Bearing replacement and strengthening of Forth Road Bridge approach viaducts, UK

Abstract

The existing Forth Road Bridge spans the Firth of Forth in Scotland. Construction of the bridge was completed in 1964. The main structure is a three span suspension bridge with a central span of 1006m and side spans of 408m. At each end of the bridge there are two multi-span approach viaducts leading up to the main crossing. The deck of the approach viaducts comprises a pair of longitudinal steel box girders supporting a series of transversely spanning steel girders, both act compositely with a reinforced concrete deck.

The steel girders of the approach viaducts are supported on steel roller and rocker bearings. The bearings are fixed to reinforced concrete portal piers which are founded on rock. These piers vary in height between 11m and 40m. An initial study of the bearings identified that the rollers had locked up due to corrosion and distortion, and the concrete beneath the bearings and elsewhere on the pier tops had deteriorated due to chloride contamination. Assessment showed that structural deficiencies in the pier were exacerbated by both the concrete deterioration and change in articulation. These factors lead to the decision to replace all the bearings on the viaducts.

The original structure was not designed to facilitate replacement of the bearings so the structure had to be strengthened and modified by the addition of jacking stiffeners and corbels to the pier tops. Initial design was carried out to BS5400 leading to substantial amounts of strengthening being required. This was then repeated using Eurocodes to gain better structural efficiency and reduce the amount of strengthening required to the deck and piers. This paper outlines the design of the strengthening and modifications to the bridge to facilitate bearing replacement, together with a detailed description of the design of the temporary works needed to maintain the bridge’s articulation during jacking. Lessons learnt during construction are discussed and the structural benefits of design to Eurocodes are emphasised.

Keywords: Roller bearings, bearing replacement, steel box girders, steel strengthening, Eurocodes, virtual reality modelling.

Introduction

The Forth Road Bridge (Figure 1) spans the Firth of Forth and was completed in 1964. The main structure is a three span suspension bridge. At each end of the bridge there are two multi-span approach viaducts comprising a pair of longitudinal steel box girders with cross girders supporting a concrete deck slab as shown in Figure 2. The approach viaducts carry two carriageways, each with two lanes, and extend from the abutments to the side towers, which are shared with the main suspension bridge. The total width of the structure is 36m.

The box girders rest on steel roller and rocker bearings on reinforced concrete portal piers, varying between 11m and 40m tall, founded on rock. The articulation of the two viaducts is shown in Figures 3a and 3b. Locations with roller bearings allow for horizontal movement.
The most feasible options for replacement of the rollers were pot bearings, sliding bearings or replacement roller bearings. A pair of pot bearings was eliminated early on as an option because there was insufficient room to achieve an adequate lever arm between them to resist the torsional moments attracted by the boxes. A single pot bearing was also considered, occupying the full width of the diaphragm, but this would have required significant widening of the pier top to accommodate it. Using smaller pot bearings, partially loading the width of the diaphragms, would have been geometrically feasible but would have required strengthening to all the diaphragms; this would have been very difficult because of the extensive services passing through them. Moreover, a single pot
Figure 3. (a) Articulation at south viaduct, (b) Articulation at north viaduct

Figure 4. Roller bearings, (a) Typical roller bearing, (b) Roller at north side tower

Figure 5. Typical rocker bearing
Steel box girders

A typical internal arrangement of a steel box is shown in Figure 7. The steel box girders were initially assessed using BS5400-3 and BD 56, which found significant overstress in the existing structure. Particular areas for concern were the resistances to shear and shear-moment interaction. In addition, the shape limits for the stiffeners (to prevent torsional buckling) were

Assessment of box girders and piers – as built

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The current bearings could not transmit torsional reaction from the box girders, necessitating strengthening to the cross girders at supports.

Replacing the current bearings with new roller bearings was also considered. This would involve no modifications to the diaphragm and there would be no change to the current articulation of the structure. However, the new bearings would have required a larger diameter than the original bearings in order to comply with BS EN 1337-4 and would not, therefore, fit within the available vertical space between the pier top and the box girder soffit. A higher material grade (with yield strength equal to or exceeding 690 MPa) was also considered to enable the diameter to be reduced, but sufficient confidence could not be obtained in the toughness of the steelwork that would be obtained.

The final solution therefore was to adopt a sliding bearing. This minimised the changes to the original articulation, whilst avoiding the potential problems with brittle materials. To minimise the effect on the existing structure, the bearing was detailed with the sliding surface on the lower surface so all the eccentricity occurred on the pier and not on the box girder diaphragms; the previous roller bearings shared the eccentricity between the pier and box. This had a negligible effect on the overall pier load effects but was beneficial to the box girder diaphragms. It was recognised that this detail could lead to durability problems so, to prevent ingress of dirt on the sliding surface, the bearings were detailed with a protective gaiter.

For the rocker bearing, a similar process was completed for the selection and replacement rocker bearings were selected. The new bearing types are shown in Figure 6.

Figure 6. New bearings, (a) Rockers, (b) Sliders
not met. To avoid strengthening the boxes, Eurocode 3 was used to reassess the boxes, particularly BS EN 1993-1-5. This re-assessment eliminated the over-stress under shear and shear-moment interaction, but the torsional buckling requirements for the stiffeners were not met. The overall amount of strengthening required to the box girders was correspondingly reduced, but not eliminated. As Eurocodes are based on specification of modern materials, it was important to investigate the impact of any departures from the material properties inherently assumed and take this into account during the assessment. Most relevant to this was steel ductility as the steel to BS968:1962 used in the construction of the Forth Road Bridge possessed lower ductility than modern steels. Coupon testing of steel from the boxes confirmed that the steel possessed adequate ductility to satisfy Eurocode requirements. Reference 1 gives further details of considerations for using Eurocodes for assessment.

Concrete piers

The pier tops were also assessed using the strut and tie rules in BS EN 1992-1-1, as strut and tie analysis is not adequately covered by BS5400 Part 4 or BD 44. The existing pier reinforcement consists of mild steel plain round bars. Typical piers and the strut and tie models used for their assessment are shown in Figure 8. This analysis showed that the existing reinforcement was not adequate to resist bursting loads in the pier tops, but the pier was adequate if the tensile strength of the concrete was invoked. Although this explained why there was no distress in the existing condition, it was not considered an acceptable permanent solution considering the ongoing concrete deterioration.

The piers were also assessed for global behaviour, including the longitudinal loading due to temperature expansion and contraction on the piers with pinned bearings. Adequacy of these piers was demonstrated by considering cracked behaviour in accordance with the recommendations given in BS EN 1992-1-1 clause 5.8.7; without considerations of cracking, the piers were stiffer, attracting too much load and their resistance was exceeded.

Figure 8. Analysis of typical piers with strut and tie analysis

Solution for bearing replacement – corbels and steelwork

General requirements

The bridge is a Category A listed structure and is highly visible, so all parts of the strengthening work to facilitate bearing replacement had to be in keeping with the existing structure and were subject to approval by the two adjacent planning authorities in conjunction with Historic Scotland. However, all parties accepted that the visual considerations had to be balanced with the structural and safety considerations of the work as discussed below. A key aspect of the scheme design was that it should also facilitate future bearing replacements, should they become necessary.

Scheme outline

Replacement of the bearings necessitated jacking up the box girders to release the existing bearings, but there was insufficient space for jacking at the pier tops and no suitable location on the
box girders to apply the jacking forces. The solution chosen was to add corbels to the tops of the piers to provide adequate space for the jacking equipment and to add jacking stiffeners to the box webs. Corbels were selected because they provided a permanent solution for bearing replacement, whilst also being more economic than options involving temporary propping. The corbels are discussed in later sections of this paper. In addition, the design needed to maintain the articulation of the bridge throughout the replacement process, which meant providing longitudinal fixity at rocker bearings and a controlled release of unintended force at roller bearing positions.

A key feature of the jacking scheme was the use of four jacking points at each box girder support. This was necessary for a number of reasons including lack of available room at the pier top in line with the diaphragms (because of the limited available clearance to the permanent moving maintenance gantry) and the presence of cross girders in line with the diaphragms limiting the available height for jacking stiffener addition in line with existing diaphragms.

A typical sequence for a bearing replacement is as follows:

a) Install internal steel strengthening
b) Install jacking stiffener
c) Cast new corbel
d) Install restraint system
e) Install jacks and jack up to remove load from bearing
f) Remove existing bearing
g) Repair concrete under bearing as required
h) Position new bearing
i) Lower box and transfer load to new bearing.

As the construction sequence was complex and the Designers’

Risk Assessment identified non-compliance with it as a significant risk, a virtual reality (VR) model was used to highlight the different construction stages in the project in order to minimise the risks of errors being made in the construction sequence. This followed successful use of a similar model on a previous bearing replacement project. The main features of the bearing replacement scheme are shown in Figure 9, extracted from the VR model.

**Concrete corbels**

The concrete corbels were designed to be permanent extensions to the existing pier and served two purposes:

- To provide adequate space to position jacks
- To facilitate installation of permanent bursting reinforcement in the pier tops.

The corbels had to carry the load from the temporary jacks to allow the permanent bearings to be replaced whilst also allowing the pier top concrete to be broken out and recast where needed to rectify the deterioration. For this reason, the corbels were cast 300mm below top of pier level so that pier top breakout would not undermine the new corbel reinforcement. Space and aesthetics were the major constraints in the design of the corbels. Their width in the longitudinal direction of the bridge was dictated by the need to tie in with a pre-existing concrete...
feature extending up the sides of the piers as shown in Figure 10. Their width perpendicular to the bridge span on its outer face was limited by the clearance to the existing maintenance gantry.

A typical final installed corbel is shown in Figure 11.

The corbels were designed using strut and tie models of the generic form shown in Figure 12, which necessitated the anchoring of new reinforcement into the existing pier. This required holes to be drilled into the existing structure and new reinforcement resin grouted in. Figure 12 shows typical reinforcement resin anchored through the thickness of the pier prior to drilling for the addition of the pier transverse reinforcement. This transverse strengthening reinforcement extended up to five metres into the pier because, in addition to functioning as part of the corbel, it was also designed to replace the existing transverse top mat of reinforcement while this was exposed during the repair work of the concrete at the top of the pier. The corbel reinforcement also provided bursting reinforcement for the permanent condition, which had been found by the assessment to be deficient.

It was accepted that some bars would inevitably be damaged during coring operations, so a coring protocol was developed in advance of construction to set out the numbers and locations of bars that could be damaged without remedial work being required. This allowed site staff to make the decision on what to do when an existing bar was hit and to update their strategy for the remaining cores.

The main reinforcement in the corbel needed to pass through the full width of the pier and be fully anchored at each end. The bars could not be detailed with a bend at both ends as they could not then be threaded through the core hole.

Figure 11. Corbel addition on site
The steelwork design was carried out in accordance with Eurocodes, specifically:
- BS EN 1993-2 (Bridges)
- BS EN 1993-1-5 (Plated structural elements)
- BS EN 1993-1-8 (Design of joints)

At the end supports, the services were found not to be ducted where they passed through access holes in the box webs and diaphragms. The risks of carrying out work adjacent to these services, comprising fibre optic cables, was considered too great so it was decided to move the new longitudinal stiffeners to the external faces of the webs at these locations.

Steelwork

To minimise the hazards associated with working in the confined space environment of the steel box and to avoid damage to the extensive existing services within the box, the main components of the strengthening needed for jacking were added to the outside of the steel box girder. This comprised single-sided box section jacking stiffeners (Figure 14) similar to those developed in reference 2 where the design methodology is described. A number of alternative arrangements were provided for these box stiffeners to give flexibility in the location of the bolted attachments to the webs. Weathering steel was used for the jacking stiffener because of the lack of access to its internal surfaces. The only internal steelwork strengthening was additional bolted longitudinal stiffeners (to strengthen the webs before drilling the holes for the jacking stiffeners) and some additional plating to existing longitudinal stiffener angles (required to prevent torsional buckling). The new longitudinal stiffeners were rolled steel angles connected to the webs through one leg. Welding was not used for the connections because of concerns over the ability to weld to the existing steel without laminating it and because of the confined space environment inside the boxes.
Strengthening of the existing longitudinal web stiffeners on the internal web faces was also replaced by additional new longitudinal stiffeners on the external web faces. Moving the stiffeners to the outside of the box in this way increased the steelwork quantities but improved buildability as the plates did not need to be brought into the box. External stiffeners were considered elsewhere but proved to be less convenient because of the difficulties of getting continuity across the jacking stiffeners at intermediate piers, as was needed in the design.

Restraint systems

The articulation of the bridge needed to be maintained during bearing replacement. Since the longitudinal forces at fixed bearings were too large to take in shear on the jacks, separate temporary restraint systems were provided to connect the bridge superstructure to the substructure during the bearing replacement process. The form of the restraints varied according to the location but at the intermediate piers, longitudinal restraint was provided by four steel brackets positioned in pairs each side of a box, one pair either side of the pier, anchored to the pier using Macalloy bars passing through holes in the pier. Transverse restraint (to the smaller forces from wind and skidding loads) was provided by the jacking stacks themselves. This required the use of a guided temporary bearing on one of the jack stacks at a box support location to resist the shear, with free bearings at the other jack locations. Similar systems were designed for the abutments and side towers; these are not described here.

The purpose of the restraint system at rocker bearing locations was to fix the bridge piers to the superstructure throughout the bearing replacement whilst the fixity of the permanent bearings was temporarily released. The steel brackets at intermediate piers were of the form shown in Figure 15 with a horizontal jack connecting the top of each restraint to the adjacent jacking stiffener base plate. The jacks allowed the differential horizontal movement between the superstructure and substructure to be controlled under the small deflections occurring in the steelwork itself. The system in Figure 15 was the arrangement for Pier S3 where two simply supported box ends landed on a shared pier and hence an additional set of jacks and tie bars was provided so the restraint system could both push and pull. These additional features were unnecessary at continuous box locations as the restraints only needed to carry forces in one direction. The horizontal restraint jacks were engaged before lifting the superstructure and remained engaged during the bearing replacement until the girders were lowered down onto the new bearings.

The restraint systems at roller bearing locations were similar but were provided only to control the release of force locked into the roller bearings. It was known such forces existed from earlier pier monitoring. Lifting of the superstructure under these conditions could have caused a sudden release of horizontal force and a springing back of the pier in a potentially dangerous sudden movement. The provision of jacks allowed this force and movement to be released in a controlled and safe manner.
In all cases, generous allowance for eccentricities was made in the design of the steelwork because aligning the temporary restraint brackets, whose position was fixed by the locations of the core holes for the Macalloy bars, with the jacking stiffener base plate stiffening had significant tolerance.

As a result of this provision of temporary fixity, no limit was set on the number of piers which could be jacked up at any one time.

Design of jacking system

The jacking scheme utilised two jacking points per bearing at each abutment and side tower and four jacking points at other pier locations. The jacking system at each jacking point comprised three jacks supporting either a transversely guided or free pot bearing; the initial concept utilised only one large jack at each jacking point but this was modified once on site to suit the equipment available – Figure 16a. This ensured that the relatively small transverse forces from the deck (due to wind and skidding) were taken through the guided bearings and then through the jacks to the piers. Longitudinally, the bearing transmitted no loading to the jacks (other than from friction at the roller bearing piers). Longitudinal forces at rocker piers were carried by the temporary longitudinal fixities described previously because the shear forces were too large to be carried by the jacks and additionally, the uneven bearing pressure resulting on the jacks from the longitudinal force would have been too great for the concrete.

At locations with four jacking points, the load at each jacking point after lock off was potentially very uneven under live loading due to the torsion attracted to the boxes and the tendency to uplift on the “back span” jacks where only one span was loaded. To prevent uplift at a jack position and to distribute the loads between them more evenly, thus minimising the strength requirements for jacking components and box strengthening, the stiffness of each jacking system was reduced by incorporating an elastomeric pad between the jack and temporary bearing within a further bespoke bearing capable of transmitting transverse shear. The assembly formed by these three components was referred to as the ‘jacking stack’ and is shown in Figure 16b undergoing a load test.

The determination of the required stiffness for each jacking stack was carried out by linear finite element analysis of the entire south viaduct. Box girders, transverse beams, diaphragms, internal stiffeners and concrete slab were all modelled with thick shell elements, while the jacking stiffeners were added as beam elements. Figure 17 shows the distribution of forces between the four jacking points as a function of elastomeric pad stiffness at pier S4. The jacking sequence included the four jacking points as a function of elastomeric pad stiffness at pier S4. The jacking sequence included the jacks being hydraulically linked for dead loads and subsequently locked-off for live loads. A maximum stiffness of 107 kN/m was needed to prevent uplift at a jack. The upper line shows the total force in all the jacks. It can be seen that below a stiffness of 105 kN/m, the forces in the jacks were very even but the flexibility is such that load was shed to adjacent piers, which was undesirable. The vertical flexibility under live load would also have compromised the ability to grout below the new bearings. Full 3D modelling was essential to model the redistribution of torque also caused by the softer bearing support; this slightly increased the stress resultants in the support cross girders.

The final design stiffness of the jack stack was tuned to a value between 108 kN/m and 107 kN/m so that uplift could not occur under any load combination and load was shared economically between jacks, but the vertical displacement under load would not be so great as to disrupt grouting beneath the permanent bearings. That latter was set at 0.2mm and this proved to cause no problem during grouting.

Cathodic protection

The inspection and concrete test data indicated that the cause of concrete deterioration was chloride induced corrosion, possibly from the initial use of de-icing salts in the first twenty years of the bridge’s life. The chloride ion content at the majority of pier tops and both abutments was in excess of the threshold value of 0.3% by mass of cement and the ‘diffusion’ calculations suggest that the critical level at the depth of reinforcement would be reached in the next 5 to 8 years. Action was therefore required to arrest continued deterioration. Various repair options were considered but the application of cathodic protection (CP) to the piers, side towers and abutments, together with minimal concrete repairs solely to the delaminated areas, was evaluated to be the most cost-efficient solution for the sub-structure.

An impressed current (ICCP) system was selected with design undertaken in compliance with the recommendations of references 3 to 5. Three types of anode systems were used in the design: titanium mesh based anode to be installed at the interface of concrete substrate and new corbels; titanium ribbon based anodes to be installed within slots cut into the concrete cover; and titanium ribbon based discrete anodes designed to be installed within holes drilled into the structure. Reference electrodes for monitoring the performance of each anode zone were also included.

The area covered by each anode zone was designed to be independently powered by an integrated power supply and data acquisition unit, which will provide the facility for controlling the voltage and current to each zone; and monitor/control the performance of the CP system. All units are installed within two
networks (one for each approach viaduct). On each network all units were detailed to be operated from a single main control unit either locally on site or remotely via a modem connection.

**Conclusion**

The replacement of the bearings on the Forth Road Bridge is a very significant undertaking because of the lack of provision for this eventuality in the initial bridge design. The solution developed, involving modification of the structure to provide permanent jacking points on superstructure and substructure, now makes any future bearing replacement or refurbishment a much simpler, quicker and cheaper operation. The permanent modification to the structure, by addition of corbels to the piers and external stiffening to the box girders, was carried out sympathetically to the original design and under the scrutiny of Historic Scotland.

A number of measures were taken to minimise the cost of the modification works and could be considered for future bearing replacement schemes. These included the use of Eurocodes for the assessment of the existing structure and design of the new works, the use of external stiffening to improve buildability and the use of elastomeric pads incorporated in the jacking system to give a better distribution of jack loads after lock off and hence reduced demand on the new stiffening and concrete corbels.

Figure 16. Jacking Stack, (a) Concept with single jack, (b) Final three jack system undergoing load test
Figure 17. Results of sensitivity analysis on elastomeric pad stiffness

References