A-frame rocker bearing replacement at Humber Bridge, UK

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Routine inspections of the A-frame bearing rockers on the Humber Bridge, UK, led to concerns over lack of articulation and potential premature wear. Following design optioneering, an innovative scheme was selected to replace the main span's A-frames with a pair of vertically orientated pendels and a wind shoe at each tower, thus separating out the horizontal and vertical forces, with benefits for construction and maintenance. This paper describes the design and £4 million, 2 year replacement project. The works were carried out in a very confined environment, and required significant strengthening to the existing bridge to accommodate the new permanent and temporary design elements. The project was completed in 2015, all under live traffic, and has ensured the future integrity of this world-class structure.

1. Introduction

Opened in the UK in 1981, the Humber Bridge carries the A15 dual carriageway over the Humber estuary between Hessle, East Yorkshire and Barton, North Lincolnshire, UK. With its 1410m suspended main span, it was the world's longest single-span bridge until 1997 (Figure 1).

The highway is carried by a 4.5m deep steel deck box. The carriageway is discontinuous at the reinforced concrete towers, passing onto expansion joints. Each end of the deck boxes at the towers and anchorages were supported by pairs of steel A-frames (Figure 2). These elements formed a pinned connection at their apexes – to the deck box at the top and to the tower and anchorage abutment at the bottom.

The A-frames provided several functions

- free longitudinal movement of the deck boxes while the supporting catenary above changes shape as traffic crosses the bridge
- free longitudinal movement of the deck boxes against other effects, notably temperature expansion and contraction, static and dynamic wind loading
- being of low plan torsional stiffness, free plan rotation of the deck box ends under wind loading
- torsional restraint for the deck box against unbalanced carriageway loading – one A-frame acting in compression, the opposed A-frame in tension
- horizontal restraint to the ends of the deck boxes under wind loading

- vertical load path for a nominal amount of dead load not carried by the hangers and for a component of traffic load up to approximately 90m from the deck box ends.

The total longitudinal displacement accommodated by the main span A-frames was approximately 2 m.

Through routine inspections, the bridge owner Humber Bridge Board (HBB) had noted degradation to the main span A-frames. Gaps that were present between the rocking A-frame and its supporting structure had closed, suggesting wear of the pin bearings was taking place (Figure 3).

In 2011, HBB commissioned designer Arup to investigate further, recommend a single preferred option for refurbishment or replacement and undertake detailed design. This paper details this process and describes the resulting 2013–2015 £4 million construction work by contractor C Spencer.

2. Investigation

2.1 Inspection and monitoring

Upon inspection of all 12 A-frame bearing rockers, only the four to the main span were noted to be heavily worn. The two rockers at Hessle tower were particularly heavily worn. Restraining keys intended to prevent rotation of the pin relative to the tower portal beam and deck box brackets had been removed. Although the bearing arrangement prevented full inspection, it was thought and later proven on demolition that the pin was free to rotate inside the connecting brackets, causing heavy wear to the bracket holes.

Figure 1. General arrangement of Humber Bridge and view onto an A-frame rocker bearing

Figure 2. The main span deck box arrangement at the towers before and after the works

Laser extensometers recording longitudinal displacement of the main span deck box had been set up (Brownjohn *et al.*, 2012). These were mounted on the tower legs, measuring the varying distance to the deck box corners. For example, passage of a 40t vehicle results in longitudinal movement over a 20 mm range. Analysis of the resulting data showed good agreement with a computational structural model. Extrapolation from a 4 week monitoring period led to the estimated longitudinal accumulated sliding distance of the main span being 4.5 km/year, or around 130 km prior to the A-frame replacement works (Hornby *et al.*, 2012).

Extensometer output demonstrated that the deck box was articulating as per the original design intent. However, due to the relatively large height of the A-frames – at 3.8 m tall – and

Figure 3. Pin housing at the base of an A-frame (left) and gap closure formed by elongation of the pin's hole in supporting bracket plates (right, circled in red)

relatively small horizontal displacements, it was not clear to the naked eye that the frames were rocking to accommodate longitudinal displacements. If a pinned bearing were seized, it is possible that it would form a moment restraint against a free rocking motion with displacement accommodated by the A-frame acting in flexure. Such a flexural action on the A-frame would impose a stress range of approximately 90 N/mm^2 on extreme fibres of the frame for each 40t vehicle crossing the bridge: a stress range which could result in rapid growth of fatigue cracks.

To determine whether the A-frames were freely rocking, a simple but effective monitoring regime was instigated. Moiré tell-tales, sensitive to differential movements as little as 0.1 mm, were attached across the A-frame pin-housing-lug-plate interface. These were visually monitored under displacements imposed by live load. All A-frames appeared to be rocking freely under imposition of live load with the exception of Barton main span's A-frames.

From the wear to A-frames at Hessle main span and concerns over the freedom of rotation of Barton main span's A-frame bearings, it was decided that refurbishment or replacement of the main span A-frames only was required.

3. Option development and design

Six possible options were developed and appraised by the designer and the bridge owner (Hornby *et al.*, 2012). Aspirational design life was 120 years, with 50 years on replaceable parts such as bearings.

Refurbishment or replacement like-for-like was ruled out since the existing arrangement only achieved 30 years' life. Consideration was given to removing all uplift forces, allowing use of a common spherical sliding bridge bearing. This could be achieved by filling the end deck box with concrete. The large quantity of concrete required made this option untenable. The existing pin-in-bush bearing was identified as a particular weak point not only due to their poor wear performance but also the difficulty of replacement now and in the future.

Proprietary plane spherical bearings with polytetrafluoroethylene (PTFE) sliding surfaces are commonly encountered within the realms of mechanical engineering and were specified. It was decided that rather than installing these bearings into new A-frames, the horizontal and vertical load components would be separated into vertical pendel and horizontal wind shoe elements (Figure 2) to simplify installation and future bearing replacement works.

Pendels (from the German *Pendel* meaning pendulum) are rocking single-bar linkages, here positioned in place of the A-frames and housing a spherical bearing top and bottom. The wind shoe is a steel box cantilevering from the deck box, with opposed sliding bearings providing horizontal reactions. An inventory of the principal elements is included in Table 1.

Eurocodes were used for determination of load effects and design of new structural components. Traffic live load to the UK implementation of BS EN 1991-2 (BSI, 2008) was found to result in load effects and longitudinal displacements far in excess of the bridge's original design load or its bridge specific assessment live loading. Bespoke amendments to the appropriate factors were

Element	Item	Number	Material	Total / comments
Steel (BS EN 10025 (BSI, 2009a), BS EN 10225 (BSI, 2009b))	Temporary pendel	4	S355	8.0 t
	Temporary pendel holding-down frame	4	S355	26.5 t
	90 mm thick temporary pendel connection plate	4	S355	16.8 t
	15 mm thick end diaphragm strengthening plate	4	S355	2.4 t
	Upper pendel connection bracket	4	S355	16.0 t
	Pendel	4	S355 and S460G2+M	24.8 t
	Pendel holding-down frame	4	S355	36.0 t
	Wind shoe	2	S355	9.8 t
			Total	140.3 t
Threaded connectors	BS EN 14399-8 (BSI, 2007) fit bolts	>4000	Grade 10.9	
	Post-tensioned steel bars	144	Macalloy 1030	12.7 t
Bearings	Spherical: pendel	8	PTFE, stainless steel	RBC Lubron supplied
	450 mm dia. pins, pendel	8	Mild steel	7.6 t
	Spherical sliding: wind shoe	4	UHMWPE, stainless steel	Ekspan supplied
In situ reinforced concrete (concrete BS EN 206 (BSI, 2013), reinforcement B500B/C to BS 4449 (BSI, 2005))	Deck box strengthening: wind shoe	2	C40/50	11.6 m ³
	Deck box strengthening: upper pendel connection	4	C50/60	24.8 m ³
	Pendel bottom anchorage	4	C40/50	17.0 m ³
	Wind shoe plinths	4	C40/50	3.4 m ³
	Side span A-frame plinths	4	C40/50	2.6 m ³
			Total	59.4 m³
Electrical	3.3 kV cast resin transformers	4	Air cooled	Installed by PCS

Table 1. Summary of principal elements

To allow removal of the A-frames and installation of the new pendel without closing the bridge to traffic, a temporary pendel arrangement was designed and implemented

made to develop a live load model representative of the bridge's capacity (Collins *et al.*, 2013).

While predominantly under compressive loads (16.4 MN at ultimate limit state (ULS)), the pendels frequently enter tension (11.5 MN ULS) when forming a torsional reaction. Loads are transferred from the pendels into the tower portal beam by a steel fabricated holding-down frame. To resist uplift, the frame is connected by post-tensioned steel bars to the portal beam. This necessitated formation of a reinforced concrete anchorage inside a void in the portal beam which houses the bridge's electrical transformers and switchgear. The frame has been designed to take loads from side spans, should the existing A-frames here require future replacement.

The top pin of the main span A-frame was just 225 mm in diameter. To reduce bearing pressure and increase wear life a larger 450 mm pin was designed. However, this larger pin is contained within the pendel in a geometrically tight area that was only big enough for the A-frame's top bearing. High-strength quenched and tempered steel BS EN 10025-6 S690QL1 (BSI, 2009a) with enhanced brittle fracture requirements was specified. Specialist guidance was sought to produce an achievable performance specification for the pendel's spherical bearings. This included close consultation with suppliers at design stage and review of load and wear tests.

The top pendel bearing and its connecting plate is attached to the deck box by post-tensioned bars. The bars are anchored against the penultimate diaphragm. An existing complex arrangement of longitudinal diaphragms in this area is strengthened compositely by reinforced concrete.

To allow removal of the A-frames and installation of the new pendel without closing the bridge to traffic, a temporary pendel arrangement was designed and implemented. It was attached to a new 90 mm thick plate cantilevering from the side of the deck box, immediately adjacent to the tower leg. This is an area of the deck box not designed to take large point loads. Steel strengthening plates 15 mm thick were therefore bolted to the end diaphragm, providing additional capacity to transfer loads from the temporary pendel to the A-frame–pendel connection area.

To limit the extent of strengthening works required to transfer temporary pendel loads, it was specified that the bridge's nearside lanes (lane 1, the 'slow' lane) in both directions would be closed to traffic while on temporary pendels. The main effect of this is to reduce the torsion on the deck box requiring reaction by the temporary pendels. When combined with reduced factors for the relatively short duration that the temporary pendels would be resisting live load, ULS loads on the temporary pendels were 6.4 MN compression, 2.5 MN tension.

Figure 4. Wind shoe hydraulically preloaded sliding spherical bearing arrangement

Opposed sliding spherical bearings are required to the wind shoe such that wind-induced loads (2.1 MN ULS including a dynamic wind loading) in either horizontal direction can be resisted and transferred into the portal beam through new reinforced concrete plinths. Unless preloaded, the windward side bearing would carry no load, opening up sliding surfaces to dirt and moisture ingress: highly undesirable if bearing life is to be maximised. Hydraulic preloading of bearings to maintain a contact pressure of at least 2 MPa was specified.

While several bearing manufacturers offer bespoke units incorporating both hydraulic cylinder and bearing, it was felt that future maintenance and replacement would be made easier if the cylinder and bearing could be separate, easily procurable items (Figure 4). Conventional PTFE in sliding surfaces has limited wear life for high movement cumulative displacements and velocities. A silicone-lubricant-impregnated ultra-high molecular weight polyethylene (UHMWPE) was specified. The deck box soffit to which the wind shoe was connected was strengthened by placement of in situ reinforced concrete.

4. Construction

Construction works commenced in July 2013 and finished in July 2015. The £4 million project was procured under the NEC3 Engineering and Construction Contract option A (priced contract with activity schedule). The designer was retained as NEC project manager and supervisor.

For accurate fabrication and installation, it was essential first to survey in three dimensions the faces and edges of the existing structure. What might otherwise be a simple structural survey was complicated significantly by the exposed windy environment and constant movement of the deck box under traffic loads. Surveying was hence at night in calm winds and low traffic volumes.

Figure 5. Removal of lane 1 carriageway expansion joint and installation of a temporary hinged plate allowed the new fabrications, such as the permanent pendel base, to be lifted by Hiab loader crane and lowered onto the portal beam

Bespoke brackets were made so that the survey station could be fixed directly to the deck box, allowing it to move with the bridge. Two surveys were then created: one of the bridge deck box (moving) and one of the portal beam (stationary). These two surveys were then brought together by plotting the A-frame's upper bearing locus.

The main area for the works, the A-frame bearing shelf atop the portal beam, is extremely cramped. Working area was increased by a temporary scaffold cantilevering from the face of the portal beam. Access to the deck box faces was provided by scaffold affixed to the steel box, positioned such that it did not clash with other bridge elements as the bridge moved longitudinally.

High-risk operations, such as lifting of the bulky replacement components, were meticulously planned. To reduce risks associated with lifting and working in confined areas, the contractor removed the lane 1 carriageway expansion joints above the portal beam. Access for lifting in the large sections of new steelwork was created by temporary expansion plates (Figure 5). These plates could be independently hinged open to allow the steelwork to be lowered in through the carriageway.

Completion of the dimensional survey allowed steelwork fabrication to start. The pendels' spherical bearings were chill-fitted into the completed pendel arms after fabrication. Aware of the difficulties of welding some of the high-strength steel grades specified, the designer's metallurgical expert, fabricator Allerton Steel and the contractor's material consultant Sandberg worked closely to ensure the high quality of welding and materials was not compromised. Weld trials to the originally specified S690QL1 were not successful and a new specification for BS EN 10225 S460G2+M with enhanced toughness and yield strength requirements was developed.

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The contractor designed and installed temporary pendels, with rotational freedom provided at their ends by pins in bearing bushes of synthetic woven material in a resin matrix incorporating solid lubricants. During the 15 months on temporary pendels, these bearings wore to such an extent that they emitted loud banging noises under deck movements. However, thanks to close monitoring of strain gauges attached to the temporary pendels it could be proven that resulting stresses in steelwork from additional restraint formed by the bearings were of a safe magnitude.

Before load transfer to the temporary pendels, the wind shoe and bearing assembly was installed and commissioned. Output from a pressure transducer allows the bridge owner to monitor the hydraulic system remotely. The steel wind shoe was installed by lowering it through the temporary expansion plates before transferring the load across the portal beam and under the deck by the use of a series of lifting points with lever hoists.

The wind shoe and strengthening plates were connected to the deck box with M24 fit bolts. The selection of these fit bolts reduces the number of bolts required compared to conventional preloaded bolts in normal clearance holes. To ensure the fit bolt shank engages the connected plies in shear, the hole tolerance is just $-0/+130\text{ }\mu\text{m}$. Holes were drilled and reamed on site with magnetic drills, using a match-bored template with undersize holes. Quality control of hole size was by go/no-go gauges on every single hole. All bolts were fitted with direct tension indicator washers to verify preload applied with torqueing by hydraulic wrenches. These performed very well, offering increased control and reduced noise compared to impact wrenches.

Load transfer to the temporary pendels was carried out under live traffic, albeit at night in calm wind conditions to make for easier monitoring of strain gauges. Taking advantage of the slope in the A-frame bearings created by 30 years of wear, the bridge was lifted by jacking up each temporary pendel in turn by two 260t low-height, failsafe hydraulic cylinder jacks connected in series to a hand pump.

The loads in the temporary pendel were monitored by way of strain gauges. Dial indicators monitored the height of lift of the A-frame's bearing housing. When it was determined by reference to the monitoring equipment that there was no residual load in the A-frames, they were severed from the deck box by oxyacetylene cutting (Figure 6). This was repeated at all four corners of the deck box over successive nights in September 2014.

With load transfer to the temporary pendels complete, the A-frames were demolished (Figure 7). A survey finding was that the deck box is significantly non-flat and therefore unsuitable as a surface for the clamping reaction formed by the post-tensioned bars. To overcome this, an ultra-low shrinkage and creep, high-

strength resin, usually used for chocking of marine machinery base plates, was used to fill the voids after engineering a minimum gap between the plates. Such a specialist, high-performance resin required exacting installation conditions, proven by detailed trials.

In situ concreting required complex reinforcement fixing and a highly workable, self-compacting, low-shrinkage concrete. Extensive trials were undertaken to prove the pouring method into the tight, enclosed spaces of the deck box and switch room.

Figure 6. Oxyacetylene flame cutting at night following load transfer from the A-frame (foreground) to the temporary pendel (background)

The new pendel was secured to the portal beam by the holding-down frame, each of the four frames held down by 14 post-tensioned steel bars. The 50mm dia. bars passed through 75mm dia. holes, cored 1.3m through reinforced concrete into the portal beam void. To allow installation of a densely reinforced 1m deep concrete anchorage block to the ceiling of the switch room (Figure 8), C Spencer installed new transformers in a temporary location on the bridge footways. This allowed the old transformers to be removed from the working area: an excellent example of the ‘avoid, reduce, control’ hierarchy of risk mitigation.

There was inadequate space in the closed lane 1 for a mobile crane to extend its outriggers fully. To transfer the steelwork down between main span and side span deck boxes through the hinged temporary expansion joint, the contractor used a 175t/m Hiab loader crane, modified with a bespoke ballast box on the rear to carry and install the steel (Figure 5). Off-site trials using test weights proved the machine could safely operate on reduced outriggers.

Pendels were manufactured in two halves with a bolted splice in the middle to aid installation and future bearing replacement. Accurate survey discipline throughout the drilling and installation works ensured that when the upper pendel connection was installed above the pendel base, the two pendel halves aligned well within tolerance (Figure 9).

Post-tensioned steel bars in the base and upper connection were each prestressed with 1400 kN prior to load transfer from the temporary to permanent pendels. Load transfer was again carried out in calm weather on successive nights with the bridge remaining open to traffic throughout. The bolted splice was installed, followed by immediate lowering of the temporary

Figure 7. Gap left by removal of a main span A-frame, restraint being provided by the temporary pendel (behind) with strain gauges monitored. An A-frame left unaffected by the works can be seen on the right

Figure 8. Portal beam void's switchroom, showing new in situ anchorage for the holding-down frame's post-tensioned bars on the ceiling and new transformer in the background

Figure 9. The lift of the upper pendel connection (left) and achieving perfect alignment of top and bottom parts of the pendel (right)

pendels, dropping the bridge onto the permanent pendels. This time the jacking operation had the benefit of strain gauges providing data for both the temporary and permanent pendels to determine when all of the load had been transferred. The strain gauges continue to monitor loads on the pendel, providing useful data to the bridge owner during unusual wind or traffic events.

5. Use of three-dimensional modelling

Design was aided by, and contract drawings created from, a three-dimensional computational model. This model was passed to the contractor and greatly aided site planning including complex lifts (Figure 10), explaining works to the client and operatives in task briefings.

6. Discussion

6.1 Learning points

With increasing awareness of high-strength steels, especially due to its inclusion in Eurocode standards, there is likely to be increased specification of grades such as S690 (Baddoo and

The existing deck box was found to be significantly non-flat, eventually leading to use of a chocking resin. With hindsight, the cost of creating access for a pre-works survey would have been less than the actual costs ultimately incurred

Figure 10. Views from the three-dimensional computational model of lifts (left) and photographs of the lifts (right), showing temporary pendel deck plate (top) and permanent pendel base (bottom)

Brown, 2015). From the unsuccessful specification of this grade in the works, caution would be advised over its use in bridgeworks with their often demanding fracture toughness requirements coupled with the heightened risk of hydrogen embrittlement (Dainelli and Maltrud, 2012).

Difficult access prevented a pre-works survey of the deck box to which new plates were to be bolted. The existing deck box was found to be significantly non-flat: in fact it did not meet the tolerances specified in the 1970s fabrication drawings. This led to protracted discussions and analysis to understand whether bolts would safely draw plates together and use of a chocking resin around post-tensioned bars. With hindsight, the cost of creating access for a pre-works survey would have been less than the actual costs ultimately incurred.

6.2 Project successes

The project was full of unique challenges and their successful solutions. Of particular note are the following.

- The client understood the complexities of their bridge and allowed adequate time to investigate and understand bridge behaviour, develop options and design.
- Inclusion of third party knowledge by early engagement with an excellent independent checker who was present when crucial detailed design decisions were made.
- Continuation of design staff to site roles with rapid response to site queries.
- Meticulous planning of site works, aided and communicated by the same building information model used by the designer.
- Low impact on the public: the bridge remained open throughout the works.
- Experienced and innovative contractor's design team leading to versatile, well-detailed temporary works such as the temporary pendel and temporary expansion joint.

- Highly skilled and disciplined site operatives, able to undertake a wide variety of complex tasks accurately to the required quality.
- Use of a predominantly Yorkshire-based supply chain, demonstrating the wide ranging ability of local resources on a project of high complexity.

7. Conclusion

The finished arrangement of pendel (Figure 11) and wind shoe to the Humber Bridge's main span play a critical part in the continuing safe operation of the bridge.

The works undertaken were the most complex structural works undertaken on the bridge to date. They exhibit the difficulties and novel solutions required when working with existing bridges. Close work between designer and contractor, with meticulous planning and care while implanting on site, resulted in works befitting one of the UK's most spectacular structures.

Figure 11. One of the four completed pendels, attached to a new bracket at the top, holding-down frame post-tensioned to the portal beam at the bottom. A bolted strengthening plate can be seen top left

The complex structural works exhibit the difficulties and novel solutions required when working with existing bridges

The unique challenges addressed by this project have been recognised by industry through award of the Institution of Engineering and Technology's Sir Henry Royce Award for Young Professionals 2012 to the Arup author; the Institution of Civil Engineers' Yorkshire & Humber Smeaton Award 2015; and shortlisting for the British Construction Industry Civil Engineering Project of the Year (up to £10 million) Award 2015.

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References

- Baddoo N and Brown D (2015) High strength steel. *New Steel Construction* **23(8)**: 24–26.
- Brownjohn J, Koo KY, Scullion A and List D (2012) Operational deformations in long span bridges. In *Bridge Maintenance, Safety, Management, Resilience and Sustainability: Proceedings of the Sixth International IABMAS Conference, Stresa, Lake Maggiore, Italy, 8-12 July 2012* (Biondini F and Frangopol DM (eds)). CRC Press, Boca Raton, FL, USA, pp. 20–26.
- BSI (2005) BS 4449:2005: Steel for the reinforcement of concrete. Weldable reinforcing steel. Bar, coil and decoiled product. Specification. BSI, London, UK.
- BSI (2007) BS EN 14399-8:2007: High-strength structural bolting assemblies for preloading. System HV. Hexagon fit bolt and nut assemblies. BSI, London, UK.
- BSI (2008) NA to BS EN 1991-2: 2003. UK National Annex to Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges. BSI, London, UK.
- BSI (2009a) BS EN 10025-6: 2004+A1: 2009. Hot rolled products of structural steels. Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition. BSI, London, UK.
- BSI (2009b) BS EN 10225: 2009. Weldable steels for fixed offshore structures. Technical delivery conditions. BSI, London, UK.
- BSI (2013) BS EN 206:2013: Concrete. Specification, performance, production and conformity. BSI, London, UK.
- Collins J, Hornby SR, Hill P and Cooper J (2013) Humber Bridge A-frame rocker bearings replacement. *IABSE Symposium Report* **99(1)**: 2037–2044.
- Dainelli P and Maltrud F (2012) Management of welding operations with high strength steels. *Soudage et Techniques Connexes* **66(7–8)**: 37–42.
- Hornby SR, Collins J, Hill P and Cooper J (2012) Humber Bridge A-frame refurbishment/replacement. In *Bridge Maintenance, Safety, Management, Resilience and Sustainability: Proceedings of the Sixth International IABMAS Conference, Stresa, Lake Maggiore, Italy, 8-12 July 2012* (Biondini F and Frangopol DM (eds)). CRC Press, Boca Raton, FL, USA, pp. 3170–3177.