RIVERSIDE EXPRESSWAY – STRUCTURAL SUPPORT FOR BEARING REPLACEMENT

Rob North¹
MEng, CEng, MICE, RPEQ

Abstract
A number of elastomeric bearings on Brisbane's Riverside Expressway between the Captain Cook Bridge and the Alice Street Ramps required replacement. This task was complicated by insufficient space to allow placement of jacks under the girders and deep soft soils of the banks of the Brisbane River, which limited the options for jacking using ground support.

This paper describes the structural design of temporary works for the girder jacking system with the constraints and difficulties encountered, demonstrating the importance of providing adequate girder jacking provisions as an integral part of the permanent works design.

Introduction
Brisbane’s Riverside Expressway (REX) is at the northernmost end of the Pacific Motorway M3, which is today an integral part of Brisbane’s motorway network. The REX was opened in 1976 and is used today by an average of 91,000 vehicles per day (Figure 1). In 2007 during structural inspections, QTMR identified that many of the bridge bearings required replacement, and subsequently commissioned Parsons Brinckerhoff (PB) to design and detail the temporary works support structures to enable jacking of the bridge decks and replacement of the bearings.

The temporary works design commission included:

• options assessment
• detailed design of the selected option
• deflection analysis for all spans to ensure adequacy for the jacking system requirements
• shop detailing of steelwork
• replacement bearing assessment and selection.

This paper describes — the constraints placed upon the design by the structure geometry; how these constraints were overcome; the modelling and analysis work undertaken to reduce the risk of damage to the structure during jacking; and the detailed deflection investigation undertaken to both minimise deflection of and to provide a baseline for monitoring the temporary works during jacking.

¹ Rob North is a senior structural engineer with Parsons Brinckerhoff, Brisbane
Design constraints
The existing design of the REX structure and the nature of the site posed a number of constraints to the design of any jacking support system. These constraints are summarised as follows:

- **Soft soil founding conditions** — The soil conditions at the site are soft alluvial material of significant depth adjacent to the Brisbane River, with the sediments overlying competent bedrock found at 20-30m below ground level, which is understood to be the founding level of the pile foundations. Geotechnical information was provided on the existing bridge foundation drawings and was used in the analysis. Evidence of the low bearing capacity of the soil is reflected in the original design decision made to suspend the car park wall panels/screens from the deck instead of supported on the ground, and that the ground beam/drainage channel beneath the wall panels has sunk approximately 50-100mm in places under its own weight since construction.

- **Lack of pile cap** — The substructure of the REX in this location consists of octagonal driven piles which take the loads directly from the headstock to bedrock with no intermediate pile cap or pier structure. Combined with the poor soil quality, this means that the only stiff support available is the headstock.

- **Diaphragm not positioned over the headstock** (Figure 2) — The girders are connected adjacent to the bearings by a 178mm wide diaphragm or cross girder, with the inside face of the diaphragm approximately in line with the outside face of the headstock. Therefore it is not possible to place a jack on the headstock directly underneath the diaphragm.

- **Low clearance between diaphragm and headstock** — The overall clearance between the top of the headstock and the underside of the diaphragm is approximately 265mm to 290mm. Therefore any false work steel beam placed on top of the headstock is limited in depth.

- **Minimal cantilever at end of headstock** — The outside of the edge girder is approximately lined up with the outside edge of the headstock. Therefore, if the headstock is used as a support from which to jack the deck, no support would be available outside the edge girder (Figure 3).

- **Limited access for ground support** — The low clearance beneath the structure made access for any significant ground improvement difficult.

- **Existing car park features** — The existing car park under the REX which services QUT students required that access be maintained with the exception of short term closures. The car park has ticketing machines and other infrastructure which made alternate ground support difficult to construct.

![Figure 2. Typical REX headstock](image-url)
Figure 3. Headstock support and diaphragm arrangement

Figure 4. Elevation showing support beams clamped around headstock
Options investigated
PB investigated various design options for the jacking frames, which were explored in ‘optioneering’ workshops in conjunction with QTMR. The options presented and discussed were:

- **Option 1** — Jacking from headstock with a stress-bar clamping system (Figure 4). This option utilises the headstock as a stiff structure to jack from, with the load path from girder to headstock similar to the existing structure i.e. in bearing on top of the headstock. This option allows differential loading to be applied either side of the headstock, allowing for more flexibility in the jacking procedure, due to the clamping action that is applied.

- **Option 2** — Jacking from headstock with no stress-bar clamping. This option was similar to Option 1, in that the support beams bear directly onto the top of the headstock. However the difference is that there is no torsional restraint provided by clamping the structure around the headstock. Removing the clamping allows for more flexibility in positioning the support beams, as the support position is limited only by the girder position, not by both girders and piles above and below the headstock. Therefore the cantilever to the edge girder could be reduced.

- **Option 3** — Jacking from headstock from brackets anchored into headstock (Figure 5). This option would involve installing a large diameter shear pin, or series of pins, into the side of the headstock, and attaching a steel bracket. One support bracket would be installed beneath each jacking position.

- **Option 4** — Jacking operations supported from the ground. This option involves a steel support structure supported on proprietary struts founded on pads bearing onto the ground.

Figure 5. Section through headstock showing support brackets
The consensus of the ‘optioneering’ workshops was that Option 1 was preferred and detailed design should commenced. This option provided numerous advantages:

- load path to supporting structure similar to existing
- positive restraint provided by clamping action
- flexibility of application to suit various pier arrangements
- ability to accommodate out of balance loads either side of the headstock.

Option 1 avoided several undesirable aspects of the other options considered:

- poor quality ground with unknown bearing capacity and potentially excessive settlement
- large number of holes for supporting brackets drilled into headstock
- instability of the system under differential load either side of the headstock.

The structural arrangement of the selected temporary works Option 1 is illustrated in Figure 6.
Detailed design

The detailed design of the chosen option presented several engineering challenges, which will be discussed below.

Limited depth of hanging beam — as described above, the geometry of the structure is such that any beam placed on top of the headstock is limited to a depth of approximately 265mm. The actual space for a beam is much lower than this, assuming a bearing pad of approx 30mm to 50mm is required beneath the beam and a stress-bar anchor plate and nut is required above. In order to fit in the components of the stress-bar above or within the hanging beam, the stress-bar size was limited to 40mm diameter, which is significant in terms of the elongation of the stress-bar under load, which is discussed in a later chapter.

A standard rolled steel section for the hanging beam was quickly ruled out due to inadequate strength. A solid steel bar was considered unsuitable due to its weight and the difficulty associated in manoeuvring the section into place on top of the headstock.

In the final design a fabricated section was adopted which incorporated a 40mm thick bottom flange, which provided the bulk of the bending strength of the member and acted as the stress-bar bearing plate. The webs were formed by two 20mm thick plates, and 20mm thick plates formed the top flange, which was made non-continuous to make welding the section easier. The ends of the section were cut at 45° to allow access to the stress-bar nut (Figures 6 & 7).
The section was assessed for bending and shear stresses using simple static methods. To investigate local stresses in the section and under the bearing plates and to check deflections of the hanging beam and bearing pads, a finite element (FE) model was built. As shown in Figure 8, the model used brick elements to represent the concrete headstock, steel hanging beam and the mortar and timber or rubber bearing pads, and beam elements and rigid links to represent the stress-bars and anchor plates. This model was used extensively to investigate load effects at the top of the jacking frame, as will be described in the following sections.

Bearing stress on top of headstock under hanging beam — given the limited room on top of the headstock to fit the hanging beams, consideration was given to limiting the compressive and bursting pressures on the bearing plates of the hanging beams. The first check undertaken was to calculate the allowable bearing pressure in accordance with clause 12.3 of AS5100 part 4, assuming that no special confinement reinforcement was provided in the headstock. Although the headstock has conventional reinforcement, this assumption was considered a necessary conservatism to give confidence that the headstock would not be damaged during jacking.

Using a compressive strength of 32MPa, this gave an allowable bearing pressure of 19MPa. To find the applied bearing pressure, the FE model shown in Figure 8 was again utilised. As the hanging beams bend under the load from the stress-bars, the bearing pressure would vary across the bearing plates, with potentially very high localised pressures under the front edge. Timber packing on high strength grout levelling pads was detailed to form the bearing surface. Timber packing is commonly used in heavy lifting applications, as it crushes under pressure, spreading the load out and reducing localised pressures. It can be demonstrated using the finite element model as shown in Figure 9 that the timber packing, when compared with a rubber bearing pad, significantly reduces bearing pressure on the concrete headstock.

The stress outputs shown are for materials representing rubber and timber, with elastic modulus and Poisson’s ratio varying. The modelling showed that rubber is not sufficiently compressible to spread vertical load sideways, thus giving larger peak bearing pressures when compared to the more compressible timber. The design specified that timber packing was to be used for one jacking cycle only and then replaced.
Figure 8. Finite element model of hanging beam and bridge headstock

Figure 9. Rubber vs. timber packing bearing stresses

Rubber bearing pad - peak Von Mises stress = 28.1 MPa

Timber bearing pad - peak Von Mises stress = 20.4 MPa
Additionally the FE model detailed the principal stresses at the face of the headstock. A check was made to verify that the bursting stresses were not large enough to cause local spalling. These stresses were assessed against the tensile capacity of the concrete and found to be acceptable. An end sectional view of the headstock is shown in Figure 10 indicating the principal stresses taken at the centreline of the stress-bar/hanging beam. The arrows indicate the principal direction of the stress, and the circled area shows the zone of tensile bursting stresses at the headstock face.

Deflection at jacking points, in particular at the edge girder — early in the design phase it was recognised that the deflection of the jacking system would be a significant constraint on the design. There were two particular concerns that needed to be addressed by the design and operation of the system. Firstly, due to the fact that the deflection of the support beam would be non-uniform, the operation of the jacks would have to be closely controlled and monitored to ensure an even lift of the bridge to reduce the risk of overstressing the deck end diaphragms.

Secondly, excessive deflection and rotation of the supporting frame could cause instability of the jacking frame or bridge deck. The initial estimate of the outside or edge girder deflection was in the order of 10mm (Figure 11), with the majority of the deflection caused by rotation of the whole system. This is due to the relative elongation of the stress-bars along the length of the jacking beam causing the beam to rotate at the more heavily loaded cantilever. The use of the stress-bar was essential in order to clamp the system around the headstock to give stability, and the size of the stress-bar was limited by the space available in and above the hanging beam (as described earlier), therefore other options were explored to limit deflection of the system.

![Figure 10. End view of principal stresses in the headstock](image)

![σ11 stresses](image)  ![σ22 stresses](image)  ![σ 33 stresses](image)

**Figure 10.** End view of principal stresses in the headstock

![Figure 11. Deflection under edge girder jack for 770mm and 1240mm cantilevers](image)

**Figure 11.** Deflection under edge girder jack for 770mm and 1240mm cantilevers

2 For details on the jacking system refer to the article in this publication titled, Synchronised heavy lifting system for bridge structures.
The option of using a stiffer support beam was investigated, but did not significantly reduce the deflection at the edge girder (Figure 9). This graph shows deflection at the edge girder when the stiffness of the supporting jacking beam is increased. The beam section used in the final design, 800WB168, has an Ixx of 2500 x 106 mm4. For reference, the heaviest welded beam section available, a 1200WB455 has an Ixx of 15300 x 106 mm4.

Therefore, at this stage of the design process, further options were investigated to reduce the edge girder deflections during jacking to a value under 10mm. The options considered included the following:

- **Anchoring of Support Beam Cantilever** — This was investigated in detail but the load generated in the anchors under the edge girder was found to exceed the structural capacity of the headstock. Strengthening of the headstock would be required but would be difficult to achieve as there was a significant shortfall in suspension reinforcement at the end of the headstock. This suspension reinforcement would be difficult to retrofit in a strengthening application, and this option was therefore rejected.

- **Pile Option** — A pile option was considered but rejected due to lack of access. In addition, the elastic shortening of the pile under the weight of the edge girder (a pile would be over 20 m in length to founding rock) would likely exceed the deformation of the steel support system.

- **Prop to Pile Cap at Ground Level** — This option involved constructing a reinforced concrete pile cap connected to the existing piles for supporting a prop to the girder. The combined girder reaction of the edge girders is approximately 150 tonnes, and the detail required to connect the pile cap for this magnitude of load was found to be impractical, in particular due to the close spacing of strand in the existing piles which would prevent the post installation of connecting reinforcement between the pile and pilecap. Also, access to the underside of the REX would be difficult, making this option expensive as well as technically challenging. It was therefore rejected.

- **Move edge girder jack to diaphragm** — The option of reducing the cantilever of the jacking beam was considered. The tension reinforcement in the bottom of the diaphragm consists of two 32 mm diameter bars, which are continued into the bottom flange of the edge girder via couplers. However, these bars appear to be only partially developed with a relatively short embedment length into the edge girder providing less than 50% of the required development length. In addition to this, the diaphragm is only 203mm wide and the bearing stresses will be difficult to retain under the allowable limits for concrete without confinement reinforcement. Therefore this option was discounted due to the risk of damaging the structure.

- **Reconfiguration of the outside jacking beam and jack/safety stool** (Figure 12) — This option reduced the actual cantilever dimension from the jack centreline to the adjacent hanging beam centreline. This was achieved by splitting the restraint beam to the underside of the headstock so that it was around the outside pile instead of behind it, and also reversing the jack and the safety stool. These two simple minor adjustments significantly reduced the deflection of the edge girder jack and were incorporated into the final design.

Although these design refinements were minor in nature, they allowed the main aspects of the original concept to be retained whilst reducing the edge girder deflections to typically 5mm to 6mm with a maximum of 7mm. To improve the stability of the jacking frame, the following provisions were incorporated into the design:

- the outside jack was specified with a rotating head, to allow the bearing pressure on top of the jack to remain uniform under any rotation.

- the packing plates on the rotating jack head were detailed with polished stainless steel plate and PTFE sliding surfaces, to prevent a lateral load being applied to the head of the jack as the frame rotates

- the jack was detailed as bolted to the support beam to resist any lateral load that could be applied to the jack as it rotated

- the tapered plates under the jack were detailed such that the jack under the edge girder is vertical under full load.
in order to facilitate the close monitoring of the jacks during operation, a detailed breakdown of the deflections at each jack position at each pier was provided by the design team. Overall, 17 piers were scheduled for jacking, totalling 169 jacking points. In order to provide the deflection information in time for the start of the jacking operation, the analysis models used to calculate deflections were simplified into line beam models, with spring supports representing the hanging beams based on unit deflection values gained from the first FE model described previously. The deflection values gained from the simplified models were calibrated against a more detailed FE model of the whole system, representing a typical final arrangement (Figure 13). This second FE model used plate and beam elements to represent the various elements making up the whole jacking frame, and was also used to determine peak stresses in the steel members. It was also possible to further break down the deflection into contribution from stress-bar elongation, hanging beam deflection and jacking beam rotation, all based on the axial load in each stress-bar. The detailed deflection summary produced allowed the site team to implement a monitoring plan for the jacking works. Following finalisation of the design concept, the design drawings and associated shop drawings were produce to enable fabrication.

**Conclusion**

To date approximately half of the bearings have been successfully replaced, with the works staged in conjunction with the required closures of the REX.

This project has been a good example for the need to incorporate future jacking provisions into new bridge designs because of the various constraints encountered when retrofitting a design to an existing structure and the large costs of fabrication and installation of the support steelwork.