Great Musgrave EDE/25

Report for The HRE Group

Revision A, 9 May 2022

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1 Summary

1.1 Remit

Bill Harvey Associates Limited (BHA Ltd.) were asked by The HRE Group to review past inspection and assessment records, planning application, and other documents related to the in-filling of Great Musgrave bridge, and to comment on the suitability of in-filling as a treatment of masonry bridges.

BHA Ltd. will not assess a bridge without conducting our own inspection. The circumstances at Great Musgrave are unusual, in that the structure is not available to inspect as a result of "in-filling" with crushed rock and foam concrete. Our remit here is *not* to provide an assessment, and this report does not provide one.

Rather the purpose is to critically examine the evidence and reasoning leading to in-filling, to the extent that it is available through publicly available documents.

Our comment here is limited to engineering aspects of the bridge and the in-fill. We have no particular expertise to comment on ecological or amenity issues.

1.2 Context

Great Musgrave Bridge is a single span masonry bridge carrying a minor road over the trackbed of the former Eden Valley Railway (EDE). The structure number is EDE/25.

The Eden Valley Railway has been out of use for many years. EDE/25 is part of the Historical Railways Estate (HRE), now managed by the HRE team within National Highways (formerly Highways England) for the Department for Transport.

National Highways took the decision to "in-fill" the bridge, building up an embankment of crushed stone and foamed concrete around and beneath it until only the parapets were exposed.

The action to in-fill EDE/25 Great Musgrave Bridge caused a significant negative response, with coverage in national and construction industry press and much discussion on social media.

Eden District Council determined that a retrospective planning application would be required. National Highways have now submitted this application, asserting that the in-filling was necessary because an emergency existed or was expected to arise.

1.3 *Available documents*

This review covers the following documents.

- Cumbria County Council assessment report from 1998.
- Completion report from 2012 for works conducted by Raynesway Construction Services Ltd. for BRB (Residuary) Ltd.
- Detail Exam (inspection) report dated 2017. Exam conducted by Carillion.
- Visual Exam report dated 2020. Exam conducted by Balfour Beatty Rail.
- Visual Exam report dated 2021. Exam conducted by Balfour Beatty Rail.
- Redacted correspondence between HRE and Eden District Council.
- Great Musgrave Bridge Infill (EDE/25) Planning, Design and Access Statement. Jacobs for HRE.

BRB (Residuary) Ltd. was a precursor to HRE and responsible for the bridge at the time.

Each of the three recent exam reports is prefixed by a "scoring matrix" sheet added by HRE.

We have not seen any examination or assessment reports dated between 1998 and 2017, nor have we seen mention of any examinations or assessments being undertaken in this time. It seems likely however that at least a visual inspection preceded the 2012 work.

We have seen no assessment report, nor evidence of an assessment having been undertaken, since the 1998 report by Cumbria CC.

We have seen reference to an additional visual inspection report in 2018, but the reference suggests that it contains no additional information over the 2017 detailed examination report.

1.4 *Conclusions*

- There is no evidence in the reports examined to suggest a current or developing risk of collapse.
- There is no evidence for a current or likely emergency.
- All evidence presented suggests that the bridge is, as stated by the 2021 examiner, "in fair condition."
- The only reported damage that is likely to be related to live load is at the joint between extrados and spandrel wall at both sides. Elastic flexing of the central part of the span opened the joint and cyclic movement has caused a slight outward movement of the spandrel walls. This behaviour is normal and unavoidable. A movement of some 10mm at the worst location has accumulated in the life of the bridge. There would be no value in repointing this unless to inhibit vegetation growth in the crack.

- It is repeatedly suggested that the soffit was pointed in 2012, but there is no mention of this in the otherwise comprehensive completion report for the 2012 work, and no evidence of it in the later photographs of the soffit. Pointing in 2012 would leave some mark, even if it fell out.

- Mortar is missing locally, not continuously across the width. In the areas photographed there is ample mortar left to carry thrust.

- Perpend joints would often not contain mortar at construction, and such mortar does not contribute to capacity. Absence of mortar in perpend joints is of no consequence.

- The measurements of degree of drop to soffit stones and mortar loss are severely subject to variable measurement method. One off measurements with no defined or recorded method, and no record of locations at which measurements were taken, do not provide evidence of change.

- The soffit stones are dressed to wedges, and cannot drop indefinitely unless mechanical damage occurs either to the dropping stone or the stones around it. There is no sign in the available photographs of such mechanical damage occurring. Were mechanical damage present, it would certainly have been recorded in the inspections.

- The 1998 Cumbria CC assessment used the Modified MEXE method with an onerous factor for mortar loss that does not reflect the localised nature of this defect. Adjusting this factor alone is sufficient to lift the assessed capacity to allow all AWR vehicles.

AWR refers to the Road Vehicles (Authorised Weight) Regulations 1998.

- Further onerous factors were used, which compound to give the result obtained. The Modified MEXE method also significantly under-estimates capacity of elliptical structures such as this.

- Exploration using Archie-M also suggests that the bridge would pass for all AWR vehicles. This remains true even with significant mortar loss.

- The case for strengthening requires *both* that a problem related to live load exists, and that the proposed strengthening can address that problem. The possibility of adverse side effects must also be considered carefully.

- Live load induced movements in masonry bridges are typically around 1 mm. For in-filling to stop live load generated cumulative damage, it must be capable of limiting these tiny movements. Even if in-fill results in a void of less than 1mm across the crown of the arch at construction, the slightest creep of the soil under the unfamiliar load of the in-fill will widen this gap. It is therefore unclear how in-filling can arrest any cumulative damage from live load that is taking place.

- In-filling creates a closed environment, which is likely to keep the stone and mortar of the soffit permanently wet, accelerating degradation of both.
- In-filling removes the possibility of inspection. The whole structure becomes a collection of Hidden Critical Elements. The longer a bridge remains filled, the less is known about its condition.
- This uncertainty creates both health and safety risk (what is the condition of the structure, and what will happen when the support that might be provided by the fill is removed) and financial risk (what will it cost to put right the defects that have developed while the bridge was hidden and uninspected) should removal of the fill be desired. We suspect these risks are not allowed for in claims that the fill can be removed in future.
- If structures that are clearly of historic and aesthetic value are to be permanently or indefinitely buried or modified, a detailed heritage record should be made, ideally by photogrammetric survey, before in-filling commences. The record should be publicly archived. The cost of this would be negligible in the context of the works.

2 The Documents

2.1 Cumbria CC inspection and assessment, 1998

This document is a scan of a paper report. The original photographs were not of great quality, and the printing and scanning process has further reduced their information content.

Photo 8, "General view of barrel soffit at crown." There is very little to be seen here, except a square of relatively fresh stone. By comparison of staining in the neighbourhood with photo ref CEBP1003/213/011 from the 2012 completion report, this is the defect repaired with a stone patch in 2012.

Photo 9, "Spalling to barrel stones 2nd course above west springing." The photograph quality is limiting, but the pattern here suggests delamination of the stone surface, rather than mechanical spalling.

Photos 10, 11, "Open joint along extrados at quarter points." This could indicate slight movement of the joint. Some mortar remains, and there is no sign of mechanical damage to the stone. The mortar remnants are suggestive of superficial pointing in cement mortar.

Photo 14, "Vertical crack in mortar joints on S.E. wingwall." Evidence of a slight movement. No evidence of a link to live loads, nor is such a link very likely in the wing wall.

Photo 15, "Spalled masonry block." As with photo 9, the break surface here suggests a stone quality issue; this does not look like spalling due to stone being overstressed at the surface.

In addition to the defects recorded in photographs, the inspection report includes the following.

- Loss of mortar in isolated areas 10mm wide, up to 30mm deep.
- Water percolating and leaching various to soffit.
- Isolated drummy areas.
- "Bulging" of the spandrel wall over the ring of 2, 3, 5, and 10mm at the four corners.

Drumminess is most likely a result of internal defects in individual stones. Some such defects have manifested as loss of surface material; it is likely that other stones have similar defects, allowing surface material to vibrate when struck.

There is no possibility of ring separation here as there are no separate rings as would be found in a brick bridge.

Bulging would normally mean exactly that, a bulge with the masonry coming progressively out of line then back in. It would not be possible to detect even a 10mm bulge in rusticated stone. From the context, we think this refers to a step between the ring and the spandrel wall.

Measuring an overhang, even in stone with dressed edges as here, to the nearest millimetre is impossible, but the measurements indicate that the spandrel walls have moved out slightly over the ring. This is almost certainly live load related.

Loss of mortar and water will be discussed below.

2.2 *Works completion report, 2012*

We have not seen any mention of an inspection prior to these works, which took place 14 years after the 1998 inspection discussed above.

The completion report records in some detail the defects addressed, with photographs from before and after. Some of the "before" photographs are extraordinary, being little more than black squares.

There is one striking omission from the record: there is no mention of loss of mortar to the soffit, nor of any attempt to rectify this.

Given the contractual significance of this report, we find it improbable that significant soffit pointing was undertaken in 2012.

2.3 *Detailed exam, 2017*

The remaining reports are from recent years. The quality of photographs available in these recent inspection reports is poor. This remains sadly normal for bridge inspections, despite the ready availability of excellent cameras and the ease with which high resolution photographs can be managed. Some of the photographs were out of focus as taken, but all have been heavily compressed, removing vital detail in the process (see figure 2.1).

Figure 2.1: Excerpt of photo 5 from 2017 exam, showing image quality degradation from compression. The quality issue is striking here because of the sharp edges in the sign. Across the rest of the image the same compression eliminates all fine detail.

Photos 7-10, "Erosion with loss of face to stonework." The term erosion is more appropriate than spalling used in 1998 report.

Photo 11. Shows repair from 2012 to small area of lost stone.

Photo 45, "Separation fracture above the extrados has been pointed." It is impossible to assess from the photograph but there is no mention of the crack having re-opened in the 5 years since this pointing was done.

Photo 72, "N/E wing wall: bulging/oversailing to the stonework." The oversailing in question is very slight. This is just below the string course, so now hidden just below the top of the embankment. Not relevant to capacity assessment.

Photo 117, "East springer course: fracture at N/E return section." Nothing of concern.

There are a great many photographs labelled, "Typical example of the condition." These are wide views, and after the image compression discussed above they give little indication of condition.

There is no evidence in these photographs of significant soffit pointing having been undertaken. If the soffit had been pointed, and the mortar since lost, the evidence would still be clear after 5 years.

Relevant observations regarding the soffit from the text of the report are:

- Slight deflection in stonework sagging up to 4mm in places to the crown region.
- Degraded mortar joints up to 15mm wide x 40mm (Av 25mm) deep at worst in widespread places.
- Pointing repairs have been carried out since the last detailed examination in isolated places.
- Evidence of water ingress.

The reference to "sagging" stonework is slightly misleading. The actual defect recorded is that odd stones have dropped slightly relative to their neighbours.

The measurements for mortar joints are similar to those from 1998. No doubt water passing through the ring continues to remove mortar, but this is a gradual process. Variation in measurement location and approach is likely to be part of the difference observed.

Pointing is noted to have been undertaken in *isolated* places. We can see that one of these is the area around the replacement stone. It is clear from the photos that most of the soffit was not pointed in 2012.

Also noted in the ring face are, "Fissure type fractured stonework up to 1mm at worst in very isolated places." We are unsure what "fissure type" means. The inspector is clearly not greatly concerned about these fractures, which are not granted a photograph.

The recommendataions made are:

- Investigate adequacy of waterproofing and drainage system.
- Remove vegetation including mature trees.
- General repairs to stonework, including eroded loss of face to the structure and parapets.

We return to water below. Vegetation removal should be a matter of routine maintenance; trees should not have the opportunity to grow to maturity close to bridges.

The damage to the parapets is an ongoing issue at this bridge, and one that is not addressed at all by in-filling.

It is not clear that any of the stone defects noted elsewhere in the bridge require urgent attention. Standard methods of repair do not stress replacement stone into place, and are thus purely cosmetic in nature.

The HRE scoring matrix finds:

- No significant risk.
- Minor masonry repairs and pointing desirable, but with no urgency ("> 3 years").

2.4 *Visual exam, 2020*

This mentions a visual examination dated 31/07/18 that we have not seen. It is explicitly noted that, "... all accessible long-standing defects show no evidence of change since the previous detailed examination dated 29/08/17 and the visual examination dated 31/07/18."

Changes highlighted in red in the report are:

- Maximum drop to soffit stones measured at 15mm, compared with 4mm recorded in 2017.
- Soffit joint mortar loss measured "up to" 170mm, compared with a maximum of 40mm recorded in 2017.

These are the only changes noted that might have relevance to assessment.

Photos 11-13, "Example of long-standing downward alignment defects." Photo 11 is of very low quality. Photo 12 is badly affected by compression, however is close enough to be of some use (photograph reproduced in figure 2.2).

Photo 14, "Example of open joints located within the soffit." Photograph reproduced in figure 2.4. It is very clear that this area of open joints has not been re-pointed in recent decades. The photo also illustrates clearly that the joints are empty locally, not continuously across significant widths.

Photo 15, "The mortar loss was noted up to 170mm where accessible." This shows measurement of mortar loss in a perpend joint, which is of no relevance to capacity.

The remaining photographs relate to parapet damage.

There is no evidence of this stone having moved since 2017; such recent movement would show in the newly exposed face.

The HRE scoring matrix sheet is identical to that from 2017:

- No significant risk.
- Minor masonry repairs and pointing desirable, still with no urgency ("> 3 years").

Figure 2.2: Photo 12 from 2020 visual, showing dropped stone. Quality is limited but exposed edge does not look at all different. We would expect a cleaner face in a recently dropped stone, and abrasion or damage in a stone dropping due to mechanical action.

Figure 2.3: Photo 13 from 2020 visual, showing dropped stone with ruler. Ruler sits within chamfer at corners of stones. Where and how was the drop measured on each occasion?

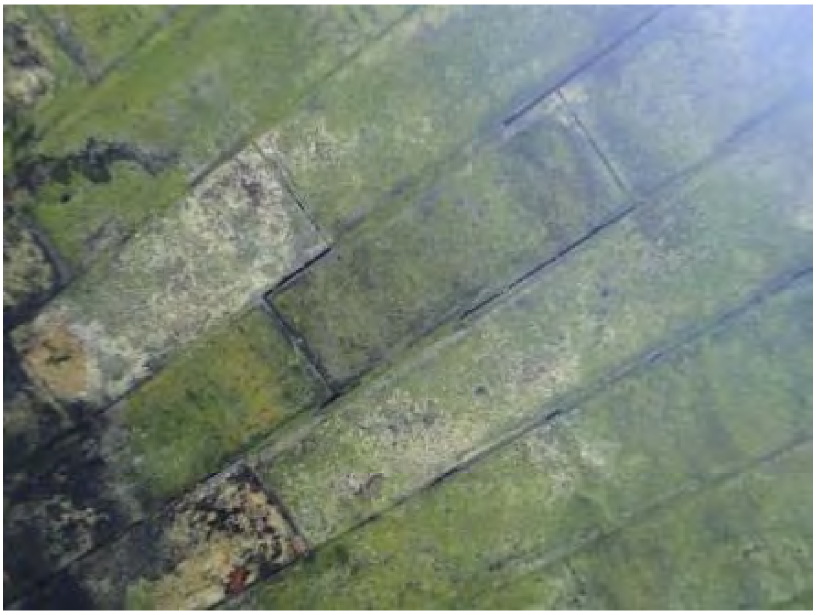


Figure 2.4: Example of open joints, presented as photo 14 in 2020 visual inspection report. The voids are over limited lengths of joint; there remains ample area to carry the thrust.

2.5 *Visual exam, 2021*

This report explicitly states, “New Defect - None. Changes to Existing Defects since Last Examination – None.”

The examiner’s general comments are:

- The bridge appears to be in fair condition.
- General masonry repairs required to the arch soffit.
- Bridge bash damage requires attention.
- Approach walls require repairing.

The third and fourth of these are not relevant to capacity, and are not addressed by the in-filling.

There is nothing in the reports we have seen to suggest that the structure is in less than fair condition.

The only noteworthy aspect of this report is the addition of a recommendation:

> General masonry repairs to the arch soffit including repointing DOJ’s and stitch & grout displaced blocks as necessary £50k.

It is odd that this recommendation is introduced, despite there being *no change in the structure condition since the last inspection, conducted by the same organisation*, without any further comment.

As general masonry repairs other than stitching and grouting of blocks were undertaken in 2012 and cost around £10,645 all in, and the pointing recommended in the 2017 report was estimated at £5K, we suppose that some £40K to £45K of this estimate relates to stitching and grouting displaced blocks.

There is no convincing evidence in the reports that these blocks have actually moved in recent years. Even if they have, they are wedge shaped, and cannot move indefinitely. Stitching, insertion of metal bars, would have no value here. Indeed drilling holes and introducing corrosion susceptible metal would be actively harmful.

The HRE scoring matrix sheet is now strikingly different from that attached to the 2017 and 2020 exam reports:

- High risk to public with medium likelihood, based on defects identified in 2020.
- Repair to "open joints and downward displacement of arch since 2017" *essential in less than 1 year*.

This scoring matrix was completed *after* the in-filling was completed, the in-filling is recorded as "dealing with previously identified movement of arch."

It is difficult to comprehend how the condition of a bridge, which has not changed at all, can be so dramatically re-interpreted.

2.6 Email communication between HRE and Eden DC

An email exchange between Eden District Council and Highways England (HRE) culminates in a record of Highways England's position regarding the justification for in-filling.

HRE assert that the in-filling work was to prevent an emergency developing that might have caused loss of human life, human illness or injury, damage to property, disruption of facilities for transport.

The key part of the final email in the exchange relates to the rationale for asserting this likely emergency.

The email does not offer new evidence, but provides an interpretation of that contained in the documents discussed above.

The same statements are repeated multiple times through the letter. We have attempted to identify the distinct statements to address each once.

> Cumbria CC assessed the bridge to have 17 tonne capacity, with pointing required to bring it up to 44 tonnes.

This is elsewhere referred to as a "structural analysis". CCC undertook an assessment calculation using the Modified MEXE method. This is not a structural analysis. See chapter 5 for discussion of assessment.

The capacity limit hinges on the condition factors chosen. The factor chosen for mortar loss does not reflect the localised nature of that loss, and changing this factor alone would result in a pass for all AWR vehicles.

> The structure was damaged by vehicles in excess of 17 tonnes.

We see no evidence in the available inspection reports of significant damage to the bridge by vehicles, except to the parapets.

> In 2012 HRE's predecessors (BRB Residuary) repointed the arch.

This pointing is referred to repeatedly. There is no record of any such pointing in the completion report of the 2012 works. The 2017 inspection notes re-pointing in "isolated" places. The photographs show no evidence of pointing except in the vicinity of a stone patch.

> In 2017, the joints in the soffit had "again" opened (up to 40mm with an average of 25mm).

Since we have seen no evidence of pointing to the soffit in 2012, we think these joints were little changed since 1998.

> [In 2017] the crown of the arch had dropped; at that time it was recorded as a drop of 4mm.

There is no drop to the crown of the arch reported. The observation is of individual stones out of alignment with their neighbours.

There is no possibility of measuring such a drop to the nearest millimetre. The stones have rough surfaces, rounded corners, and chamfers.

It is unlikely that a 4mm drop would be detected by visual inspection. This is likely to be an under-estimate of the displacement at this time.

> [In 2020] the downward movement of the arch had increased to 15mm.

Again, there is no record of downward movement of the arch as a whole. Misalignment of individual stones is measured as 15mm in 2020. There is no record of which stones where measured on each occasion, nor of how the measurements were taken. Variation in measurement between inspectors could easily result in a 10mm displacement being measured as 4mm on one occasion and 15mm on another, with no movement taking place.

If there is serious concern about ongoing movement of stones, simple methods would allow an objective measurement to be made over a few years.

> [In 2020] the joints between masonry [had] opened up to 170mm.
> . . . Mortar loss increased from 10% to 38% in a short period.

The phrase "opened up" suggests widening, which is not the case. The phrasing also suggests widespread joints empty to 170mm. In fact 170mm is the maximum depth found. The report does not say how many locations were found to have missing mortar to anything like 170mm depth. As previously, the missing mortar is localised, not continuous across transverse joints. The photograph provided shows that 170mm was measured in a perpend joint, which may never have been filled, and has no influence on capacity.

It is more likely that the 170mm deep void was not measured on previous inspections, or that the ruler stuck on a roughness in the joint.

The suggestion that 10% or 38% of mortar is missing is highly misleading.

> This combination of defects indicates a structure that is suffering from being continually overloaded. Without intervention those defects would continue to develop . . .

There is evidence of relative movement between arch and spandrel near the crown. This is normal. As a result of this movement, the spandrel walls have worked outwards very slightly and now oversail the ring by a few millimetres.

There is no conclusive evidence in the inspection reports of defects developing as suggested.

> . . . disruption to the network through the closure of the road over the bridge would be the "best case" scenario.

This is absurd.

> The last visual examination on 22 January 2021 confirmed the extent of the distress to the arch though no measurements were recorded on that occasion.

The 2021 inspection report, like those in 2017 and 2020, is factual in nature, aside from the recommendations and summary comments. It explicitly records *no change* to the structure since 2020. These reports give no indication that the inspectors were at all concerned about the structure. There is no hint of "distress" to the arch. Even the 2021 report describes the condition as "fair".

The only change in 2021 is the addition of a recommendation to pin and grout the dropped stones. This is a change of interpretation without any change of evidence. There is no explanation of why the inspector feels pinning and grouting is needed, or why it would be helpful.

> [The 2021 visual exam] reaffirmed that the mitigation works were required as a priority to "prevent" a collapse and thereby an emergency as defined within Class Q.

This is an extraordinary statement. At no point previously has the prospect of collapse been raised. The claim is not supported by any of the evidence available. The claim was not made by the examiner, who regarded the structure as in "fair" condition.

> [The 2021 report] indicated a significant cost [£50K estimate] for remedial works; that cost estimate makes no allowance for access, scaffolding, road closures etc.

We note that the actual contract value for the stone repairs and pointing undertaken in 2012 was £11K. Was this excluding access arrangements?

The cost estimate for repointing "very deep open joints to the soffit" in the 2020 report was £5K. There is no change to the repointing part of this work between 2020 and 2021, and no mention of any additional defects to the soffit, which suggests that stitching and grouting the dropped blocks is £45K of work.

Stitching and grouting is a damaging process. If the stones are dropping into the wedge, stitching will prevent that movement without providing an alternative load path.

> Infilling the arch to form an embankment is, in these circumstances, the most reliable form of mitigating the risk to road users, our employees and our contractors who would have to continue examining the bridge. It stabilises the structure in the long term and avoids the disruption of closing the road to carry out repairs to the structure beneath. Additionally infilling represents a better use of public funds compared with frequently having to repair the arch when the root cause, the traffic loading, remains unchecked.

We discuss the merits of in-filling in chapter 7.

2.7 *Great Musgrave Bridge Infill – Planning, Design and Access Statement*

This report was prepared by Jacobs for HRE in support of a retrospective planning application.

Key quotes from the document are, from section 1.3:

> In 2012, the arch barrel was repointed to increase its structural capacity, but that repair lasted only 5 years with traffic loading believed to have accelerated the defects beyond the point at which repointing alone would suffice . . . . A subsequent survey in 2017 reported that the joints between the masonry in the arch had again opened up (to 170mm) and that the crown of the arch had dropped (by 15mm). A further survey in February 2020 then reported a further drop of the bridge's arch.

There is a detailed record of the work done in 2012, which does not include any mention of soffit pointing.

The 2017 inspection found empty joints "up to" 170mm. The location pictured is in a perpend joint, where the mortar loss can have no impact on capacity.

It is not realistically possible to measure mortar loss at every point. The fact that a 170mm deep void was found at a particular inspection is no evidence that it was not present at earlier inspections.

The wording suggests that the whole crown of the structure is dropping. This is misleading. The inspection reports in fact identify slight misalignment of individual stones, which probably took place when the bridge was de-centred following construction in the early 1860s.

Masonry bridges are built on timber "centres". The act of removing these is "de-centring".

The 2017 survey measures this misalignment as 4mm. The 2020 report measures it a 15mm. In neither case is the method or location of measurement recorded. The soffit is rough, and the transverse corners of stones are chamfered. Without more detailed records these measurements are not comparable.

No evidence is presented for live load causing significant deterioration. Photographs show none of the damage that would be associated with such deterioration.

The report continues:

> To prevent further deterioration of the bridge from occurring and remove the possible risk of structural collapse, and to enable unre-

> stricted use of the bridge by traffic . . . , it was considered necessary to undertake works to support the bridge.

There is no credible risk of structural collapse, nor credible argument that such a risk might develop sufficiently rapidly to constitute an emergency.

From section 1.5:

> The reasoning and justification for the infilling of the bridge rather than, for example, masonry strengthening and repairs to the bridge (including ongoing maintenance/examination), largely relate to the cost benefit analysis over a 60 year horizon and also other local considerations (that are assessed in Chapter 3 of this statement). In summary, it has been estimated that infilling offers significantly better value for money by being between 50% - 60% less expensive than strengthening and repairs, and as the scheme is not considered to have a significant detrimental impact on local environmental considerations nor conflict with local planning policy.

We see no evidence on these reports that the bridge has a capacity issue. We consider whether in-filling could be effective in section 7.

3 Condition

3.1 Defects

We have not been able to inspect the structure ourselves. If we were to do so, it is possible that we would detect defects in the structure that are not mentioned in the inspection reports. Given what we can see of the structure in photographs, we think it *unlikely* that any defects present but not recorded would materially influence our assessment of the capacity of the structure.

Any defects not recorded in the inspections cannot have contributed to the decisions leading to in-filling, as these decisions were based on the inspections.

The spandrel walls, wing walls, parapets of masonry bridges are qualitatively assessed only. An expectation of future maintenance needs might have been included in estimating future costs, but condition of these elements cannot contribute to the assessment of the structure as unable to carry particular traffic.

The only defects that can influence the assessment result are those to the ring of the arch, as defects elsewhere are explicitly excluded from the assessment. The defects recorded can be summarised as follows.

1. A few small cracks, not considered worthy of photographs by the examiner.
2. Some deterioration of stone, perhaps as a result of being wet for a long time.
3. Some localised loss of material from the face of stones, related to this deterioration.
4. Misaligned stones near the crown.
5. Localised loss of mortar in the soffit joints.
6. Open joint between ring and spandrel wall, repointed in 2012; no re-opening recorded.
7. Over-sailing of ring by spandrel wall by a few millimetres.

Localised loss of material, whether stone or mortar, does not impact capacity if material remains across enough of the width to carry the forces present. The areas of material available are large relative to the forces.

Mortar loss in assessment tools models mortar missing across the full width of the bridge. That is not the situation displayed in the photographs from Great Musgrave.

The misalignment of stones is slight. The misalignment itself has no consequence for capacity. The stones are wedge shaped and cannot drop out without the dropping stone or those around it suffering obvious mechanical damage. No such damage is recorded or visible in the photographs.

The oversailing of the ring by the spandrel walls and the open joints here are surely a result of traffic load. The fact that the oversailing is only a few millimetres, when this has developed over the whole life of the structure, indicates that this is not developing at a rate that should be cause for concern.

3.2 *Water ingress*

A few mentions are made of possible issues with the waterproofing. This is very probable, indeed it is normal in bridges of this age. It would certainly be desirable to renew the waterproofing, as this would allow the stone and mortar to dry and is likely to reduce the rate of deterioration of both.

Unfortunately neither checking the waterproofing nor improving it is very practical.

One way to make things worse is to stop the water that gets into the ring from escaping and drying. Filling to close to the soffit with foam concrete will do that.

3.3 *Progression of defects*

The argument is made that *progression* of defects constitutes grounds to consider an emergency likely.

There is no evidence of joints being opened by mechanical action. Mechanical action would not remove mortar in short patches. Loss would be in clear areas, and the joints would look worked.

Applicaton of mortar in bed joints of masonry bridges was not always perfect. Indeed, it was common for mortar to fall from the joints during construction, being replaced (if deemed necessary) by repointing from below. Such repointing contributes little to the capacity of the structure as it remains unstressed. Mortar was even less assiduously applied in perpend joints. Some of the missing mortar may never have been present.

The measurements offered are far from conclusive evidence of progression. Measuring depths of voids between dressed masonry blocks is a hit and miss affair. The ruler may stop against mortar, or it may stop where the stones get closer together locally. Whether you measure 50mm or 150mm can depend critically on where the measurement is taken. Where there are numerous voids across the soffit, these will not be systematically measured, but sampled; the particular locations sampled are not recorded, and will not be the same at each visit.

In the absence of any evidence of mechanical working of the joints, it is improbable that 100mm or more of mortar was washed out in the space of 3 years. The application of Occam's Razor suggests that the deep void measured in 2020 was missed in 2017. This might be by chance, or it might be because the 2017 inspector understood that perpend joints were not relevant to capacity.

The most likely mechanism for removal of mortar is dissolution by water passing through the ring. This process is proportional to the rate of water flow, and generally slow.

Even if more mortar is lost, as long as the mortar loss remains localised it will remain irrelevant to capacity.

Similar can be said about the dropped stones. There is no possibility of measuring this misalignment to better than ± several millimetres. The difference between 4mm and 15mm is the difference between an under-estimate and an over-estimate of a movement of around 10mm. It is unlikely that a 4mm drop would be detected by eye, given the surface roughness of the stone and the chamfered edges. The drop was almost certainly greater than 4mm in 2017.

Figure 6.7 on page 39 shows a stone dropping due to mechanical action, and the damage at the joint is immediately clear. No such damage is visible around the only stone photographed in any detail at Great Musgrave. Any such damage would surely have been photographed and noted in the reports.

A drop of 10mm would leave a newly exposed face at the edge. There is no mention of evidence of change other than the measurements in the report, and none is visible in the photograph of the one stone shown (see figure 2.2 on page 13).

We are not convinced that these stones have moved significantly in recent decades, let alone recent years.

If serious concern existed about progression, the rational response would be to establish a simple means of objective recording, and to monitor the situation over a few years.

4 Geometry

Measurements for Great Musgrave are presented in the 1998 Cumbria CC report. In the main, these are sufficiently redundant and consistent to give good confidence. There is one measurement, a quarter point crown rise, that is material to the assessment and is highly questionable.

The dimension sheets from the report are reproduced below. All dimensions – whether measured yourself or, as here, from an unknown source – need to be checked carefully for sense.

Figure 4.2 gives plan dimensions, with widths at each abutment, skew span at each elevation, and both diagonals.

The widths given are 6230 mm and 6260 mm. Converting to imperial, this suggests a width of 20.5 feet, or 6250 mm. The difference in width reported is unlikely to be real, not least because 6230 mm is 20 mm short for the likely design width.

The spans are also reported as 30 mm different. As the bridge was built on centres, which would not vary by this much, for this variation to be real would require the abutments to have moved after construction. The variations are small, however, and can be ascribed to measurement uncertainty.

We can estimate the skew angle from the span, width and diagonal dims using the cosine rule (eqn 4.1, with terms as shown in figure 4.1). Depending on the triangle chosen, the skew comes out at 12.7° to 12.8°.

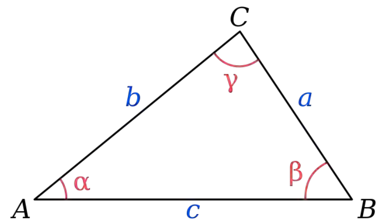


Figure 4.1: Triangle with sides and angles annotated for cosine rule.

$$c^2 = a^2 + b^2 - 2ab\cos\gamma \tag{4.1}$$

With skew bridges, it is important to establish whether the centres were placed skew or square. On over-line railway bridges, the answer is usually square, as the centre defines the loading gauge, and the same centres will be used for many bridges along the line. The skew span as measured, converted to the imperial measurements used at the time, is 27.6 feet to 27.7 feet. That is most unlikely to be the span of the centre, which would normally be a round number of feet or, occasionally, a sensible number in fractional feet.

With a skew span of 8450 mm, the square span would be:

$$8450\cos 13° = 8230\,\text{mm} = 27\,\text{feet}$$

That is quite short for a twin track railway span, but not impossible. Given the round number and the redundancy of measurement we have reasonable confidence in it.

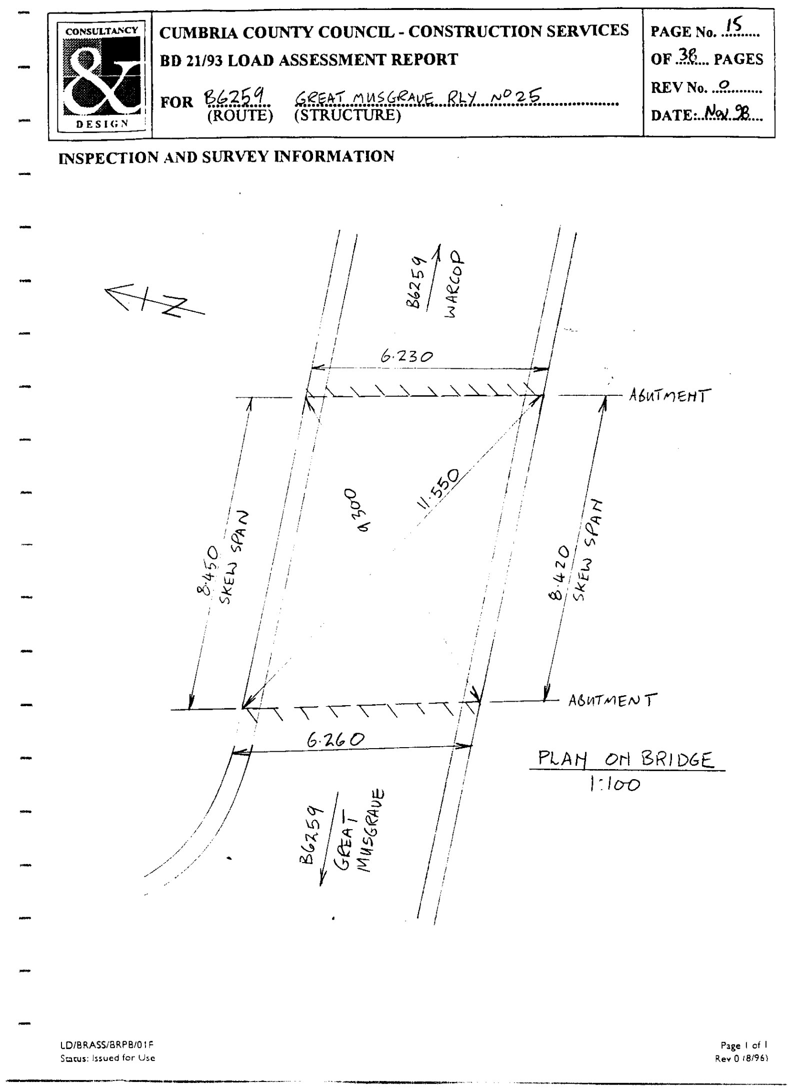
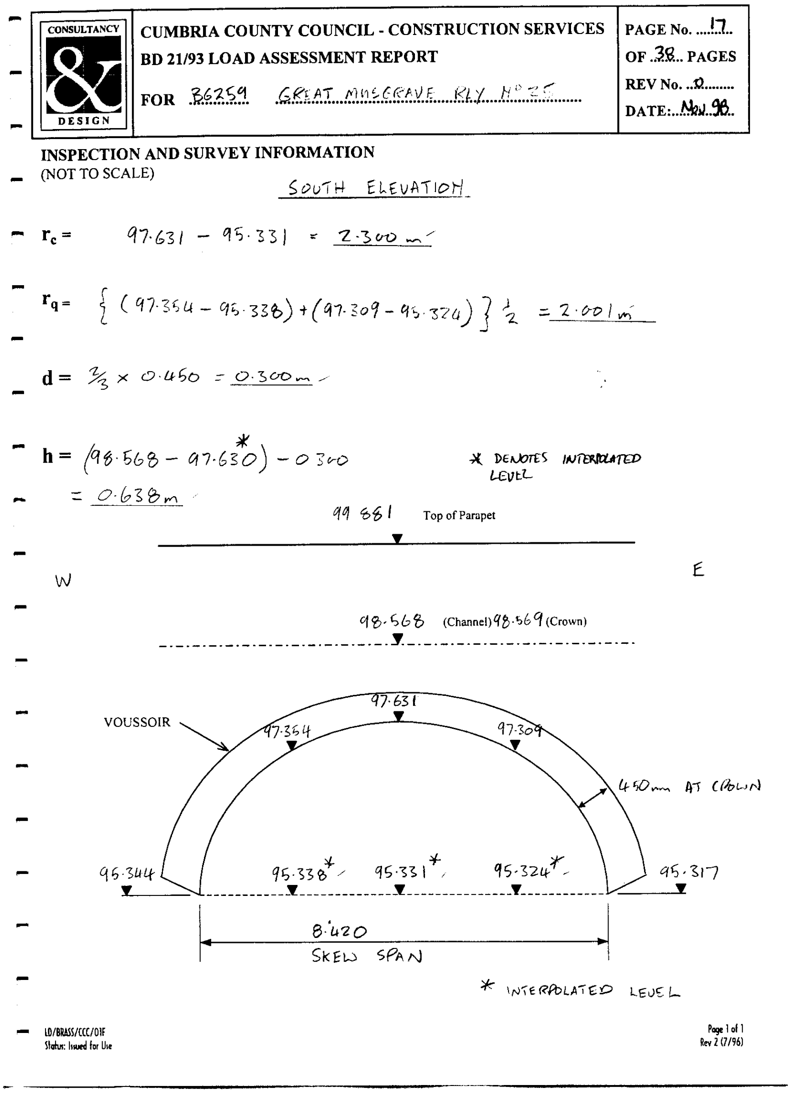


Figure 4.2: Plan dimensions from 1998 assessment report.

Thus the centres were 27 foot span and placed square to the abutments.



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CUMBRIA COUNTY COUNCIL - CONSTRUCTION SERVICES
BD 21/93 LOAD ASSESSMENT REPORT
FOR 36259 GREAT MUSGRAVE RLY No 25

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DATE: Nov 98

INSPECTION AND SURVEY INFORMATION
(NOT TO SCALE)

SOUTH ELEVATION

r_c = 97.631 − 95.331 = 2.300 m

r_q = { (97.354 − 95.338) + (97.309 − 95.324) } ½ = 2.001 m

d = ⅔ × 0.450 = 0.300 m

h = (98.568 − 97.630*) − 0.300
= 0.638 m

* DENOTES INTERPOLATED LEVEL

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Figure 4.3: South elevation dimensions from 1998 assessment report.

Figure 4.3 shows the south elevation dimensions. These are shown as levels, measured at the springings, quarter points, and crown.

We do not know how these levels were obtained. Measurements are reported to the nearest millimetre, which is unjustifiable whatever the method. The level difference at the springings of 30 mm is surprising and would warrant comment and checking. Quarter point measurements are particularly difficult, as the they are very sensitive to position in the span.

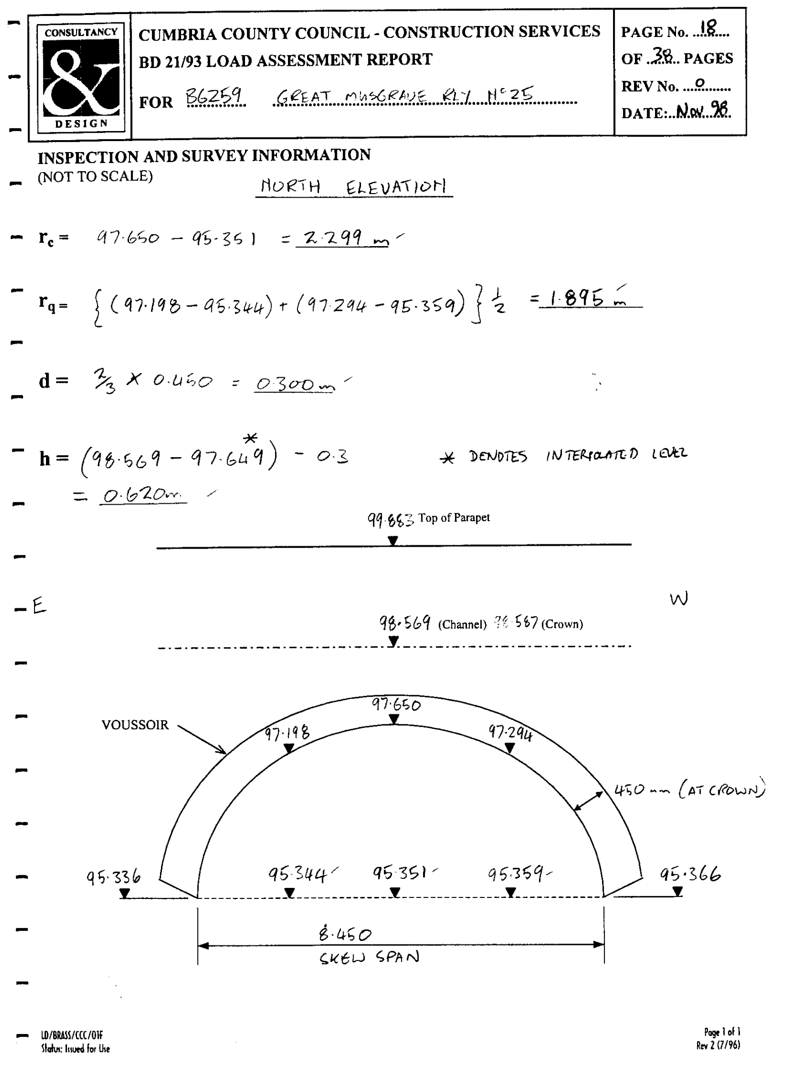
Crown and quarter point rise can be obtained from these by substracting soffit level from springing level. The crown rise is 2300 mm, or 7.55 feet. That this is slightly over 7.5 feet is a little surprising. The arch will normally drop at de-centring, and a rise that falls slightly short of a round number in fractional feet is more common.

27/7.5 gives a span:rise ratio of 3.6, which again is unusual. 7.5 feet would be a common rise for a 30 foot span, giving a 4:1 span:rise ratio.

The quarter point rises reported are 2020 mm and 1990 mm. These are not expected to be meaningful in feet, so staying in metric, the height of an 8230 mm × 2290 mm semi-ellipse at $x = 2060$ mm (the quarter point) would be:

$$\sqrt{(1 - 2060^2/(8230/2)^2) * (2290/2)^2} = 1980\,\text{mm}$$

So the shape is within measurement error of a true ellipse.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CUMBRIA COUNTY COUNCIL - CONSTRUCTION SERVICES
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FOR 86259 GREAT MUSGRAVE RLY Nº25

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INSPECTION AND SURVEY INFORMATION
(NOT TO SCALE)

NORTH ELEVATION

r_c = 97.650 − 95.351 = 2.299 m

r_q = {(97.198 − 95.344) + (97.294 − 95.359)} ½ = 1.895 m

d = ⅔ × 0.450 = 0.300 m

h = (98.569 − 97.649*) − 0.3 = 0.620 m

* DENOTES INTERPOLATED LEVEL

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Figure 4.4: North elevation dimensions from 1998 assessment report.

Looking now at the north elevation, dimensions in figure 4.4, things get less convincing. The 20 mm difference in level between the abutments is repeated here, but a drop of this scale across the span would show as distress in the spandrel wall masonry. A 20 mm level difference between elevations is also suggested, and such a slope across the abutments would be quite visible if one looked along the string course at the springing.

Rather than addressing the question of whether these tilts are real and, if they are, how they might have come about and what that means for the bridge, the 1998 assessment proceeds by taking them as given and interpolating between them.

The north elevation, east quarter rise is reported as 1940 mm, the west as 1850 mm. This difference comes about because the quarter span *levels* are recorded as 97.198 m and 97.294 m. A nearly 100 mm distortion to the arch would be clearly visible to the eye in the elevation. The implied transverse distortion allowing this deviation to dissipate between the north and south elevations would also be striking. This scale of movement would be very obvious indeed in the masonry of the elevation. The only possible conclusion is that the measurement is wrong, probably an error of transcription.

After correction, we have quarter point rises of 1940 mm and 1950 mm, compared with 1980 mm for the true ellipse.

These values for levels and rise show a range of deviations from likely truth. The bridge was undoubtedly level across both span and width at construction. Tilts of 20 mm in both directions would surely have visible effects.

Deviations of rise from expected values for a true ellipse are readily explained as measurement uncertainty. That these dimensions were not noted as unlikely and checked is disappointing, but typical.

So, although the span of 27 ft is unusual, as is the combination of a 27 ft span and a 7.5 ft rise in a true ellipse, the dimensions are consistent enough to provide reasonable confidence in the shape of the arch.

5 *Assessment*

5.1 *Quantitative assessment of masonry bridges*

The words quantitative assessment are carefully chosen, as is the description (masonry bridges) of the structures.

The assessment of Great Musgrave carried out in 1998 by Cumbria CC was based on the Modified MEXE method and is quite deeply flawed. The analytical basis of MEXE has only a tentative connection with the reality of behaviour. Pippard and Chitty, who developed the process understood how tentative that connection was and set out their reasons for (among other things) not including longitudinal distribution of live load between the surface and the arch.

Their underlying model was of a parabolic *arch* with a span rise ratio of 4, with pinned supports and a varying depth of ring.

They performed some calculations which purported to deal with the (always reduced) capacity of arches if their shape differed from this ideal. Their calculations made no allowance for the support provided by backing and so grossly underestimates the capacity of semi-elliptical bridges such as Great Musgrave.

Set against this, the elliptical shape produces a crown section which is flexible, without having the large dead load thrust which would be produced in a circular arc with similar crown "radius".

There are several other factors within modified MEXE, all of which are arbitrary in a black box way: we have no way of knowing how they were derived. Most of these have relatively modest effect individually but when compounded produce a large reduction factor.

5.2 *Cumbria CC assessment, 1998*

The geometry used for the Great Musgrave assessment in 1998 is that discussed in section 4. It is interesting to note that the quarter point rise used at each face is the *average* of the two rises recorded. Again, the supposed 100mm deviation in rise at the two quarters of the north side is averaged away without comment.

In the case of Great Musgrave bridge, the most significant factors are for joint and condition.

The joint factor is made up of three parts, joint width, mortar quality and joint depth. The values assigned are 0.9, 0.9 and 0.8.

These look like generous factors but quickly compound to 0.64. Some thought is needed. The joints are thin and filled with relatively soft mortar. The softness is important to provide a cushion between the stones, the combination of thinness and softness means that the cushion is provided with negligible reduction in the capacity of the arch to transmit thrust. The depth factor assumes that the loss of mortar recorded is over the whole intrados of the arch. In this case, the patches of loss are small and of very variable depth. In this case, a combined joint factor less than 0.9 is very hard to argue.

The condition factor is not well defined and entirely arbitrary, depending absolutely on the judgement of the engineer. A good description of its meaning would be:

To what extent does the engineer believe the assessed capacity should be reduced to account for pre-existing damage or deformation not covered by the existing factors.

The issue of shape is interesting. Measurement of shape is a poor indicator, especially if what is measured is only the mid point and quarter point rise. Here, the figures are quoted to 1 mm. The equipment available in 1998 was certainly not capable of measuring to that precision. A much better way is to stand back and look. If the arch appears distorted, some allowance may be necessary. If (as in this case) it does not appear distorted then it is almost certainly closer to the as built shape then any simple measurement can test.

The tightest definition of condition factor is that if a value less than 0.4 is assigned, the bridge is in need of immediate intervention. That rule is, naturally, applied in reverse. If the condition of a bridge frightens the engineer, they should give it 0.35 and demand immediate repair. The fact that the engineer gave this bridge a value of 0.75 shows that they had no such concern.

Among other factors that might make a significant difference to the assessment is the profile factor (in this case based on Rq/Rc values of 0.824 and 0.87 measured at the two sides, giving profile factors of 0.81 and 0.676. This reduction is quite unreasonable.

From an understanding of the basis of MEXE we would have assigned values of no less than 0.9 to any of these three, giving an over-all reduction of 0.72 rather than the 0.38 and 0.33 assigned here. This would lift the modified axle load from 7.5 tonnes to 15 tonnes, well in excess of the 11.5 tonnes for a 44 tonne vehicle.

5.3 *A look at capacity using Archie-M*

Figure 5.1 gives a view of the elevation and soffit. The water marks suggest backing at least to the sixth voussoir from the end but more likely the ninth, which would be more or less level with the arch crown intrados.

On that basis, the bridge would suffer no damage from any AWR vehicle. The worst case would be as shown in figure 5.2. For loads with impact, the worst case is shown in figure 5.3

Figure 5.1: A recent photograph showing elevation and soffit.

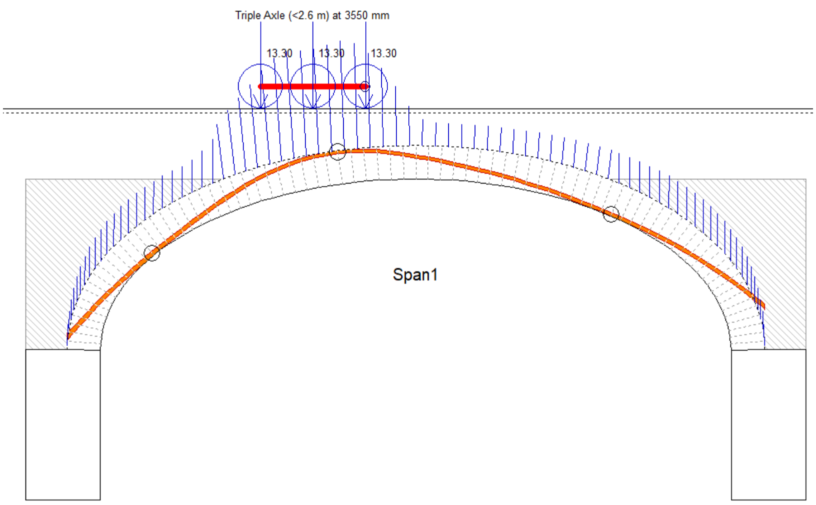

Figure 5.2: Worst case AWR load for Great Musgrave geometry, with backing to crown intrados level.

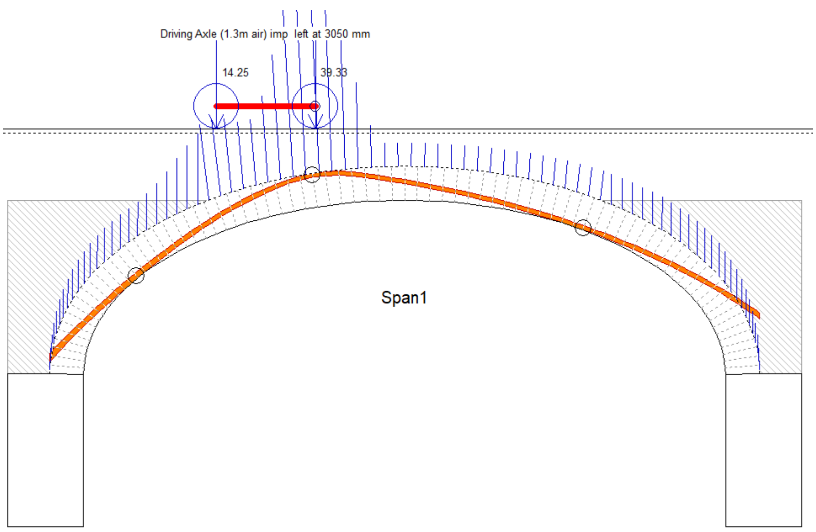

Figure 5.3: Worst case AWR load with impact for Great Musgrave geometry, with backing to crown intrados level.

Sensible application of either Modified MEXE or the Archie-M approach shows a capacity in excess of that required for 44 tonne vehicles.

In a rural area like this, it is extremely unlikely that the bridge has not been subject to maximum standard vehicle loads. Indeed, some un-sprung tractor trailers might produce more onerous loadings. If the bridge were to suffer damage from normal traffic it would surely have done so already. The available evidence suggests that is has not.

6 Severe damage and failure of masonry bridges

6.1 Collapse

None of the inspection reports so much as hint at a risk of collapse at Great Musgrave. The possibility of collapse is raised in email communication and the planning report.

In this section we look at a few actual cases of collapse and the typical causes.

The main cause of collapse of masonry bridges is scour. Scour undermines support, either at abutments or piers. Normally, if the damage does not progress to collapse during a flood event, clear evidence of distress develops before collapse. It is often possible to restore support and rescue a bridge that has suffered from — even severe — scour.

At Eastham in 2016, a three span bridge over a river collapsed. Tragedy was narrowly averted by a school bus driver observing the collapse in progress and reversing off the bridge. The event was some time after a significant flood. Scour was asserted as the cause. Sadly there is nothing to be learned from this case, as public forensic investigation so clearly necessary was not undertaken, and any internal investigation has not been published.

BHA Ltd. collected a set of photos and built a 3D model of the site shortly after the collapse: https://bhal.co/3d-eastham

Masonry bridges can collapse without applied load. The obvious case is collapse at or soon after construction. Most famously, William Edwards' Bridge in Pontypridd collapsed within a few weeks of construction, the crown bursting upwards. This was a very specific case, a result of the unusual geometry and thus weight distribution. The issue was rectified on rebuilding by removing weight from the haunches and adding weight over the crown.

Substantial collapse can occur after sustained neglect. A masonry bridge collapsed in Nottinghamshire in the early 1980s (figure 6.1). This took place under dead load only.

Loss of containment to the abutments can lead to gradual failure and could lead to collapse. This scenario is well illustrated by figure 6.2, which also shows clearly just how far the abutments need to move to allow not just distress, but collapse.

Figure 6.1: A masonry bridge in which the arch ring dropped out.

Cases of collapse are extremely rare. They are important, and should be thoroughly investigated. Their importance however is precisely in that the outcome is *very far from normal*.

Figure 6.2: Impending collapse of a decorative stone arch, showing the degree of abutment movement required to allow this.

6.2 *Severe damage to masonry bridges*

It is far more common for masonry bridges to sustain quite remarkable damage and yet to continue to support live loads. The damage can be a result of support movement, or live load.

Examples of the former are less numerous but not uncommon. Figure 6.3 shows part of an arch near Raymouth Road in London, where a pier suffered about 100mm subsidence. This elevation is inaccessible, and the movement probably developed slowly over many years. The railway above was unaffected, though presumably the track was realigned to compensate at intervals.

There are very many under-line bridges across the rail network with cracks that work visible as trains pass. While the risk to public from falling masonry is under-appreciated, these bridges continue to carry passenger and freight traffic.

Figure 6.3: Damage resulting from a pier dropping around 100mm. The structure continued to carry rail traffic without risk of collapse.

Highway loads are less severe, and there are relatively few masonry bridges on the trunk road network, but there are highway bridges in similar condition. Figures 6.4 and 6.5 show damage to a bridge carrying the A45 at Great Dunchurch. Neither here, nor on the rail network where similar damage is common, is this considered to be an emergency.

Figure 6.4: Damage to brick bridge at Great Dunchurch. Cyclic live loads have caused a crack to develop through the ring where the spandrel wall stiffens the edges.

Figure 6.5: Here, the crack has bifurcated, creating an island patch which has then fallen out.

There are bridges comparable to Great Musgrave on the Glasgow-Dumfries-Gretna line, built of good sandstone. Perhaps as many as ten bridges of 3.66m span suffered loss of mortar to the point where stones began to move visibly under load. The loads here were 25 tonne axles in very large numbers. One bridge reached a limit without intervention (figure 6.6). Stones dropped, in places by up to half the ring depth. There was still no possibility of collapse.

Figure 6.6: Severe damage to a bridge, with stones dropping due to mechanical action.

Any one stone could only drop as far as the neighbouring stones allowed. Figure 6.7 shows how spalling of the next stone is required to allow further movement.

Figure 6.7: A single dropped stone due to mechanical action. Note the clear damage to the surrounding stone, without which the stone could not have dropped this far.

Where there is no risk of scour, and no likelihood of loss of horizontal support to the abutments, deterioration due to live load is a long, slow process. There is ample warning of decreasing serviceability. The first risk to develop is from masonry dropping from the bridge, and even this risk does not develop without warning.

7 Can in-filling be effective?

It is suggested that in-filling removes both risk and future costs from the buried parts of the structure.

Masonry bridges are very stiff, which is to say that they move very little under a given load. BHA Ltd. have conducted live load deflection measurement at a range of structures. We have seen movements from as little as 0.5mm peak to peak under 100 tonne freight wagons, to at most a few millimetres in the vicinity of a damaged pier. We would expect movements under road traffic loads at Great Musgrave to be low single digit millimetres.

These very small movements do not mean that the structures can tolerate given loads. There are very many masonry structures, especially but not limited to viaducts, mostly but not exclusively on under-line rail structures, that are suffering severe cumulative damage from these loads.

The scale of these movements is however relevant to design of "strengthening" measures. In order for strengthening to work, live load has to be transferred out of the existing masonry and into the strengthening. To do this, any additional structure (which includes in-fill) has to be:

1. In firm contact with the existing structure.
2. Significantly stiffer than the existing structure.

Lack of contact is an extreme case of lack of stiffness. If there is no contact, there is nothing to stop the movement of the existing structure.

Lack of stiffness is more subtle. If you try to support the structure with sponge, it will clearly have no effect. If the existing structure moves by 2mm total under load, but the material used to "strengthen" the structure must compress by 5mm to develop a useful reaction to that load, then the strengthening will pick up negligible load.

We cannot see how in-filling with crushed stone and foam concrete can fill to the soffit completely, to within the fraction of a millimetre required to stop typical masonry bridge movements under live load.

Even if the foam concrete fills this completely initially, it will shrink away as it cures.

The crushed stone and concrete represents a new load on the trackbed, which will respond over time, opening the soffit gap further.

Within the embankment, the bridge will continue to move, and any damage from this movement will continue to accumulate.

If there were serious potential for collapse, it could be argued that the in-fill will stop this, by holding the parts of the structure in place. It cannot begin to act, however, until those parts drop enough to transfer load onto the in-fill.

If the structure within the embankment deteriorates sufficiently for in-fill to pick up any load, it will move enough that normal traffic causes rapid break-up of the road surfacing. Maintenance of the road would become impossible before the in-fill became effective at supporting the remnants of the structure.

There are very likely to be unforeseen consequences from in-filling. In particular, filling close to the soffit will create a permanently damp environment, which will accelerate degradation of both stone and mortar.

8 Summary and examination of arguments

We summarise by examining the arguments presented for in-filling in the retrospective planning application.

The arguments appear to be that, at the time of planning this work:

1. The structure is unable to carry 44 tonne traffic as is.
2. It is vital that the structure is safe to carry 44 tonne traffic.
3. Pointing was attempted but joints re-opened within 5 years.
4. The pointing failure was due to live load.
5. Stones in the arch crown are dropping progressively. (Some wording implies that the arch crown itself is dropping; there is no suggestion of this in the inspection reports.)
6. This drop is caused by a deficit of capacity to carry the traffic using the bridge.
7. Intervention is required to stop deterioration.
8. Pointing is no longer enough to increase capacity to allow use by 44 tonne traffic.
9. The only options available are complex, intrusive strengthening works, or in-filling.
10. An emergency situation exists or is likely to develop in the near future.
11. A risk of collapse may develop.
12. In-filling is capable of arresting deterioration and protecting against collapse.

Examination of the structure using in Archie-M (widely used across the UK for masonry bridge assessment) suggests that there would be no risk of collapse even with uniform mortar loss to 170mm depth.

We find no evidence that the soffit was pointed except in isolated areas, and the pointing in these areas was noted and, by implication, intact in recent inspections.

We see no evidence of live load damage to the joints in the soffit. Mechanical damage from live load movement is clear, and would extend across the width. The mortar loss is localised.

The only live load damage recorded in the inspection reports is an open joint at the extrados, and very slight oversailing of the ring by the spandrel walls. This joint was pointed in 2012, and there is no suggestion in later reports that it has re-opened. The oversailing of the spandrel walls is the cumulative result of traffic throughout the life of the bridge and is not of concern.

We are unconvinced that the soffit stones have moved in recent years. Measurement of defects such as this is highly dependent on method. A drop of 9mm could very easily be measured as 4mm on one occasion and 15mm on another.

If the stones were dropping as a result of live load action, there would be evidence of mechanical action in the joints around these stones. The photographs show none.

If stones dropped by 10mm between 2017 and 2020, the newly exposed face would be clean.

Installation of a passive device such as a Moiré Tell-Tale on angle brackets to measure out of plane movement would allow the rate of movement of dropped stones to be objectively determined.

See moiretelltales.com. Declaration: these are a product of Bill Harvey Associates. Other types of tell-tale would also work, but would require access at soffit level to read and are less sensitive to movement.

The soffit stones are wedge shaped. Even if odd stones are moving, they will naturally come to rest. They cannot fall out.

In the absence of evidence of live load damage, we can see no justification for urgent intervention. The masonry repairs proposed in recent reports are cosmetic.

Missing mortar is localised and discontinuous. Until it is continuous over a significant proportion of the width of the arch, it can have no impact on capacity. Missing mortar in perpend joints can have no impact on capacity.

There is no evidence of a capacity problem, so no strengthening is needed.

There is no evidence of rapid deterioration, so no likelihood of an emergency situation developing. The 2017 and 2020 inspection reports and HRE risk matrix sheets agree with this statement.

There is no possibility of a risk of collapse developing. This suggestion is preposterous, and would be so even if live loads were causing significant cumulative damage.

In-filling as undertaken cannot be and remain tight enough to the soffit to stop the sort of movements that cause damage in masonry bridges. It is thus incapable of stopping the accumulation of damage from live loads.

The slow outward movement of the spandrel walls as the centre span flexes elastically under load will not be modified by the in-fill. The in-fill does however remove the possibility of monitoring this.

Were collapse under load a plausible scenario, in-filling would stop this. If the structure was in such poor condition that collapse was likely however, it would also be highly mobile. In-filling would not limit this movement sufficiently to stop this damage progressing. Such movement typically causes damage to road surfacing, which in-filling would do nothing to address.