Proceedings of the Institution of Civil Engineers
Bridge Engineering 160
December 2007 Issue BE4
Pages 163–172
doi: 10.1680/bren.2007.160.4.163

Paper 14935
Received 06/09/2006
Accepted 14/06/2007

Keywords:
bridges/contracting/dynamics

Alan H. W. Baker
Associate, Scott Wilson, Glasgow, UK

David Webb
Associate, Scott Wilson Basingstoke, UK

Rachael Fearnhead
Project Engineer (Civils), Network Rail, London, UK

West Coast route modernisation: River Tame viaduct

A. H. W. Baker BSc, CEng, MICE, MIHT, D. Webb MA, CEng, MICE, MIStructE and R. Fearnhead BEng

Network Rail is increasing the capacity of the West Coast main line north of Birmingham by doubling the number of tracks from two to four. Within the Trent Valley section of the route, the first of a pair of three-span viaducts has been constructed to carry the railway across the River Tame. The structures are each 94 m in length and comprise half-through superstructures supported on piled foundations. The current paper describes the development and detail design of these viaducts, and in particular the design requirements relating to dynamic effects caused by the passage of high-speed trains. These are some of the first bridges to have been designed to recent guidelines which require a bridge-specific dynamic analysis to be undertaken for any structure of this type. This analysis was undertaken using a finite-element program in which each bridge was modelled as a three-dimensional stiffened plate employing both thick shell and shear beam elements. Aspects of the construction of the first viaduct are discussed, including piling methods, environmental and archaeological considerations. Protection of the river and its ecology were important, and measures taken to address these are outlined.

1. INTRODUCTION

The UK's West Coast main line railway is currently undergoing a substantial modernisation programme with a view to delivering major benefits to passenger and freight customers on a route connecting Glasgow to London, with diverging routes to other major industrial centres. These enhancements include:

(*a*) 200 km/h line speed to provide faster journey times
(*b*) the capacity for significantly more long-distance passenger trains
(*c*) the capacity for freight traffic growth
(*d*) improved commuter flows
(*e*) improved safety.

The Trent Valley section of the route in Staffordshire is particularly restricted; work to upgrade it, however, is now well under way. In a £350 million project implemented by Network Rail, the current two-track configuration will be doubled by adding a further two tracks between Tamworth in the south and Armitage in the north, a distance of some 19 km. Within this section of the route, 37 new or reconstructed bridges need to be built, substantial earthworks undertaken and major modifications to the track, signalling and overhead line equipment executed.

2. DESIGN DEVELOPMENT

The railway currently crosses the River Tame north of Tamworth on a skewed half-through girder bridge comprising three 22·5 m simply-supported spans, built in 1895 to replace an earlier structure with an identical span arrangement immediately to the east. The superstructure of this early bridge was demolished at the time, but the piers and abutments remain (Fig. 1). At the design development stage, it was proposed to construct a new bridge to carry the two additional tracks on the same alignment as the early structure. Although the piers were deemed to be too insubstantial and would have to be replaced, the existing abutments, if strengthened, were judged to be capable of reuse. The new bridge was also intended to adopt the same span arrangement as the existing structure. This is not the most efficient configuration for what was intended to be a continuous superstructure, but as the piers would be in the river, adopting the same alignment as the existing adjacent bridge would minimise any consequential adverse hydraulic effects within the river regime.

Consultations were held at a very early stage with the Environment Agency, which manages the river, and further meetings took place as the designs evolved. These were particularly valuable, as they allowed the Agency to become involved in the design process and for its views and concerns to be expressed and addressed. (The benefit of this approach was to be confirmed later when the Agency was able to provide the necessary consents with few conditions.) One of these concerns related to the extent of the temporary works that would be required in the river to demolish both existing piers and construct their replacements. The Agency felt that these would adversely constrict the river flow, and advised that only one pier at a time could be replaced. This had the effect of extending the envisaged construction programme.

Around this time, however, Network Rail became increasingly concerned about the condition of the existing bridge. A structural assessment was commissioned, together with a corrosion survey of the superstructure, and material sampling and testing were undertaken. The assessment concluded that the stresses were within the category which required the bridge to be strengthened or repaired within 10 years. Network Rail was of the view that attempting to strengthen an already ageing bridge on a difficult site may be false economy, and a replacement strategy report was

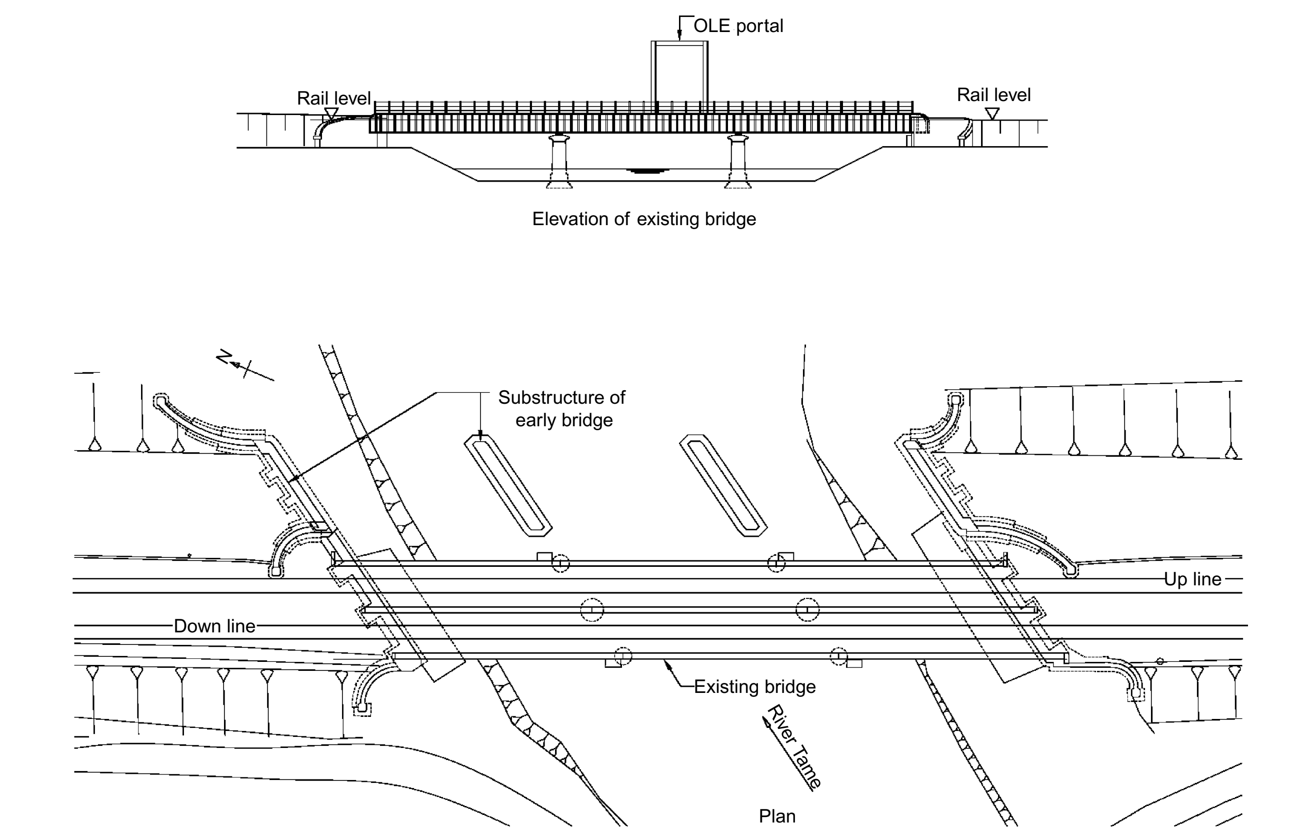

Fig. 1. Existing bridge

commissioned to examine the options for replacement either at the same time as the construction of the new bridge, or at a later date within the 10-year period. A decision was taken to replace the existing bridge shortly after the new bridge was commissioned. As there would now be two new bridges, the opportunity was taken to review the design solution that had emerged to date. The centre span was made longer than the outer spans, which resulted in a more efficient solution (Fig. 2). It also enabled the piers to be located nearer the river banks, which meant that the temporary works in the river were significantly reduced, allowing both piers to be constructed simultaneously, with savings in construction time.

3. STRUCTURAL FORM

The structural form of the new bridges was influenced by several key factors, *inter alia*

(*a*) buildability and safety during construction, including access
(*b*) the availability of possession times
(*c*) the proximity of the existing bridge
(*d*) the proximity of rail traffic
(*e*) the requirements of the Environment Agency
(*f*) ground conditions
(*g*) dynamic effects.

3.1. Superstructure

The choice of structural form of the bridge decks was a compromise between the need to control dynamic effects of high-speed traffic by the provision of a heavy structure with high mass and high inertia, and the need to devise a solution wherein the bridge components could be easily transported and assembled on a constrained site adjacent to a live railway with limited possession times. A network of route-wide haul roads was to be established as part of the Trent Valley upgrade, including a temporary bridge across the River Tame downstream from the existing bridge, which allowed access to most of the areas allocated for the contractor's works; nevertheless, one area remained where access was likely to be difficult and this had to be recognised in devising the appropriate deck type. After considering a variety of options, the proposed solution was a pair of three-span continuous bridges, each with a 37·8 m centre span and two 28 m end spans at a 30° skew. Each superstructure comprised two 3·6 m deep outer steel plate girders supporting an 800 mm thick composite steel–concrete deck (Fig. 3). These dimensions resulted from the conclusions of a dynamic analysis of the deck, which sought to limit vertical accelerations under high-speed loading to within prescribed limits. The east girder was pinned at the north abutment, with guided bearings beneath its remaining supports, allowing longitudinal expansion. The remaining bearings at the north abutment provided longitudinal restraint, and free-sliding bearings elsewhere allowed for all movement.

3.2. Substructure

Ground investigations indicated a shallow band of sands and gravels overlying relatively weak siltstones and mudstones with bands of clay and sand. The existing bridge piers are large-diameter steel caissons filled with brick and concrete. It was considered that piled foundations would be required at all supports to limit the differential settlement between the piers and abutments, and small-diameter and large-diameter options were investigated. Once again, availability of possession times and the proximity of the live railway, existing bridge and its foundations were significant factors. The advice of experienced railway

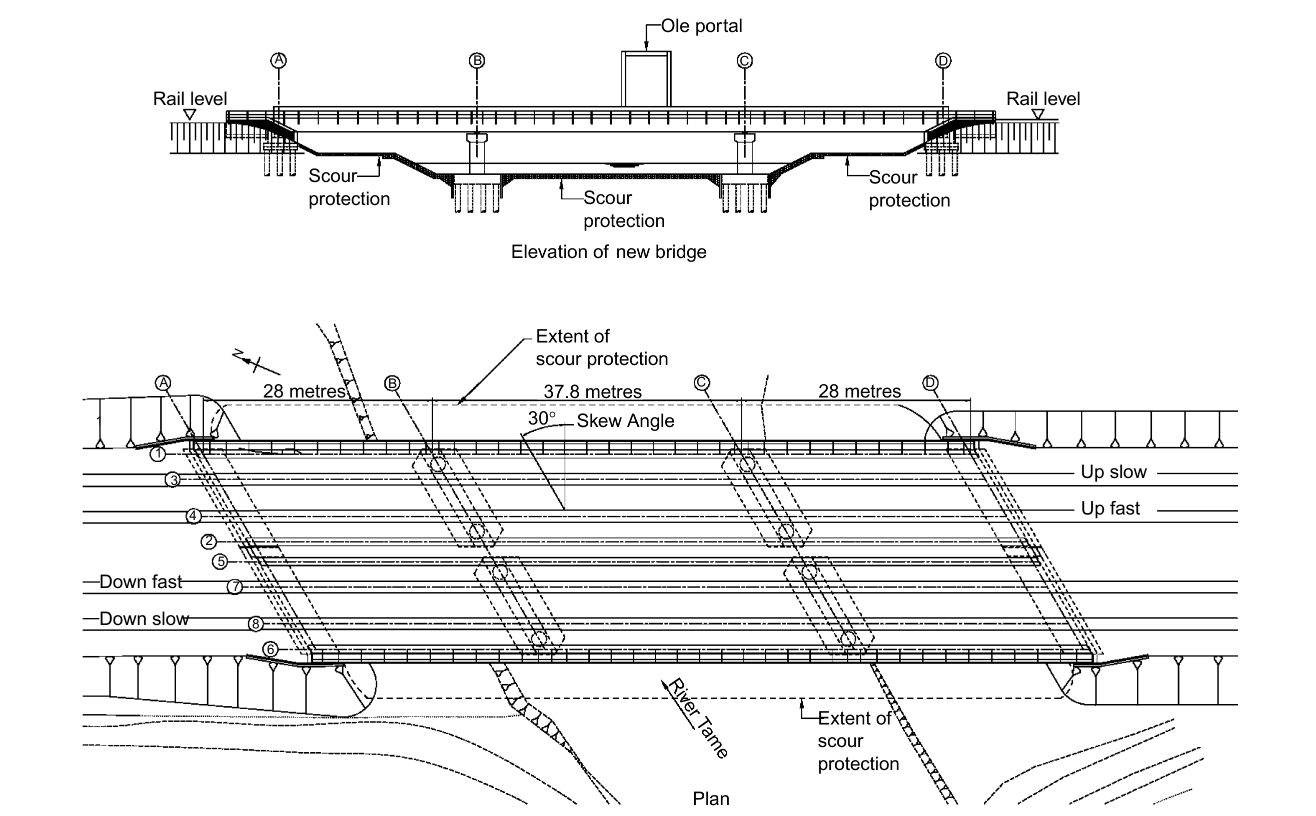

Fig. 2. Proposed bridge

contractors was taken into account, and small-diameter piles emerged as the preferred solution with up to four rows required at each support to resist the combination of applied vertical and horizontal loads. The piers consisted of a pair of reinforced concrete columns supporting a crosshead. As the final span arrangements and magnitude of the support reactions rendered the reuse of the existing abutments unfeasible, reinforced concrete bankseats were located behind the existing abutments. A monitoring strategy for the existing bridge was prepared with the objective of ensuring that any movement of the existing track during construction of the new bridge was recorded and appropriate action taken.

4. DYNAMIC ANALYSIS

One of the most significant factors influencing the design of the new viaducts was the need to implement Network Rail's recently published *Guidance Notes for the Design of Railway Bridges Subject to High Speed Operation.*[1] The route forms part of the Trans-European High Speed Rail Network for which the EU interoperability criteria apply and, as a result of this and the form of the bridge, the Guidance Notes require a bridge-specific dynamic analysis to be undertaken.

4.1. Introduction

The issue of coupled vibration between moving vehicles and bridges is not new, and a fairly straightforward analysis demonstrates that, for most trains crossing bridges, the effect is small and can satisfactorily be dealt with by the use of a dynamic multiplier. With both higher train speeds and a regular pattern of wheels, however, there is a tendency to cause noticeable accelerations in the bridge and it is for this reason that the Network Rail guidance has been produced for both the relevant parameters and acceptable values of acceleration.

Acceleration is limited to ensure that the risk of ballast destabilisation is minimised, and based on test data an allowable value of 3·5 m/s^2 has been adopted for track laid on ballast. The analysis was carried out in accordance with the guidance notes, applicable parts of EN 1992[2] and 1994,[3] BS 5400 Part 3[4] and Part 4,[5] BD 37/01[6] and UIC 776-1R.[7] A finite-element (FE) analysis was undertaken to ensure that accelerations in the deck and fatigue stresses in the structural elements were acceptable.

4.2. Modelling and analysis procedures

The FE model of the bridge deck was built and solved using the Abaqus suite of programs linked to custom pre- and post-processors. The bridge was modelled by representing the girders with two-dimensional (2D) thick beam elements and the deck with 2D thick shell elements that were connected by specifying tied constraints. Bearings were modelled as earth fixings with appropriate freedoms to reflect the design. The use of stiffness values for the bearings was considered unnecessary for this analysis, as the deck behaviour was not significantly dependent on this. Piers and foundations were not modelled, as the dynamic investigation was essentially a vertical loading assessment, and using a similar logic to that adopted for the bearings, modelling the vertical stiffness of these was considered unnecessary.

A separate static analysis was undertaken to confirm the extent of the concrete which could be modelled as uncracked. The maximum principal stresses in the deck from a standard mainline railway loading together with the self-weight of the bridge were examined and found to be less than 4 N/mm^2, taken to be the

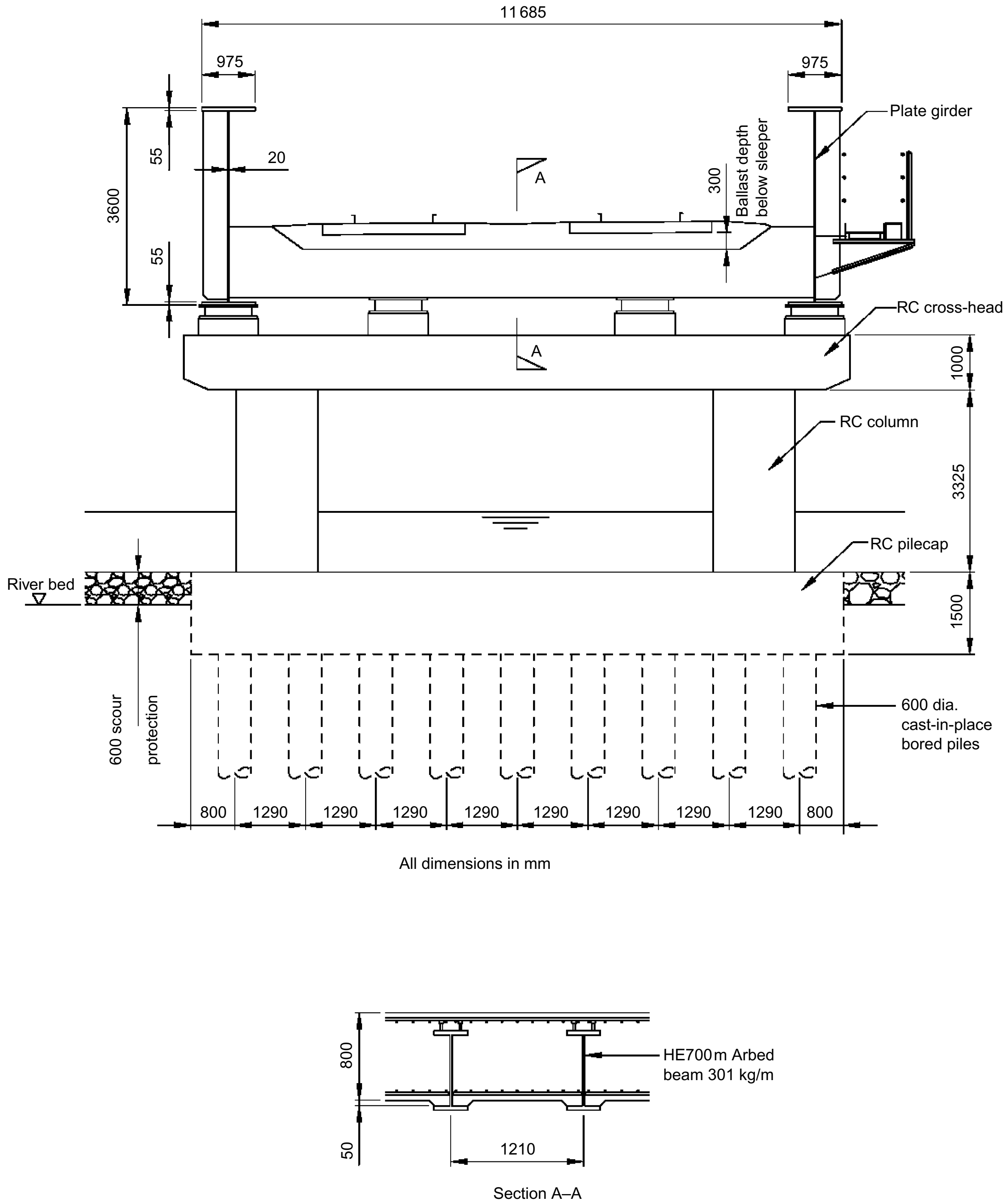

Fig. 3. Deck cross-sections

limiting tensile strength appropriate to the C40/50 class of concrete adopted in the deck. The Young's modulus was increased by a factor of 1·13 based on data from the Comité Euro-International du Béton (CEB)[8] to account for the rate of loading.

U-frame action was considered in the static design to provide restraint at top flange level against buckling. The U-frame analysis consisted of a 2D frame analysis of a cross-section consisting of the effective main girder web/stiffener section together with the composite transverse girder section. Load effects were added to the global model combinations, taking account of the cross-girder skew effects. The close spacing of the steel cross-beams within the composite deck meant that no reduction was necessary for shear lag in the transverse beams.

In order to model realistically the passage of a high-speed train over the bridge, a pair of beams was created to represent the rails. The length of these rails was taken as 220 m, which provided a 25 m run on and 100 m run off. The length of rail was varied in order to assess its effect on the analysis and it was decided that the 220 m length was appropriate as it became clear that the viaduct was insensitive to changes in rail length over the lengths considered. In the run-off and run-on zones, the rails were connected to ground at each sleeper by springs, which modelled the ballast. Over the bridge deck, the rails were connected to the bridge deck through the ballast springs. Following a sensitivity analysis on the effect of the bridge response to a change in the spring stiffness, and from research undertaken by the designer's organisation, the

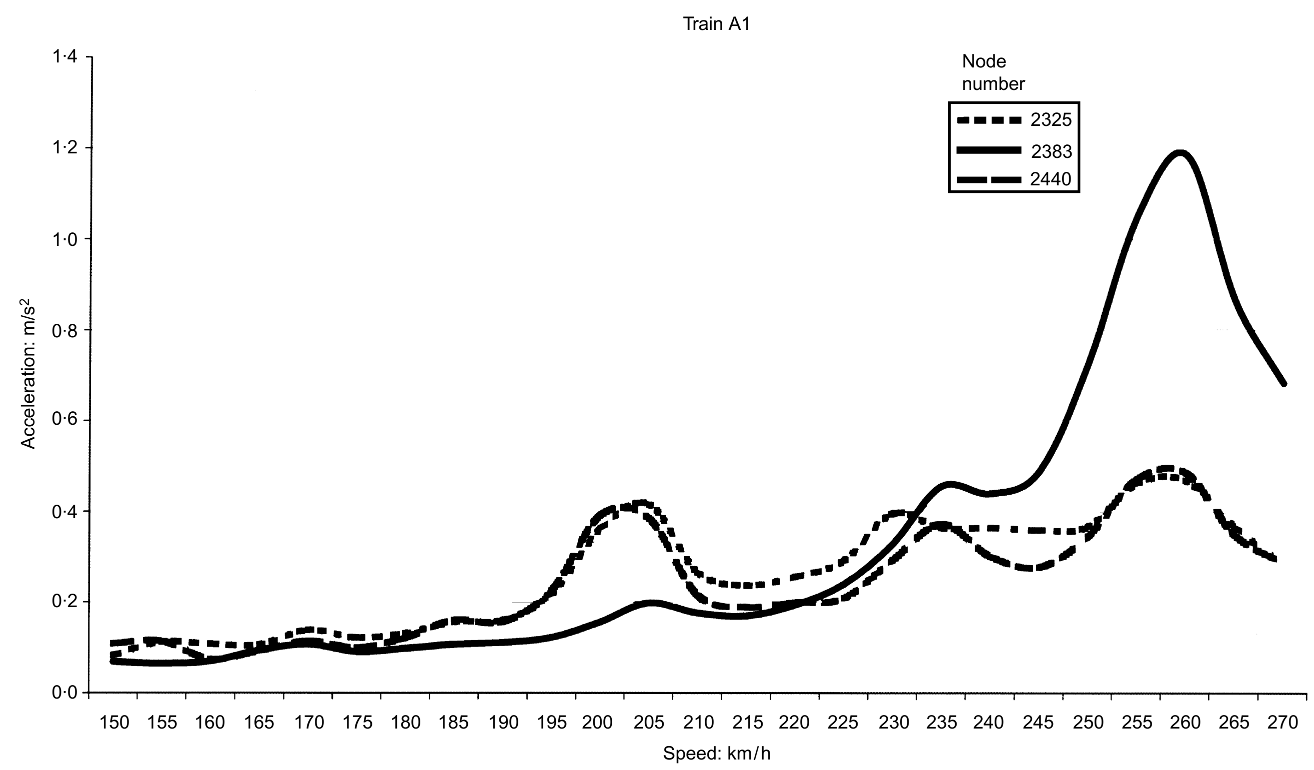

Fig. 4. Deck acceleration as a function of train speed

stiffness of the ballast springs was taken as 17 000 kN/m. This is a conservative value, albeit that the behaviour of the bridge was relatively insensitive to changes in stiffnesses of this magnitude.

Network Rail requested that the structure be designed for maximum line speeds of 225 km/h (fast line) and 200 km/h (slow line), and hence a maximum design speed of 270 km/h was used, representing $1{\cdot}2 \times$ the maximum line speed.

The structure was analysed using the following train sets, generally over a range of speeds from 150 km/h to 270 km/h in 5 km/h increments

(*a*) class 390 Virgin tilting train
(*b*) class 373 Eurostar capital sets
(*c*) Virgin X country class 220 DEMU
(*d*) Virgin X country class 221 tilting train DEMU
(*e*) class 67 loco + mark II or III coaches + DVT
(*f*) a series of 10 generic high-speed load models (HSLM A1–A10).

Only trains on one line were analysed, in accordance with the guidance notes.

A 300 mm thick layer of dry ballast was modelled in the analysis. During the course of the investigation, greater ballast thicknesses and densities were provided, in order to determine the bridge's sensitivity. Other research work undertaken by the designer's organisation has concluded that higher-bound ballast data were never worse than lower-bound data, and thus the lower-bound ballast density and thickness were used for all trains. For verification of the effects of changed ballast properties, HSLM train A1 was run on the bridge with higher-bound properties. The results obtained for the higher-bound were similar to, although less than, those obtained with the lower-bound values.

For numerical efficiency, the response of the structure to train-induced vibration was calculated using modal superposition techniques. The dynamic analysis was of sufficient duration to ensure that the full decay arising from the passage of each train was captured.

The highest modal frequency used in the analysis was 30 Hz, this value being assessed on the basis of the low-frequency mode shapes. The level of modal damping applied to the model was taken as $\zeta = 0{\cdot}005$, in accordance with the guidance notes.

During the course of the investigation, the bridge's section properties were adjusted to ensure that the vertical bridge deck accelerations did not significantly exceed $3{\cdot}5\ \mathrm{m/s^2}$. Accelerations were factored by using a track defects factor of 1·005, in accordance with the guidance notes and UIC 776-1R.[7]

4.3. Acceleration and displacement results

For each train or HSLM, peak instantaneous accelerations produced from the analysis were plotted against train speed for different locations (nodes) on the bridge in the form of graphs similar to Fig. 4. Bridge accelerations were also reviewed as videos to gain an understanding of the spatial and temporal extents of localised areas of high accelerations. The maximum vertical acceleration for any location on the bridge for all trains and speeds was $3{\cdot}6\ \mathrm{m/s^2}$.

A total of 44 natural modes of oscillation below 30 Hz were found. Many exhibited common forms in terms of cross-beam and main beam deflected shapes. The lowest natural frequencies are shown in Table 1. A typical mode shape is given in Fig. 5.

Mode	Natural frequency: Hz
1	3·4
2	4·0
3	6·3
4	7·1
5	7·3
6	9·4
7	10·5
8	10·7
9	11·1
10–14	12·3 to 18·1
15–20	18·1 to 18·9
21–30	19·7 to 23·7
31–44	24·0 to 29·7

Table 1. Lowest natural frequencies

Equivalent flexural and torsional rigidities for the deck (calculated on the loaded span) were found to be as follows

Vertical bending: $EI = 3$ to 6×10^8 kNm^2

Torsion: GJ = 9 to 11 $\times 10^7$ kNm^2/rad

Lateral bending stiffness: $L^3/EI < 0{\cdot}05$ mm/kN

All displacements were found to be less than 1 mm and therefore acceptable for both absolute displacements and twists.

4.4. Fatigue results

A fatigue analysis was undertaken on the 21 most highly stressed elements on the bridge, which were chosen to provide the most onerous case for the different types of connection. The locations for the checks were as follows.

(*a*) Main girders: longitudinal shear between the flange and web at the supports and at the welds between the doubler plate on the flange.
(*b*) Cross-girders: axial, bending and shear stresses at the welds at bridge mid-span and near the supports.

Stress concentration factors were applied as appropriate.

Peak stress ranges were calculated at five locations (four extremities on the I-beam and at the web–flange interface) as a function of train and speed. A plot of stress range against frequency is given in Fig. 6.

The greatest stress range obtained was less than 25 N/mm^2 and thus below the minimum value of non-propagating stress for all weld types in accordance with BS 5400 Part 10.[9] There was therefore no need to examine the high-speed traffic mix and train speeds as these would not add to the fatigue effects calculated for the bridge in accordance with BS 5400 Part 10.

The fatigue assessment was based on the passage of a single high-speed train across the bridge. As there are two lines on each bridge, it is possible that two high-speed trains could cross in opposite directions at the same time, which would increase the stress range calculated as a part of this analysis. Clearly, to model this would have required an unrealistic amount of computer time and therefore it was conservatively assumed that

(*a*) the number of stress cycles is doubled
(*b*) the stress range is doubled
(*c*) some combination of the above.

However, a fairly straightforward analysis demonstrated that the number of occasions when the stress range would actually be doubled is very small, and therefore for all practical purposes the contribution to fatigue from high-speed trains is not of concern.

5. SOME ASPECTS OF DETAILED DESIGN

The detailed design was undertaken to the relevant Railway Group Standards, British Standards and Codes of Practice. In addition to influencing the overall bridge dimensions, the dynamic analysis also brought about the need for the following details.

(*a*) With the bridge on a 30° skew, the preferred orientation of the cross-girders is normally orthogonal to the main girders. This arrangement caused unacceptably high accelerations, however, which were reduced by placing the cross-girders on the skew. The reasons for the higher accelerations with orthogonal cross-girders is not completely understood, but it is believed to be the way in which a relatively small mass is mobilised in responding to the initial impact of a train arriving on the bridge. This approach has since been tried successfully on a number of other bridges.

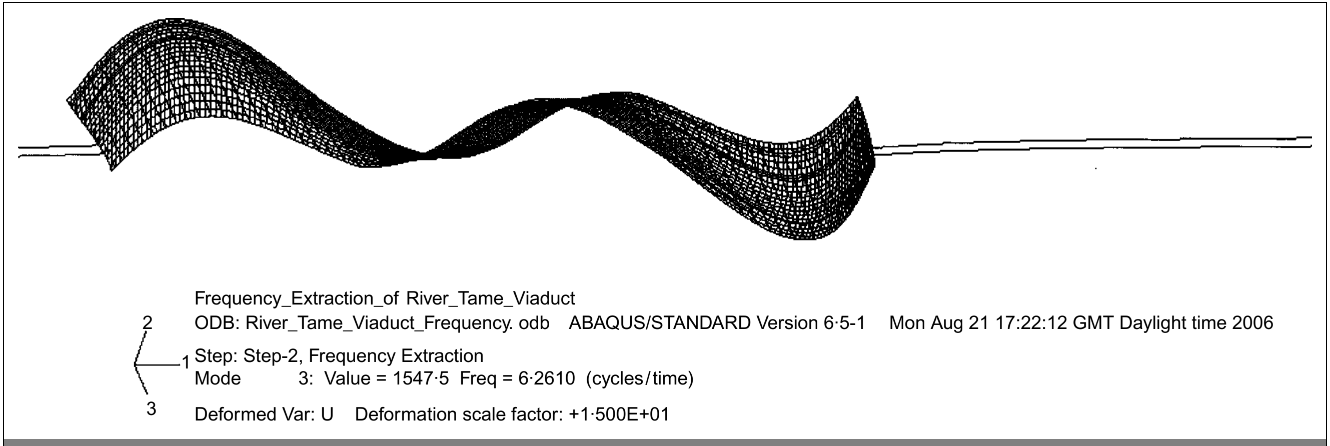

Fig. 5. Typical bridge eigenmode

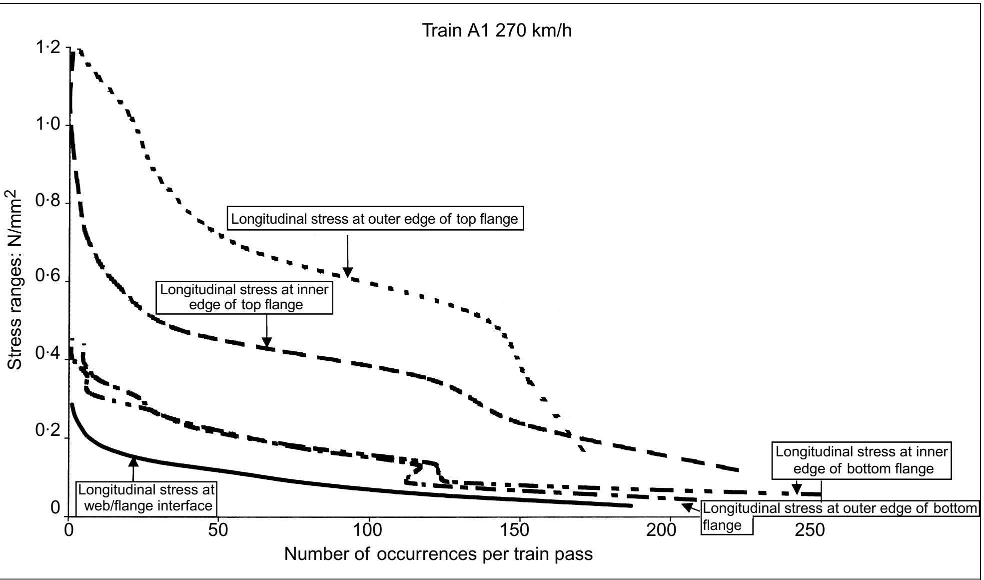

Fig. 6. Typical fatigue stress spectrum

(*b*) It was recognised that the connections to the main girders would be more difficult to fabricate and install, and therefore specialist advice was sought to confirm that the details were feasible. It was deemed likely that the bolts would have to be installed with their heads on the inside.

(*c*) For similar reasons, in addition to placing bearings beneath the main girders, additional bearings were necessary at the supports immediately beneath the tracks. Although the cross-girders supporting these bearings were sized for the static loadings, they were also found to satisfy the dynamic requirements.

6. FLOOD RISK ASSESSMENT

Early in the design development, a hydrological study was undertaken to examine the short-term and long-term hydraulic and hydrological effects of the new construction, temporary works, haul roads and temporary bridge within the floodplain of the river. Later, a more refined flood risk assessment was carried out at the detailed design stage after the twin bridge option had been approved, and the likely temporary works had been explored in greater detail. In addition, by this time, it was clear that the soffit levels of the new bridges would be lower than that of the existing bridge. The assessment concluded that the minimum clearance between the river level and the soffit of the new bridge would be 770 mm for the worst flood condition, thereby complying with the Environment Agency's recommended freeboard of 600 mm.

7. CONSTRUCTION OF THE FIRST NEW VIADUCT

7.1. Advantages afforded by the River Tame site

A major advantage of the River Tame site is that there was space to build the first new viaduct 'off line'; indeed this was a requirement since two additional tracks have to be accommodated alongside the existing ones. The original redundant embankment left by the 1895 bridge replacement was available for the Trent Valley 4-tracking project team (the project) to build the first viaduct, leaving the existing bridge in place until the project was ready to slew the tracks back onto the original alignment.

The 'long-span' design allowed the new river piers and bankseats to be constructed with little interference with the existing structures. The original bridge substructures only required demolition down to just below river bed or ground level, and the new piled foundations were able to be built in largely virgin ground.

As mentioned above, the Environment Agency gave consent to build both access causeways out into the river simultaneously, saving significant programme time. Advantage was also taken of the fact that the existing viaduct's girders allowed the construction activities to be physically separated from the live railway; this reduced the need for possessions of the railway, particularly for ground works and steel fixing.

The 'twin bridge' design solution also allowed the project to keep the railway operational at full line speed at all times (other than planned possessions) during construction.

7.2. Construction sequence

Work commenced in August 2005 with the demolition of the original viaduct's piers (Fig. 7). Sheet-piled cofferdams and temporary embankment retaining walls were then driven, with the retaining walls tied back into the embankment by two rows of ground anchors. During installation of the first set of anchors, slightly greater settlement than predicted occurred under the railway. This was picked up by the track monitoring regime and corrective action was taken. An investigation found that water flushing while drilling had washed away a small but significant amount of embankment material. The drilling technique was modified and track settlement brought back within predicted limits.

Fig. 7. August 2005—demolition begins on the original piers

October brought heavy rain and high water, which flooded the worksite for a week. Once the water had subsided, the pier and bankseat piles were installed using a continuous flight auger method. Piling close to the live railway was carried out during a series of four weekend overnight possessions, using low headroom piling rigs, carefully oriented such that a mast collapse or dropped auger would be unlikely to impact the railway infrastructure. These short-masted rigs required the augers to have segments added and removed as work progressed. Similarly, the pile reinforcement cages had to be installed in two spliced sections. This slowed the work and care had to be taken with scheduling concrete; early problems were encountered when, after delivery, placing of the concrete was delayed and the concrete began to harden in the bottom of the piles before the cages could be fully inserted. It is thought that this was exacerbated by the presence of sand lenses which were penetrated by some pile bores. The sand wicked water away from the concrete, causing a localised initial set sufficient to impede the cage installation.

Installing the piles to the required depths proved difficult and the augers regularly hit refusal in the underlying mudstone stratum at levels higher than the specified pile base levels. Observing the piling works, the project was confident that the piles were achieving embedment in competent material, and subsequently, after discussions between the designer, checker and Network Rail, the foundations were deemed adequate as built.

The superstructure construction commenced in February 2006 (Fig. 8). Each main girder was transported in five sections: a single inner-span section and two sections for each outer span. A 1000 t crane and two 200 t service cranes were used for the bridge lifts.

Owing to the 30° deck skew, the cross-girders were quite simple to lift in, slew and offer up for bolting (Fig. 9). As anticipated at the design stage, however, there was insufficient space in the acute angle between the cross-girder webs and endplates to operate the nut driving tools (Fig. 10), and the affected tension control bolts had to be inserted from the inside. For consistency of appearance, all the cross-girder end connection bolts were installed in this manner.

Fig. 8. February 2006—the first main girder section is lifted in

Fig. 9. Cross-girders and shear studs

Proprietary glass fibre reinforced plastic (GRP) permanent formwork was used for the deck soffit, placed on the cross-girder bottom flanges. Where needed, joints were formed using galvanised expanded metal mesh stop ends over the cross-girders; once the deck concrete had hardened sufficiently, the mesh was partially cut out and grouted over to restore the cover to the steel.

Construction of the bridge deck proceeded according to programme until finally, over two weekends in May 2006, the railway was slewed across to the new deck and the bridge was made fully operational (Figs 11 and 12).

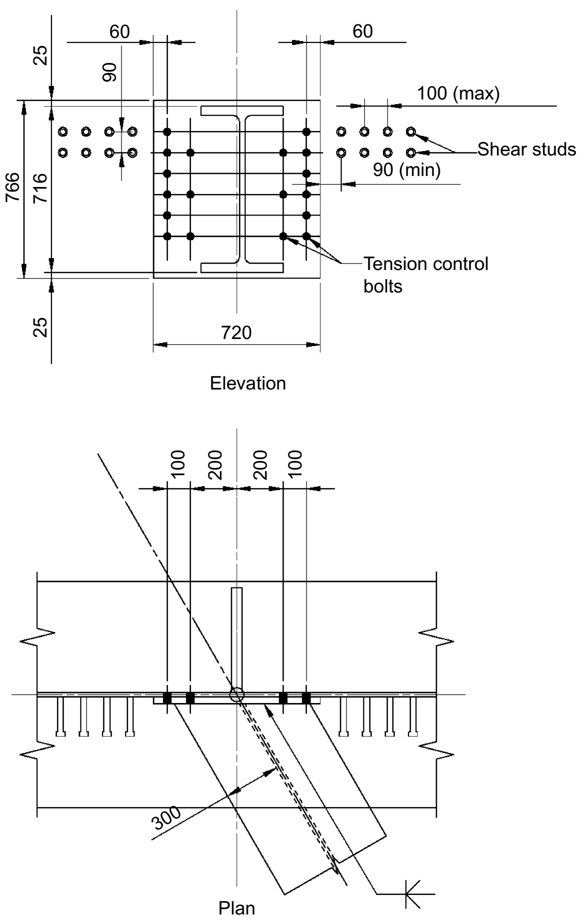

Fig. 10. Cross-girder connection (dimensions in mm)

Fig. 11. May 2006—completed bridge

7.3. Environmental and archaeological considerations

Protection of the river and its ecology was paramount during construction. Protection measures included having spill kits on hand, booms stretched across the river, the provision of filtration/stilling basins for the removal of silt and contamination from the water pumped from the works and the use of pontoons to protect the river when working over the water. Wildlife issues included rabbit infestation, nesting ducks and other birds nesting in nearby hedgerows. Archaeologists investigated the worksite, and although there were no significant ancient historical finds, the project did lift out and deliver a World War II pillbox to the nearby Staffordshire Regiment museum.

A stream to one side was diverted to make way for the widened embankment. The new diversion was provided, to the approval of the Environment Agency, with fish 'resting' areas and an otter holt. These had to be left for several weeks to naturalise before the Project could infill the original stream.

8. CONCLUSIONS

Early discussions with the Environment Agency enabled both the design and a suggested construction methodology to be developed with confidence. The long span layout allowed construction of the new piers and bankseats to be undertaken with minimal interference to the existing structure and river regime.

The twin-bridge configuration chosen for the new viaduct is advantageous. It allows for a phased construction with two railway lines remaining open at all times, and will minimise disruption to the railway during future maintenance.

The large, or complex, FE models used in the dynamic analysis necessitated a means of physical (in addition to numerical) understanding of what was and should be happening in order to state with confidence that the results were both credible and reasonable.

Owing to increasing computer power, it is expected that run times will become quicker and increasingly within the capacity of normal desktop machines. Such analyses are likely to become routine, therefore, and models are becoming increasingly complex.

Peak accelerations are very dependent on the level of damping. Damping is poorly understood, however, and data are limited and

Fig. 12. Aerial view from the north bank. The diverted stream is visible on the right

therefore conservative. It is important to improve this understanding, and to augment the body of available data in order to produce more efficient bridge designs.

9. KEY PARTICIPANTS

Key participants in the design and construction of the bridge were

client: Network Rail
designer: Scott Wilson
checker: Mott MacDonald
contractor: Birse Rail
steelwork: Fairfield Mabey.

REFERENCES

1. Network Rail. *West Coast Route Modernisation Guidance Notes for the Design of Railway Bridges Subject to High Speed Operation, Considering Dynamic Effects Including Resonance.* Network Rail, London, 2004, RT/WCRM/QMS-MP/APP/905.
2. British Standards Institution. *Eurocode 2: Design of Concrete Structures. Part 1-1: General Rules and Rules for Buildings.* BSI, London, 2004, BS EN 1992-1-1.
3. British Standards Institution. *Eurocode 4: Design of Composite Steel and Concrete Structures. Part 2: General Rules and Rules for Bridges.* BSI, London, 2005, BS EN 1994-2.
4. British Standards Institution. *Steel, Concrete and Composite Bridges. Part 3: Code of Practice for Design of Steel Bridges.* BSI, London, 2000, BS 5400-3.
5. British Standards Institution. *Steel, Concrete and Composite Bridges. Part 4: Code of Practice for Design of Concrete Bridges.* BSI, London, 1990, BS 5400-4.
6. Highways Agency. *Loads for Highway Bridges.* Highways Agency, London, 2001, BD 37/01.
7. International Union of Railways. *Loads to be Considered in Railway Bridge Design.* IUC, Paris, 1994, UIC Code 776-1R.
8. Comité Euro-International du Béton (CEB). *CEB Bulletin 187. Concrete Structures Under Impact and Impulsive Loading—Synthesis Report.* CEB, Switzerland, 1988.
9. British Standards Institution. *Steel, Concrete and Composite Bridges. Part 10: Code of Practice for Fatigue.* BSI, London, 1980, BS 5400-10.