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Scour, Settlement & Stabilisation of Linton Bridge

Affouillement, Colonisation et Stabilisation du Pont Linton

A. Willoner
Mott MacDonald, Sheffield, UK

ABSTRACT: On Boxing Day 2015 Storm Eva peaked flooding large parts of North Yorkshire and damaging over a hundred bridges. Scour of the river bed at Linton Bridge resulted in sudden settlement of the south pier and severe damage including loss of thrust to the south arch. The bridge, a grade II listed structure which connects the villages of Collingham and Linton, was deemed to be in a dangerous condition and immediately closed.
This paper highlights risks associated with old multi-span masonry river bridges on spread footings which may be perceived as being stable following serviceable life well in excess of 120 years. It describes the key design decisions and complex works carried out to safely assess the damage, provide stabilisation measures and design and construct a new reinforced concrete structure hidden within the bridge's original fabric. Experiences are documented from several challenges including designing a new deck within the limited depth between the crown of arches and carriageway, boring through strong slender piers with restricted weight plant to form stiff piles to an integral deck and grouting long piles in permeable ground beneath an ecologically sensitive river.

RÉSUMÉ: Le lendemain de Noël 2015 la tempête Eva était à son comble. La tempête a inondé de grandes parties du Yorkshire du Nord et a causé des dégâts à plus d'une centaine de ponts. L'affouillement du lit de la rivière à Linton Bridge a entraîné un tassement soudain de la pile sud et de graves dommages, y compris la perte de poussée à l'arche Sud. Le pont, une structure classée de grade II qui relie les villages de Collingham et Linton, a été considéré comme une structure dangereuse et immédiatement fermé. Cet article met en relief les risques associés aux vieux ponts de maçonnerie à travée multiple sur des fondations étalées qui peuvent être perçues comme étant stables après une vie utile de plus de 120 ans. Il décrit les décisions clés de conception et les travaux complexes effectués pour évaluer en toute sécurité les dommages, fournir des mesures de stabilisation et de conception et de construire une nouvelle structure en béton armé caché dans la structureoriginale du pont. Les expériences sont documentées à partir de plusieurs défis, y compris la conception d'un nouveau pont dans la profondeur limitée entre la couronne d'arches et lachaussée, en forantà travers des piliers élancés et solides avec le matériel à poids limité pour former des pieux raides sur un pont intégral et injecter des pieux longs dans un sol perméable sous une rivière écologiquement sensible.

Keywords: Scour; heritage; buildability; mini piling; grouting

1 INTRODUCTION

In the UK highway networks, many multi-span unreinforced arch bridges were built during the eighteenth and nineteenth century. These structures are often characterised by piers with shallow foundations placed in a riverbed, which can be subject to hydrodynamic turbulences in cases of river floods and further localised scour phenomena. Scour-induced settlements can occur progressively over time or suddenly following extreme storm events many years after construction.

In December 2015 Storms Desmond and Eva flooded approximately 10,000 properties and damaged 400 bridges in the north of England. Linton Bridge suffered severe "Serviceability Limit State" failure and was closed with a 5m exclusion zone placed around the bridge with man-access above and below prohibited until stabilisation measures had been carried out.

The repairs to Linton Bridge presented many additional challenges relative to Pooley, Elland, Copley and Tadcaster arch bridges where the December 2015 floods resulted in "Ultimate Limit State" failures.

The emergency assessment, stabilisation and permanent works design were awarded to a Bam / Mott MacDonald JV via a Cost-Plus Compensation Event to the Leeds Flood Alleviation Project. The permanent strengthening works were procured via a competitive tendering process with the Leeds City Council (LCC) approved framework suppliers and use of an NEC option C target price construction contract. This was awarded to AE Yates, who in turn appointed Bachy Soletanche to construct the new piled foundations, between January and May 2017.

2 BACKGROUND

Linton Bridge is a 3-span, 50m-length, masonry arch bridge which was built sometime between 1851 and 1893 to replace an old ford crossing of the River Wharf between the villages of Linton and Collingham, 3km upstream of Wetherby in North Yorkshire, as shown in Figure 1.

Figure 1. Site location plan

The structure is valuable to the local heritage and environment with wildlife including trout, otter and kingfisher.

LCC had carried out previous inspection and maintenance works to prolong the life of the structure before storm 'Eva' occurred. Recent interventions included:

- 2007: Waterproofing, repointing and, more significantly, saddling the bridge with 300mm thickness of C40 concrete across the deck;
- 2013: Underwater inspection and Level 1 scour assessment to BD 97/12. Debris had accumulated on the south pier and a small scour depression noted on the upstream side of the bridge. The assessment concluded that a Level 2 assessment should be carried out;
- 2014: A Level 2 Scour Assessment for a 1 in 200-year return period provided a scour risk rating of 2, on a scale of 1 to 5 (with 1 designating the structure as an immediate risk). The design manual recommends that where the theoretical scour depth exceeds the foundation depth, but the bridge has no history of scour problems, further investigation should be carried out and considerable further investigation was carried out at Linton Bridge.
- 2015: The 400mm deep scour depression was filled, a bathymetric survey carried out and, more significantly, a ground investigation undertaken comprising 3 rotary cored boreholes through the bridge deck at the locations of the south abutment, south pier and north pier and documented in the Ground Investigation Report, September 2015.

2.1 Ground Conditons

The geological map indicates bedrock to be near to the Lower Plompton Grit unit of the Upper Carboniferous Millstone Grit Group with a shallow dip to the north east.

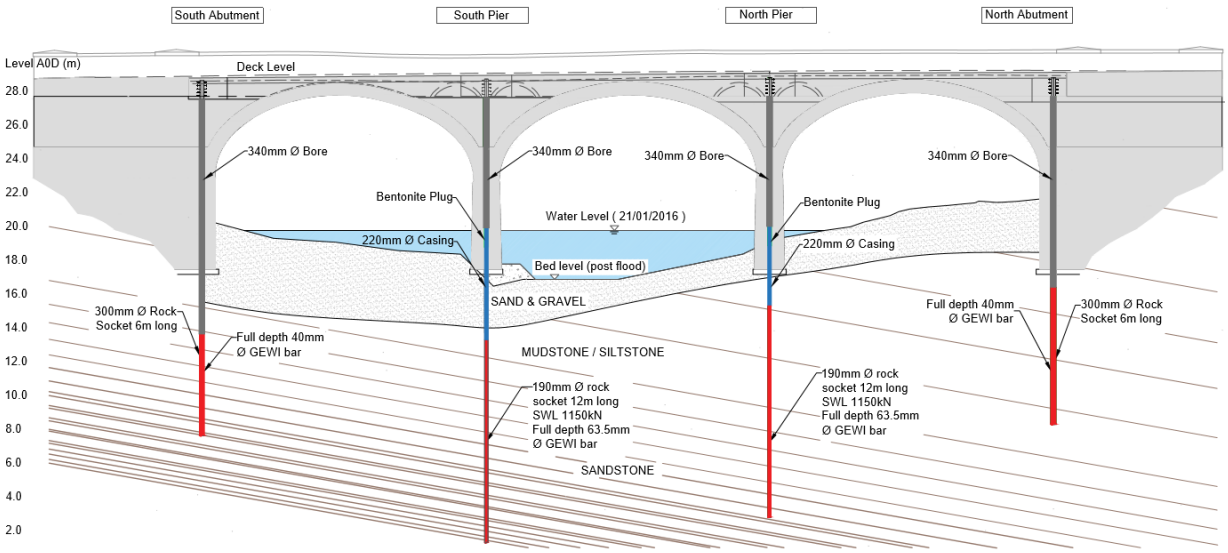

Figure 2. Geotechnical long section

The exploratory holes were bored at 112mm diameter with a Polycrystalline Diamond drill bit and air/mist flush, through the bridge foundations and up to 10m into underlying natural ground. The Ground Investigation confirmed the anticipated solid geology which comprises interbedded mudstone, siltstone and sandstone. RQD's were in the range 0 to 67% with an average 37% and rock strength was described as very weak and weak however point load testing indicated an average UCS of 20MPa.

The most significant geotechnical information regarding scour risk was the discovery that the south pier was founded on 2.75m of granular alluvium and the north pier on or close to rockhead (Figure 2).

3 ASSESSMENT OF DAMAGE

A series of surveys were carried out in January 2016 to assess the stability of the structure.

Remote survey techniques were used to mitigate safety risks for personnel involved and included the following:

- A drone survey to obtain high resolution images of the structure;
- A Point Cloud survey to obtain high resolution topographic information;
- A bathymetric survey of river bed levels;
- A dive survey under the central arch after underpinning the south pier;
- Manual monitoring targets and wireless robust tilt sensors attached to temperature compensated beams on each parapet.

Figure 3 illustrates the deformation of the sunken parapets following settlement of the south pier.

The point cloud survey confirmed the crack through the south arch barrel to be 30mm wide and the crack through the central arch barrel to be 10mm wide. This survey also highlighted a 35mm horizontal displacement of the south pier towards the center of the channel, 29mm further settlement at the upstream end of the pier relative to the downstream and a rotation of the south pier towards the center of the channel of 0.4 to 0.5^0.

Figure 3. Sunken parapet above the south pier

The central section of the south arch, approximately 5m in length, was left supported by tension within the mortar joints and the 2007 concrete saddle. Masonry is very poor in tension and collapse can be sudden and without any warning. If the mortar failed it was expected that this 150t section would drop out and result in a domino failure of the entire bridge into the river as illustrated by the Eastham Bridge collapse in May 2016.

Transverse cracks were clearly visible running through the entire length of each arch barrel adjacent to the south pier and up through the intrados (Figure 4).

Figure 4. Transverse crack through south barrel and intrados

Figure 5 illustrates the longitudinal deformation of the south span using data from point cloud survey and a simplified diagram.

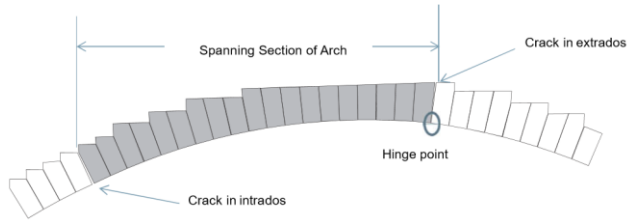

Figure 5. Diagram illustrating flattening of arch

The shaded voussoirs have rotated around a hinge point in the intrados at the south end. The settlement of the south pier to the left of the grey blocks opened up the transverse crack across the arch. The cracks in the intrados and extrados indicates that compression in the arch is lost. The assessment assumed that there would be reflective cracks on the extrados of the arches and this was confirmed when the south and central arches were exposed. Two large transverse cracks on the extrados of the central arch and small cracks on the south arch were observed beneath the carriage. Due to the twisting nature of the deformation, the south arch closed up on the extrados whilst the central arch opened.

Some immediate support was provided to the south span for the following reasons:

- Water running through the crack in the south arch barrel could freeze and expand, widening the cracks further;
- Highly stressed masonry suffers from progressive cracking over time and delays in providing stabilisation would increase the risk of bricks ‘cracking out’;
- The likelihood of another storm causing further scour increases as time passes before some form of emergency stabilsation is provided.

Discussion with the Environment Agency enabled swift approval for a temporary stone causeway, underpinning the south pier with 18m^3 of underwater concrete and placement of rock armor (Figure 6).

Figure 6. Emergancy underpinning of south pier

Following the underpinning, it was considered safe to carry out a dive survey beneath the central arch where the crack through the barrel arch was narrow relative to the southern arch. The diver's records revealed a worrying picture of jostled stones and voids beneath the south pier. The sonar bed survey undertaken in January 2016 shows river bed scour between 200 and 1800mm relative to levels recorded in September 2015.

Following sudden settlement on the night of the storm, subsequent manual monitoring indicated that rapid settlement ceased within a week. Further progressive movement of 4mm was detected over a four-month period until the under-bridge arch support was provided.

The primary cause of damage is clearly scour of the river bed and the distribution of concentrated scour was in the deepest part of the channel where flows were fastest and where vunerable granular foundation soils were located.

4 DESIGN

Temporary works options considered included:

1. Demolition and rebuild estimate: £12.5M;
2. Tieback system to demolish and rebuild south span; estimate £7M and carried risks for temporary and permanent works;
3. Underbridge (Figure 7) and Overbridge supports, estimate £5.3 to 5.4M respectively were preferred and developed further.

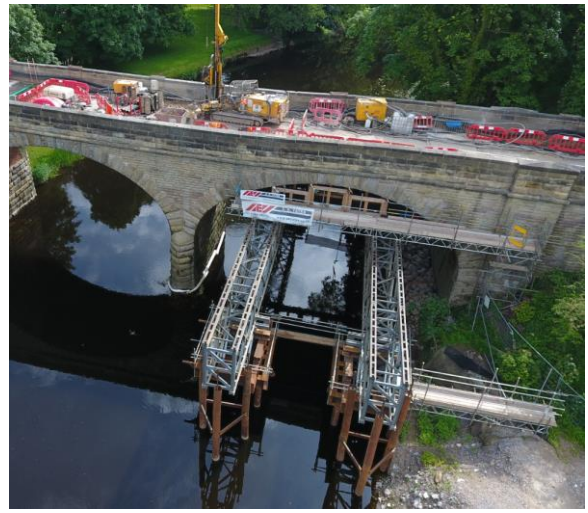

Figure 7. Temporary underbridge support

The temporary arch support needed to enable the permanent works to be constructed had to be considered concurrently with the permanent design to ensure that it did not conflict with this or increase the overall project cost.

Overbridge support would mitigate the risk of the temporary works causing an obstruction in the river channel increasing the flood risk upstream of the bridge during the works. This would reduce the piling risks by enabling work to be carried out from the river bed, however the high predicted deflections in overbridge longitudinal steel created a problem and the programme would be extended due to the temporary steel trusses restricting access to the works in the deck. Following these considerations, the underbridge support option was progressed and an animation of the proposed stabilisation works was created to help engage stakeholders.

The temporary foundation solution, designed by Bam Nuttall Technical Services, comprised tubular steel piles driven at a safe distance from the bridge with compressed air flush. Pontoons were used to float temporary steel beams under the bridge before lifting them and the falsework onto the piles. Jacking support had to be in contact with the arch but it was important that the arch was not inadvertently lifted by over-jacking,

therefore grout socks were positioned at the top of the arch support which was then jacked to within 50mm of the arch and the grout socks slowly pumped to sufficient pressure to evenly support the undulating intrados without lifting the arch.

Once the arch support system was installed and works could safely be carried out on and beneath the bridge, the cracks in the arches were stitched and "Tube àManchette" grouting was carried out through the south pier at four locations.

4.1 Key Decisions

There were 4 key decisions to make regarding the conceptual design of new bridge foundations before proceeding with construction of the temporary underbridge support:

1. The decision regarding piling from deck or river bed level had a strong influence on the temporary works and permanent mini piling. Once it was concluded that an underbridge support would be progressed, careful consideration was given to the form of the bridge deck and method and diameter of piles to be constructed through the slender piers.
2. A simply supported deck was not feasible due to the high sagging moment at mid span and limited available depth between the crown of the arch and the carriageway level, and there were numerous reasons why the highway level could not be raised. The chosen form of the deck was 'integral' in order to minimise the mid-span sagging moment during high temperatures by generating hogging over the outer spans. It was possible to achieve the required deck stiffness once the gas and water mains were raised by splitting the pipes into twin smaller diameters.
3. The conundrum created by designing an integral deck was finding a pile diameter large enough to prevent excessive mid-span hogging in cold temperatures and a size small enough to pile through strong slender piers using restricted weight plant without damaging the bridge. Pile diameters of 504/450mm and 340/300mm were considered with the smaller size being the minimum possible to achieve the required integral stiffness at the minimum practical pile spacing comprising 13 piles at 550mm centres. The larger 504/450mm diameter option offered benefits in terms of cost and programme as the quantity of piles in each bridge support would reduce from 13 to 9, however there were concerns about drillability with low vibration methods and tollances resulting in piles potentially daylighting through the side of the piers.
4. Several piling methods were considered. The most economical method, due to fast production rates, is down-the-hole-hammer with compressed air flush. As the integral deck required high stiffness piles, the compressed air pressure would be very high and was considered to present a serious risk to the stability of the piers. The Ellemex method with a 410mm diameter bore and an extended ring bit to direct the flush downwards reduces the risk of 'blowing out' masonry bricks from the pier, but the risk was still considered to be unacceptable. A rotary duplex method was considered with a lower pressure water or foam flush and this equipment was brought to site as a contingency plan. The chosen solution comprised a simple traditional auger studded with rock teeth where no flush was required, and vibrations minimised by use of the lowest gear and slow revolutions.

5 PILE CONSTRUCTION

The key events during construction of the piles are provided in chronological order to illustrate the difficulties encoutered during grouting and the solutions attempted until the pier piles were sucessfully constructed:

The 26Nr abutment piles were constructed during January and early February without incident, comprising 340mm dia temporary casing sealing a nominal 1m into weak mudstone at 14m depth and 6m long 300mm dia rock sockets.

During construction of the first pier, pile grout loss to the river occurred when the temporary

casing was lifted above rockhead and the casing was immediately sealed back into the rock to minimise the pollution incident, producing a localised over-stiff pile. Several further pile bores were put down through each pier and falling head tests carried out by filling the bore with water to deck level before lifting the temporary casing. These tests indicated that the problem was widespread and changes were required to prevent further grout loss to the river.

A grout plug was installed from pile toe to 1m above old pad footing with the temporary casing withdrawn after a 3 hour 'wait' while grout was observed to 'gell'. Temporary casing was successfully withdrawn, and the grout plug remained stable overnight. Unfortunatley re-boring the pile resulted in loss of the fragile seal and a subsequent falling head test indicated a sudden loss of water as the temporary casing was withdrawn.

The CHS reinforcement in the upper 10m of the pier piles was changed to a 220mm dia permanent segmental casing to provide a long-term seal into the rock and the pile sockets were lengthend to compensate for the reduction in casing diameter. The 340mm bore no longer penetrated the base of the old spread footings and a precautionary bentonite pellet seal was formed around the 220mm casing. Further falling head tests were carried out in advance of grouting and this new method of work proved to be initially sucessful. However, at some locations highly fractured rock was encountered and falling head tests indicated that a seal into rock could not be achieved even when the 220mm dia casing was progressed to 28m depth, some 10m below rockhead.

Further changes were required, including use of a higher cement content in the grout and, more significantly, introduction of a 2-stage grouting process with a cold joint in order to keep the 1st-stage hydrostatic head within the distal section of the pile no higher than river level to allow the fluids within the pile to stay in equilibrium preventing generation of a driving head pushing the grout through the fractured rock into the river.

6 PROJECT ACHIEVEMENTS

The limits of mini pile design and construction were pushed to achieve the desired integral deck solution. Piles of 340mm dia reducing to 190mm rock sockets were bored by a 13t rig to 28m depth and reinforced with full depth 63.5mm dia GEWI steel, to resist a Safe Working Load of 1150kN. CHS or steel casing was used to provided pile stiffness required for the integral deck, equivalent to a pile moment capacity of 250kNm.

During construction the full power of the Klemm 709 rigs were required. As the pile depth increased, the torque of the auger reduced and penetration rates within the lower sandstone section of the piers approached the point of refusal. Maximum vibration levels recorded did not exceed 5 mm/s at 1m from source and there was no evidence of any damage caused to the bridge during pile instillation. The restricted weight plant, could not have constructed piles through the strong masonry at a larger diameter than 340mm without using an alternative method where flush pressure and higher vibrations would increase the risk to the integrity of the slender piers.

Unforeseen highly fractured rock was encountered beneath the river which resulted in delays to the piling works and changes in the grouting methodology, however the overall programme was maintained by resequencing pile cap and deck construction at the abutments before the pier piles were constructed and mobilising a second piling rig.

Strengthening of the bridge included the realignment of the sunken spandrel wall and parapets to achieve a new bridge with 120-year design life within the old listed masonry structure without compromising the visual appearance.

The end result was a difficult project completed to budget, planned timescale and quality, without impacting on the heritage and without any significant health & safety incidents.

7 CONCLUSIONS

This paper is intended to highlight the vulnerability of old masonry river bridges with spread foundations on granular soils, when subject to extreme storm events. There appears to be a trend of more frequent extreme storm events, potentially related to global warming, therefore further investment on assessment and improvements to old masonry arch bridge foundations may be required to provide future resilience.

It was desirable to put additional cored boreholes through the piers, but not only is this an expensive activity, it was also not possible to do this until the temporary support was in place by which time decisions on the form of the permanent works had already been made. Therefore the risk of ground variability beyond specific borehole locations was realised during the contract. Ideally piles through bridge piers would not be integral to enable use of very small diameters carrying predominantly axial compression. Ideally piling would be carried out from river bed level to reduce hydrostatic grout pressure and reduce the risk of damaging the structure.

Due to the access restriction at Linton Bridge this project pushed the limits of mini pile design and construction and indicates what is likely to be the limiting size and safest method of constructing stiff piles through slender bridge piers.

8 ACKNOWLEDGEMENTS

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