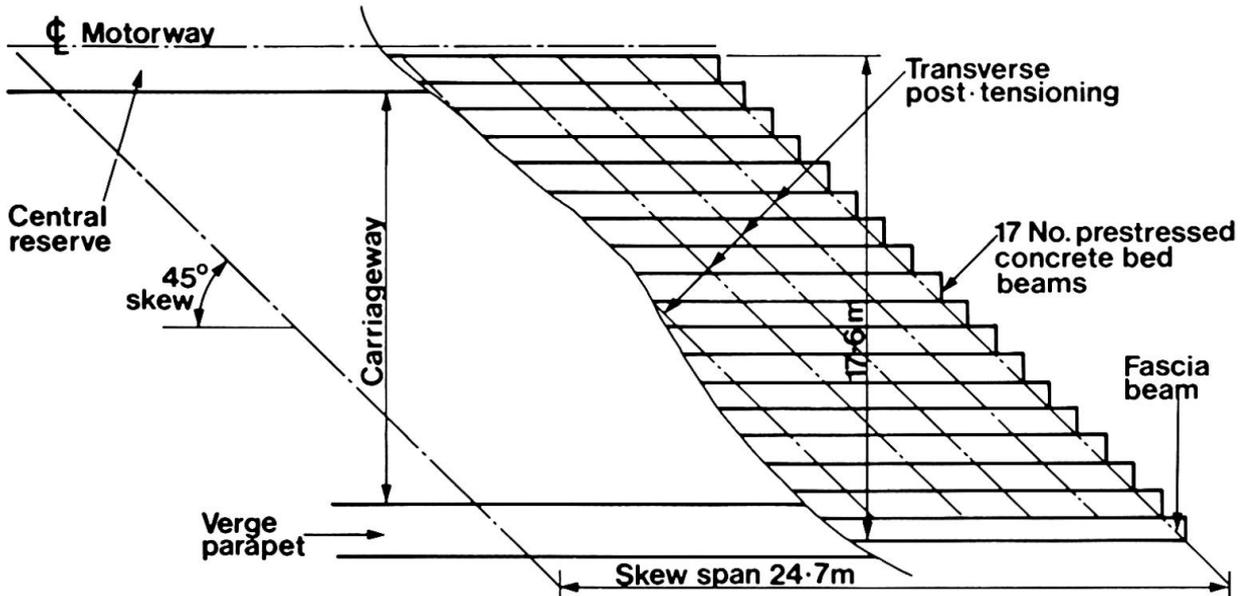


Fig. 1 Plan on Bridge Deck

2. INITIAL DESIGN AND SUBSEQUENT CHECK

The bridge was designed for the Ministry of Transport standard loading of HA and 45 units of HB, all as given in the British Standard ⁽¹⁾. In designing the main beams use was made of the reduction factors allowed in the Standard for HB loading, the transverse stressing being proportioned using the rules suggested by the Prestressed Concrete Development Group ⁽²⁾ cross referencing the recommendations of Gifford ⁽³⁾. Subsequent to completion of the bridge it was felt that the above rules might not be applicable to a deck of such a high skew and an analysis was made using computer grillage techniques. This demonstrated that, in order to achieve the lateral distribution assumed in the design, a much higher level of transverse strength would be required than that provided by the post-tensioning. The bridge was limited to a maximum vehicle loading of 100 tonnes.

3. PRELIMINARY STRENGTHENING CONSIDERATIONS

Alternative systems for strengthening included:-

- (a) Additional overlay reinforced concrete slab. Discounted due to additional dead weight and need to undertake considerable road works to preserve a satisfactory highway alignment.



- (b) Enhancing strength of box beams by addition of longitudinal plates to soffits. Whilst increasing the bending strength this did little to enhance the deficient shear capacity.

4. ADOPTED SOLUTION

The strengthening technique finally adopted consisted of bonding, 400 x 100mm steel plates to the soffit of the deck in a direction parallel to the abutments using a 2 pack epoxy resin. Primary strengthening was provided in line with the transverse diaphragms, intermediate secondary plates being provided over the middle third of the spans.

Whilst the technique itself was not innovative two aspects called for further investigation.

- (a) With a 2% cross fall on the deck and each box beam placed vertically a step of over 25mm. occurred at the junction of each beam. Little information was available on the behaviour of epoxy resin adhesives at this thickness.
- (b) What restrictions, if any, should be placed on traffic using the deck during fixing of the plates and curing of the resin.

Swamy and Jones of Sheffield University were retained to undertake a testing programme and report. Their conclusions (4) (5) confirmed the soundness of the strengthening technique and that the above would have negligible deleterious effect on its effectiveness.

5. PRACTICAL CONSIDERATIONS

In detailing the proposals it was decided to limit plate lengths to 2.8m., bonding 1.75m. cover plates across the gaps. 20mm. diameter expanding anchor bolts were fixed into the interbeam insitu concrete which served the dual purpose of retaining the plates in position during curing of the resin and for safety in holding the plates should long term deterioration of the plate/Resin/concrete interface occur. (Fig. 2.)

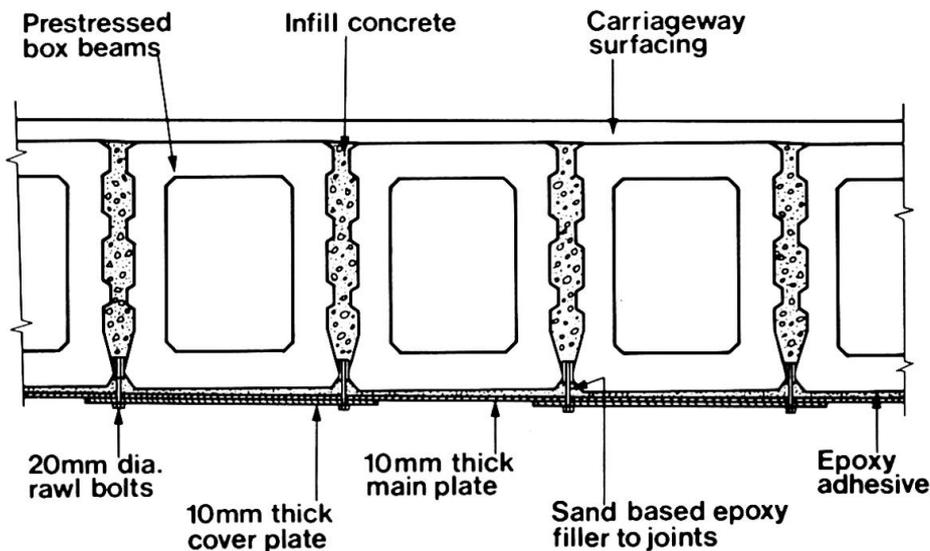


Fig. 2 Section on Deck



The specification required all plates to be cleaned and grit blasted and all dust removed by vacuum cleaner, the first coat primer being applied within a stated period. A similar treatment for cleaning and blasting was applied to the soffit concrete to remove all laitance and to expose the aggregate.

The resin adhesive for the concrete/steel interface was initially applied to both surfaces as a primer followed by the main filled resin, all to Formulated Resins specification XD 808. The specification for the steel/steel bond on the cover plates was Epoxy RSI again as supplied by Formulated Resins Ltd.

Implementation of the strengthening was undertaken the County Engineer's Department. Whilst the staff and operatives within the Maintenance Division had had no previous experience of this or similar techniques, test procedures enabled a satisfactory method of working to be developed and a high level of output was maintained throughout. Figure 4 shows the use of extensions to the expanding bolts to provide a "table" for application of the resin. Spreader beams were used to support the plates during setting and curing of the resin to avoid localised distortion of the plate.

As the work was undertaken from October to December, artificial heating was employed to ensure satisfactory curing.



Fig. 3 Bonding of Steel Plates

On completion all external steel surfaces were given a four coat protective treatment of epoxy primer followed by Zinc Phosphate/M.I.O. Chlorinated Rubber under and finishing coats.

6. TEST LOADING OF STRUCTURE

The main incentive for undertaking the strengthening was the routing of a series of 456 tonne loads for which no suitable alternative route was available and it was decided to use the first of these loads to monitor the behaviour of the bridge deck with its strengthening.

The transporter, carrying a 334 tonne casting, was of 24 axles, axle spacing being a constant 1.6m. Equalisation of loading between the axles was achieved by spreader beams directly under the load and an interconnected hydraulic system to each axle. The vehicle, with load, is shown in Fig. 4

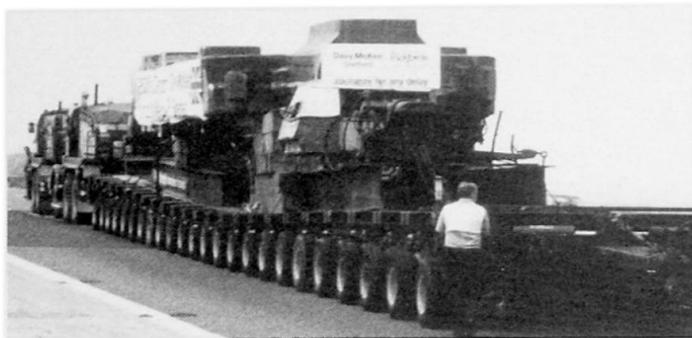


Fig. 4 Test Load Vehicle

The transporter was positioned on the longitudinal centre line of the bridge deck, all other traffic being diverted to the adjacent carriageway. With an overall wheelbase of 36.8m. only some 67% of the total gross load was carried by the bridge super-structure but this was equivalent to 46.4 units HB for bending 44.4 units HB for end shear.

Deflection readings were taken transversely across the mid span of the deck together with 50 Demec strain gauge points located to monitor the transverse bending effects at mid span, the distribution of moments across the deck, localised strains over the precast beam interface and strains in the web adjacent to the central reserve. The strain gauge readings were supplemented by 6 low voltage linear transducers fitted to detected strains in parts of the deck which were inaccessible during the test.

7. TEST RESULTS

The deflection profile on the skew centre line of the deck is as shown in Fig. 5. It is of interest to note that during the 100 mins. the load remained on the bridge a uniform increase of deflection of 0.4mm. occurred across the deck. The test took place in the late afternoon with a falling air temperature and a differential temperature of 1°C across the deck would result in a 0.6mm. deflection.

Unfortunately the correlation between sets of Demec and L.V.D.T. readings was not particularly good but, in general, the strains recorded confirmed the low values of the deflection profile. It is also of interest to note, that strains measured across the beam joints under the load showed nearly double the value of strains in the adjacent steel plate.

Following the load test PUNDIT (Portable Ultrasonic Non-Destructive Digital Indicator Test) tests were undertaken on bridge beams to attempt to ascertain the elastic modulus for the concrete. Readings gave values between 32.3 and 50.4 kN/mm² and a mean value of 41.3 has been taken.

8. PREDICTIVE ANALYSES

In conjunction with the independant checkers, two mathematical analyses were performed on the deck using a grillage idealisation and used to check the strengthening system for adequacy. In both cases full section properties were taken for the members modelling the main boxes and a transformed section assuming full tensile cracking of concrete section transversely.

The analyses were then performed using

- (a) the assumption of no torsional rigidity in the grillage members,



and
 (b) taking the full torsional properties on all members.

In addition the charts produced by Rüsç and Hergenröder⁽⁶⁾ for an isotopic slab of aspect ratio of 1.0 and skew of 45° were used to make a comparison with the test load results.

Figs. 5, 6 and 7 show the results of the analyses for deflection, longitudinal and transverse moments on the skew deck centre line.

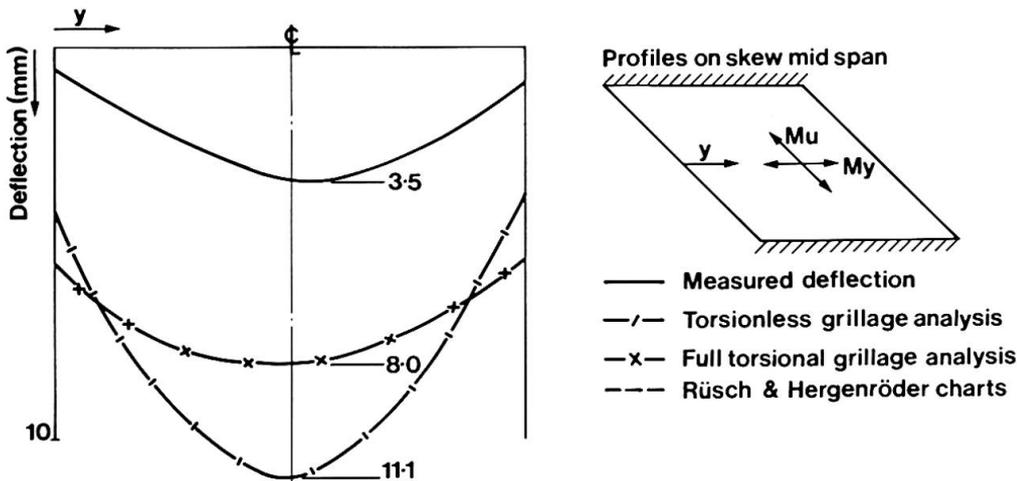


Fig.5 Deflection profiles

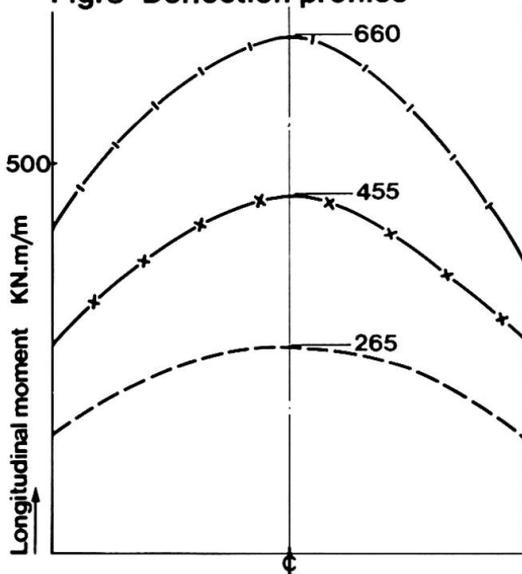


Fig.6 Longitudinal moment (Mu) profiles

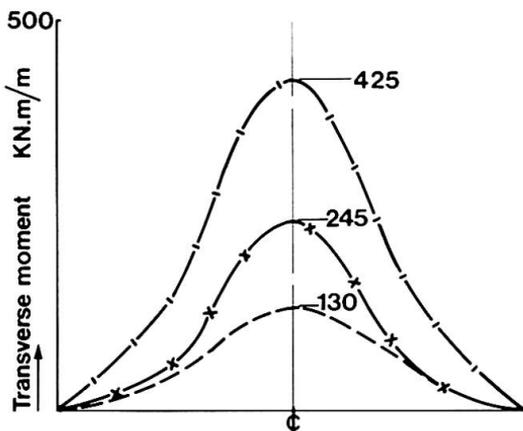


Fig.7 Transverse moment (My) profiles

Fig. 5, 6 and 7 Deflection and Moment Profiles

9. COMPARISON OF TEST RESULTS WITH THEORY

The first and obvious point of note is that the measured deflections were substantially less than those predicted by either of the grillage analyses. Whilst the participating effect of surfacing may be a contributory factor it is not considered that this is sufficient to account for the wide variation. On the basis of a mean measured deflection across the deck of 2.5mm, the equivalent effective span is of the order of 18.2m, and it would appear as if the deck is behaving in a much more "square" manner than the analyses would indicate.

With this order of deflection the maximum moment in the boxes will have reduced to the order of 250kN/m which compares favourably with that obtained from the Rusch and Hergenroder approach.

Transversely the deflection measurements would indicate a moment M_y of the order of 150kN/m, a figure which again is comparable with the isotropic slab method.

10. CONCLUSIONS

Whilst there is no comparable load test data for the bridge deck in its original unstrengthened state it is highly probable that the equivalent effective span would have been much nearer that of the primary box beams. The strengthened deck has behaved in a far better manner than that predicted by grillage analyses and, for the particular vehicular loading position, approaches quite closely the characteristics of an isotropic slab.

Whilst, in retrospect, it may be felt that the bridge may have been over-strengthened, the comparative ease and economy with which this was done has more than justified the approach taken.

11. ACKNOWLEDGEMENTS

The work of evolving the strengthening proposals and their implementation was carried out by the South Yorkshire County Council as agents for the U.K. Department of Transport and thanks are expressed to the County Engineer, John F. Kirkham for his encouragement at all stages of the project and for permission to publish this paper.

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