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A CRITICAL ANALYSIS OF THE OUSE VALLEY VIADUCT, WEST SUSSEX

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Abstract: This paper is a detailed assessment and critical analysis of the Ouse Valley Viaduct in West Sussex. The aesthetics, structural capacity, construction and durability will all be considered. Past restorations of the viaduct, as well as the possibility of any future changes or improvements will also be discussed.

Keywords: Viaduct, red-brick, multi-ring arch, railway bridge, heritage status

Figure 1: Ouse valley viaduct

1 Brief History

The Ouse Valley Viaduct is located in West Sussex, England and is locally know as the Balcombe Viaduct. It was completed in 1842 to serve the London to Brighton rail main line, carrying a two track railway across a valley and the River Ouse. It still carries around 110 trains a day.

The viaduct consists of 37 circular brick arches and pierced piers. It is 450 metres long and 28m high, as well as being a grade 2 listed building.

The majority of the structure was designed by experienced Railway Engineer John Rastrick, who with his partner built only the third steam locomotive to be made in the world. Other aspects of the viaduct, such as the ornamental parapets and pavilions are credited to architect David Moccata Ref [1].

During construction the 11 million bricks and limestone, for the parapets and pavilions, were transported from Holland up the River Ouse in barges to a wharf in the vicinity of the viaduct.

Throughout its life there have been many restorations to the viaduct. The most recent of which was a £6.5million operation between 1996 and 1999 by Railtrack Ref [2], which has since been taken over by Network Rail. This restoration replaced the severely weathered limestone parapets and also repaired the brickwork of the arches and piers.

2 Aesthetics

The aesthetics of this viaduct will be analysed against the perspective of Fritz Leonhardt, arguably the most famous bridge engineer of the 20th Century. He believed that there are ten main areas of bridge aesthetics, all of which will be considered in this paper to try and achieve a sense of objectivity.

2.1 Function

The structural system is simple and clearly expressed through multiple red brick arches and pierced piers which transfer the loads down into the

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foundations and subsequently into the ground. The deck and piers are not excessively large or small, demonstrating good functionality of the elements. Masonry arches are one of the earliest and simplest forms of construction, dating back to before Christ, but it was the Romans who were the first to realise the potential of this form of construction. They provide a strong sense of strength and stability to the observer through its shape and size, both as individual arches and collectively.

2.2 Proportions

The Ouse Valley Viaduct is probably best known for its elegant proportions. When looking at the viaduct transversely, the balance of its solids (the deck, piers and arches) with its voids results in the structure to appear well poised. This is achieved through well sized piers in comparison to the span of the arches. If the piers were thicker or spaced closer together, it would cause the viaduct to seem excessively engineered and start to create an opaque barrier, which is aesthetically unpleasing. On the other hand if the piers were to be thinner or further apart, they would be perceived to look too slender and the structure as a whole to seem unstable.

Arguably the viaducts most graceful proportions lay within the piers, see figure 2.

Figure 2: Pierced piers

The oval piercing of the piers allows for more light to pass through them whilst not compromising the sense of rigidity that the piers provide. These apertures also saved weight and cost when the viaduct was constructed.

There is however room to criticise the viaducts proportions. The spacing between the piers and therefore the span of the arches are constant throughout the length of the bridge, but the ground level rises at both ends of the bridge, see figure 4 on the following page.

This leads to the piers being shorter at the ends and the area of the voids between piers smaller at the ends as well, displaying an element of poor balance, even if a minor one.

The apertures in the piers themselves look out of proportion and shape when condensed down into the shorter end piers.

2.3 Order

The repetition of the arches and piers, both their shape and size, gives good order to the structure. Both the piercing of the piers and the shape of the parapets resemble the shape of the arched apertures between piers, obtaining an aspect of visual continuity within the bridge. The edges of the viaduct and its parapets run straight and parallel to the railway tracks. These ordered parameters leave an aesthetically pleasing and balanced appearance.

2.4 Refinements

The major refinement of the structure is the piercing of the piers, to allow light to pass through and relieve a potential opaque visual barrier. The piers are also tapered both longitudinally and transversely, meaning they are thicker at the base than at the top. This method was used by the Greeks and prevents the optical illusion that the top of a parallel-edged pier looks wider than its base, therefore looking top heavy and illogical.

At both ends of the viaduct there are four Italianate pavilions, shown in figure 3. These look to ‘guard’ the entrances to the viaduct and give it a clear sense of closure, by highlighting the ends of the bridge.

Figure 3: Single Italianate pavilion and parapets

Another interesting refinement of the structure is the choice of material used to construct the viaduct. 11 million red bricks were transported from Holland to construct the viaduct. Limestone was brought from Caen, Normandy for the pavilions and parapets, which were designed by David Mocatta the Architect. This is likely to be because there were not the resources in the UK for the magnitude of brickwork needed and this limestone was desired due to its homogenous nature, which makes it suitable for carving to achieve the shapes present.

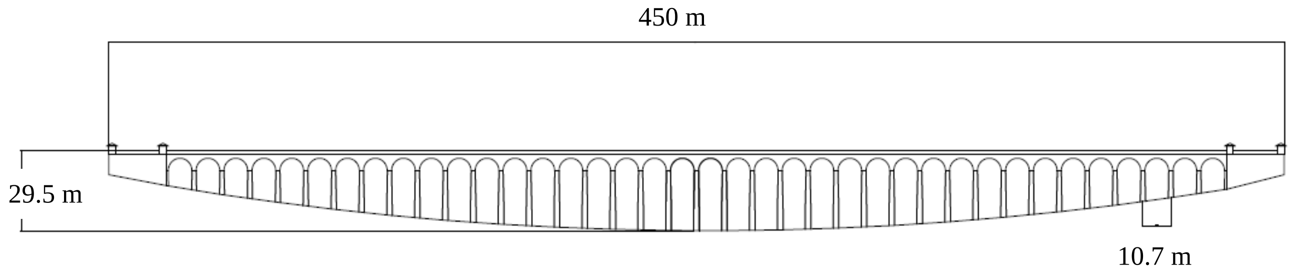

Figure 4: Elevation of viaduct with basic dimensions

2.5 Integration into the Environment

The viaduct passes through a valley in a very rural area, with practically no buildings within the proximity of the site. It also passes over, what was and still is a popular public country walkway. It was therefore important to construct this bridge so that it does not become too imposing and intrusive on the surrounding area.

It easily achieves this and more. It fits the ground contours seamlessly without scouring the ground or altering the shape of the environment. The use of the arches enables anyone on ground level to pass under the bridge and view through it. Also the use of red brick as the primary construction material gives a rustic texture that fits the viaducts agricultural surroundings.

2.6 Texture and Colour

Texture is something that is often ignored in modern bridges, but not in this case. The use of red brick as the primary construction material was common during the period of its construction, and gives exceptional texture and colour appeal.

This is contrasted by the smooth limestone which is of a lighter fairer colour. This is to attract the observer's eye to the slender deck and elegant pavilions and parapets. This material has since stained and weathered considerably over time, slightly detracting it from its original colour and texture. However during many restorations of the bridge the majority of this deteriorated limestone have been removed and replaced. Also different brick sections have been replaced with stronger bricks and mortar, to supposedly relieve stresses even though it resulted in the opposite effect. These bricks have a different colour and leave a distinguished contrast which leaves the structure looking disjointed and blemished, but also enhance its aged look which demonstrates character, shown in figure 5.

Figure 5: Opposing brickwork within piers

2.7 Character

It is difficult to define whether or not a bridge has 'character' as it is a very subjective matter. However, as this viaduct has been standing for 170 years and has been given grade 2 heritage status, this proves that the building must have some character. Its rich colour and texture along with its history, and that it still carries a vast amount of traffic on a major rail line, further ads to this quality.

2.8 Complexity and Nature

This structure shows very little if any real complexity in its structure, it is the simplicity of its repetitive arches and piers that makes it so appealing.

The use of nature as a factor in the design in this bridge is not apparent; however nature has had its effects on it since its construction. Such as the weathering of the limestone and brickwork, altering their colour and it is the surrounding nature that gives the bridge the backdrop that makes it so beautifully poised.

3 Loading

The loading the structure is subject to will be calculated from dimensions obtained both from Ref [3] and a survey that I did myself. These values will be used in accordance with formulas from BS 5400 to get the loading conditions.

3.1 Dead Load

The dead load is the load that is created by the weight of all the structural elements within the structure. For this viaduct this broadly implies the masonry brickwork. More specifically this consists of the abutments, foundations, piers, arches, spandrel walls and wing walls.

I have assumed the abutments to be solid throughout as there is not sufficient evidence on the size of the hollow sections within them. This will lead to a conservative dead loading value, as in fact there are hollow areas within the abutments. This is to lighten them.

The density of the brickwork will be assumed to equal, 21 kN/m^3.

Table 1: Structural elements and their volumes

Structural Element	Total Volume (m^3)
Pierced Piers	7 236
Abutments	798
Pier Foundations	976
Arches	392
Wing Walls	594
Spandrel Walls	2532
TOTAL	**12 528**

Taking the area of the bridge in plan (450m length by 8.7m breadth), the approximated dead load is:

$$\text{Dead Load} = \frac{12\ 528 \text{ x } 21}{(450 \text{ x } 8.7)}$$

$$\approx 67 \text{ kN/m}^2$$

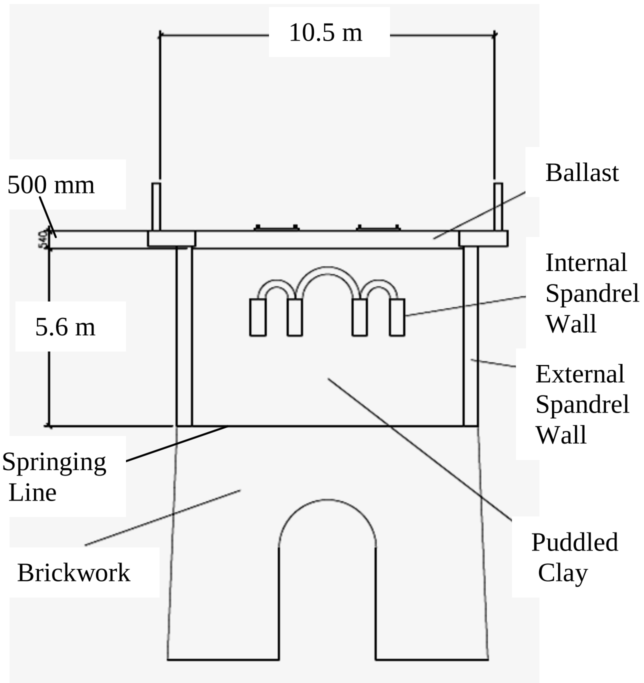

Figure 6: Section through centre of pierced pier

3.2 Superimposed Dead Load

This load is the rest of the permanent loads on the structure that aren't structural elements. These are the parapets and pavilions (Caen limestone), ballast, infill (engineering puddle), rails and sleepers. In my calculations I have assumed the volume of the sleepers to be filled up by ballast instead to make the calculations easier and a little conservative.

Figure 6 shows a section through the centre of a pier, showing the viaducts internal constituents, which were given in ref [3].

Table 2: Values to calculate superimposed dead load

Material	Volume (m^3)	Density
Caen Limestone	219	25.6 kN/m^3
Puddled Clay	6937	19.2 kN/m^3
Ballast	1780	18 kN/m^3
Rails	-	0.7 kN/m length

Superimposed dead load =

$$\frac{(219\text{x}25.6) + (6937\text{x}19.2) + (1780\text{x}18) + (0.7\text{x}450\text{x}4)}{(450\text{x}8.7)}$$

$\approx$ 44 kN/m^2

3.3 Live Load:

Standard railway loading consists of two types. RL loading is reduced loading for when only passenger rapid transit railway systems are in use and no freight is transported along the railway. In this situation however, freight is occasionally passed over the viaduct therefore it is subject to RU loading. This allows for all combinations of normal vehicles running or projected to run in the future, in Europe.

Nominal type RU loading, per railway, is shown in figure 7.

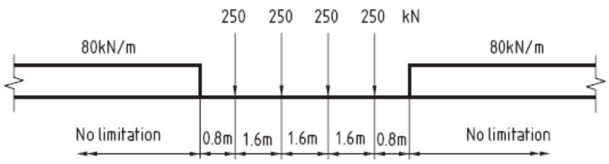

Figure 7: Nominal RU loading Ref [4]

The concentrated loads on the rail will be distributed both longitudinally by the continuous rail to more than one sleeper and transversely by the sleeper and ballast.

However this load can be assumed to be uniformly distributed at a depth of 800mm or more from the underside of the sleeper. Since the supporting structures, the internal spandrel walls and top of the arches are 1.5 metres from the bottom of the sleeper, the live load can be assumed as a 133 kN/m length uniformly distributed load for each rail track, therefore for this viaduct would come to 266 kN/m.

This is then multiplied by an appropriate dynamic factor for evaluating bending moments and shear loading. This is to allow for dynamic effects including those caused by track and wheel irregularities. Looking at Table 16 in BS 5400 part 2 and using half the arches span as L, these dynamic factors are 1.84 when evaluating the structure in bending and 1.56 in shear.

3.4 Wind Loading

The worst wind loading combination is horizontal wind loading with vertical wind, whether uplift or downwards force. There can also be longitudinal wind effects but these are insignificant in comparison to the two major wind loads, which will both be discussed.

3.4.1 Horizontal wind loading

The horizontal wind load will act transversely as a UDL on the surface area of one side of the viaduct facing the oncoming wind. Before this force can be calculated the maximum wind gust speed, V_d, is obtained from this equation:

$$V_d = S_g \times V_s \tag{1}$$

In this equation $V_s$is the site hourly mean wind speed obtained from:

$$V_s = V_b \times S_p \times S_a \times S_d \tag{2}$$

The basic hourly mean wind speed, V_b, is taken from a map of isotachs shown in BS 5400-2. In the area of West Sussex where this viaduct is, V_b equals 22 m/s. The values on this map are obtained from an equivalent return period of 50 years in flat open country at an altitude of 10m above sea level. These assumed values are altered and corrected to fit each different case by many different factors.

The probability factor, S_p, is taken as 1.05 for railway bridges, which give a more appropriate return period of 120 years due to the age of the structure.

The altitude factor, S_a, is used to adjust the relevant altitude of the site above sea level and is calculated to be 1.3.

The last factor used in calculating the basic hourly wind speed is the direction factor, S_d. This is to adjust the basic wind speeds to produce wind speeds with the same risk of exceeding the allowable the speed in any direction, and will be taken as 1.00. With all these factors V_s can be calculated:

$$V_s = 22 \times 1.05 \times 1.3 \times 1.00$$
$$\approx 30 \text{ m/s}$$

Looking back at Eq. [1] the gust factor, S_g, can be calculated from the following formula.

$$S_g = S_b \times T_g \times S_h' \tag{3}$$

Where, $$S_b = S_b' \times K_F \tag{4}$$

S_b', is the bridge and terrain factor taken as 1.65 and K_F is the fetch correction factor which shall be taken as 1.00, therefore S_b equals 1.65.

T_g is the town reduction factor and since the Ouse valley viaduct is in a very agricultural area, and the nearest town in the upwind direction is further than 3 km away, this factor can be taken as 1.00.

As the viaduct crosses a valley, the topography factor, S_h', should be calculated from specialist advice and be no less than 1.1. In this case it will be assumed to equal 1.1 as it is a wide and shallow valley, which shouldn't tunnel the wind significantly.

Therefore the gust factor can be calculated:

$$S_g = 1.65 \times 1.00 \times 1.1$$
$$\approx 1.8$$

The maximum wind gust speed, V_d, can now subsequently be calculated from Eq. [1]:

$$V_d = 30 \times 1.8$$
$$= 54 \text{ m/s}$$

This value can then be used to calculate the horizontal wind load, P_t, acting at the centroid of the transverse surface area facing the oncoming wind.

$$P_t = q \times A_1 \times C_D \tag{5}$$

Where, $$q = 0.613 \times V_d^2 \tag{6}$$
$$= 0.613 \times 54^2$$
$$\approx 1788 \text{ N/m}^2$$

A_1 is the solid surface area facing the oncoming wind. This consists of the piers, arches, exterior spandrel walls, wing walls, parapets and pavilions. It has been calculated as 3160 m^2.

C_D is the drag coefficient, which is a function of the b/d ratio of the deck. The deck of this viaduct does not fit any of the standard cases covered in BS 5400 so in practice it would need to be modelled in a wind tunnel to achieve a higher degree of accuracy. For this paper it will be assumed that b/d is approximately 5, which gives a drag coefficient of 1.3. The horizontal wind load for the whole structure can now be calculated:

$$P_t = 1788 \times 3160 \times 1.3$$
$$\approx 7345 \text{ kN}$$

This seems very large, but this large value is due to the significant magnitude of the viaduct both in length and height. This force distributed uniformly over the area of this face is equal to 2.3kN/m^2 which seems reasonable.

3.4.2 Wind Uplift

The underside of the bridge is open to the wind therefore wind uplift, P_v, should be considered. The equation is similar to that of horizontal wind loading:

$$P_v = q \times A_3 \times C_L \qquad (7)$$

As before q is the dynamic pressure head and is the same value of 1788 N/m^2. A_3 is the underside area that the wind acts on, which equals the sum of all the underside of the arches, which is 4835 m^2. C_L is the lift coefficient which comes from a chart using the b/d ratio again, it comes to 0.4. Hence:

$$P_v = 1788 \times 4835 \times 0.4$$
$$\approx 3458 \text{ kN}$$

3.5 Temperature Effects

Both daily and seasonal fluctuations in shade air temperature and solar radiation can affect the materials within the bridge superstructure and can cause additional stresses and strains.

British standards provide data and charts that give the maximum and minimum shade air temperatures for different areas of the UK and a 120 year return period. It gives a minimum temperature of 35°C and a minimum of –16°C, leaving a range of 51°C.

To calculate the strain induced by temperature change Eq. [8] is used:

$$\varepsilon = \alpha \times \Delta T \qquad (8)$$

In determining ΔT, its value is not the difference between the maximum and minimum shade air temperatures (51°C). The temperature at the time the structure is effectively restrained shall be taken as the datum in determining ΔT. Assuming the viaduct was built at a when the surface air temperature was somewhere in the middle of the extreme temperatures of 35°C and –16°C, ΔT will be assumed to equal 27°C.

The thermal coefficient of expansion, α, differs between every material. For brickwork its value is 5.5 x 10^{-6} per degree Celsius. The strain can now be calculated:

$$\varepsilon = (5.5 \times 10^{-6}) \times 27$$
$$\approx 149\ \mu\varepsilon$$

This strain would usually lead to an expansion of materials in the bridge superstructure, however different elements of the structure, such as the piers and arches, form a restraint for this associated expansion. Restraining this expansion induces thermal stresses within the structure. These are calculated through Eq. [9]:

$$\sigma = \varepsilon \times E \qquad \$ \qquad (9)$$

The young's modulus, E, of brickwork is 15 GPa which equals 15 x 10^3 N/mm^2. Therefore;

$$\sigma = (149 \times 10^{-6}) \times (15 \times 10^3)$$
$$\approx 2.2 \text{ N/mm}^2$$

This stress will apply throughout the whole bridge and the force that this stress exerts on a certain area can be found by multiplying the stress with the area exerted on.

3.6 Oscillation Effects

Oscillations will not be an issue in this viaduct due to its large size and self weight. The depth of the deck at its shallowest, above the centre of the arch, is still significantly large to prove the deck to be substantially stiff in the vertical direction and not be susceptible to major oscillations or resonance. Light, flexible structures such as footbridges are more prone to these effects.

3.7 Impact Loads

Accidental Collisions with bridges occur predominantly from highway vehicles. This could occur through vehicles colliding with the base of the pier, however the piers are over 150m from the nearest roadside so the risk is substantially small to disregard.

The more major risk is the collision of a locomotive that has derailed, with the parapets. The parapets themselves are less than half a metre thick and would probably not prevent a train travelling at 50mph passing through it. However it would be very impractical to design these parapets to resist this impact load, as the size of the parapets would increase significantly. Also as the viaduct is straight, there is a reduced risk of a train derailing, though should still be considered.

3.8 Derailment Loads

Railway bridges now have to be designed so that they do not suffer excessive damage or become unstable in the event of derailment. The following conditions must be taken into consideration through equivalent static design loads.

For serviceability limit state, derailed coaches or light wagons that remain close to the track shall cause no permanent damage. The equivalent loading for this is a 100kN concentrated vertical load anywhere within 2m either side of the track centre line.

For the ultimate limit state, derailed locomotives or heavy wagons that remain close to the track shall not cause collapse of any major element, however local damage is accepted. The design loads applied for this are eight concentrated vertical loads each 180 kN, arranged on two perpendicular lines 1.4m apart, on each of these lines four loads are applied 1.6m apart. This loading can be placed anywhere on the deck.

For overturning or instability, a locomotive and one following wagon that are balanced shall not cause the structure as a whole to overturn, however other damage is accepted and probably expected. This is demonstrated via a single vertical load of 80 kN/m applied along the parapet to a maximum length of 20m.

These three load cases should not be considered together but separately, as the summation of these loads would cause the structure to be over engineered or assessed to take an unrealistic loading condition.

3.9 Creep

Creep is only regarded as an issue for structures consisting concrete and as this viaduct contains zero concrete it will not be considered a problem.

4 Analysis

The primary method used for assessing masonry arch bridges in the UK is the MEXE method. This method can determine a bridges load carrying capacity as shown here. The method has been largely taken from Ref. [5].

4.1 MEXE strength assessment

This method was used extensively during World War II to classify bridges according to their capacity to carry military vehicles. Since then it has been adapted for civil use and compromises of a calculation for a provisional axle load (PAL):

$$PAL = 720\ (d + h)^2 / L^{1.3}$$

L is the span of the arch, equal to 9.144m in this case. d is the thickness of the arch adjacent to the keystone and is taken as 0.457m. h is the average depth of fill at quarter points of transverse profile which is approximately 1.6m. However there is a maximum limit of 1.8m for (d + h) so this will be taken instead. Therefore:

$$PAL = 740 \text{ x } 1.8^2 / 9.144^3 \quad (10)$$

$$\approx 135 \text{ tonnes}$$

This provisional axle load is then modified by multiple different factors to give a modified axle load (MAL) shown in the following expression.

$$MAL = F_{sr} \text{ x } F_p \text{ x } F_m \text{ x } F_j \text{ x } F_{cm} \text{ x } (PAL) \quad (11)$$

The span/rise factor (F_{sr}) assumes that steeper profiled arches are stronger than flatter ones. A span/rise ratio of 4 or below is seen as the optimum. The arches of this viaduct are semi-circular therefore have a ratio of 2, hence a factor of 1.

The profile factor (F_p) takes account of the different shape of the arch. For elliptical or semi-circular arches it can be calculated by:

$$F_p = 2.3[(r_c - r_q) / r_c]^{0.6} \quad (12)$$

r_c is the rise at the crown and r_q is the rise at quarter point, their values are 4.572m and 3.962m respectively, these values are shown in figure 8.

$$F_p = 2.3\ [(4.572 - 3.962) / 4.572]^{0.6}$$

$$\approx 0.71$$

The material factor (F_m) is determined using the following expression:

$$F_m = [(F_b \text{ x } d) + (F_f \text{ x } h)] / (d + h)$$

Where F_b is a barrel factor which is taken as 1.00 for building bricks, and F_f is a fill factor assumed to equal 0.7 for well compacted materials, this presumes that the puddle clay infill is well compacted.

$$F_m = [(1 \text{ x } 0.4572) + (0.7 \text{ x } 1.6)] / 1.8$$

$$\approx 0.88$$

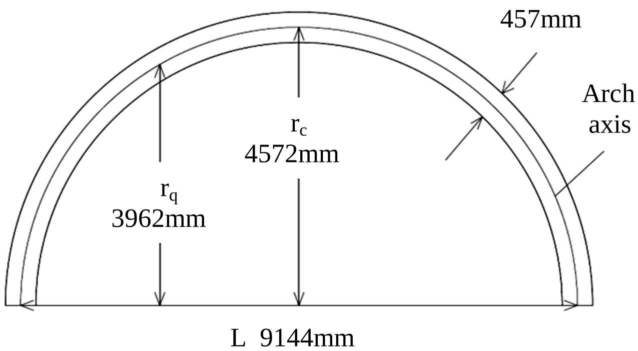

Figure 8: Single semi-circular arch and its dimensions

The joint factor (F_j) is determined by the following equation:

$$F_j = F_w \text{ x } F_d \text{ x } F_{mo}$$

The width factor (F_w) is taken as 0.9. Depth factor (F_d) presumed as 0.9, and the mortar factor (F_{mo}) equals 1.0. These values have been taken from conservative assumptions of the viaduct and the state of its joints and mortar, due to the age of the structure. The joint factor value comes to:

$$F_j = 0.9 \text{ x } 0.9 \text{ x } 1.0 = 0.81$$

Finally a condition factor, F_{cm}, is applied. This relies heavily on engineering judgement and the confidence in the calculations, giving a value between 0 and 1.0. For this case a value of 0.8 will be taken, however this is a broad assumption.

The modified axle load can now be calculated:

$$MAL = 1.0 \text{ x } 0.71 \text{ x } 0.88 \text{ x } 0.81 \text{ x } 0.8 \text{ x } 135$$

$$\approx 55 \text{ tonnes}$$

However this assumes that the bridge is single span between abutments, which this viaduct is not. One modification factor that was suggested at the time of the original development of MEXE was a value of 0.8 for arches supported on two piers. This leaves a further modified value of 44 tonnes, however this factor appears to have no theoretical justification.

This final value is the maximum permissible axle load from a double-axled bogie. As these bogies are approximately 1m between each axle, this MAL will be assumed to equal a permissible 440kN/m of live loading over the length of the bridge.

Looking at section three, the live loads that British Standards states a railway bridge should take are 133 kN/m for each track, therefore for this bridge would be a total of 266 kN/m, which according to this method of analysis this viaduct is capable of.

4.2 Strength

From the loads discussed in section three of this paper, different load combinations can be applied to the bridge to check its strength capacity. In this case the condition that will be assessed will be the addition of all the permanent loads (dead and superimposed), primary live loads and the appropriate live loads.

Impact and derailment loads will be ignored, as the live loading assumes the locomotive and its wagons are on the rails. The combination taken is for ultimate limit state and is given by the following expression:

$$w = (1.15\ \text{Dead}) + (1.2\ \text{Superimposed}) + (1.2\ \text{Live})$$

These factors of safety are for concrete as none are given for masonry. Using the previously obtained values in section four, the UDL under ultimate limit state that the bridge is to bear under British Standards is:

$$w = (1.15 \times 67 \times 9.1) + (1.2 \times 44 \times 9.1) + (1.2 \times 266)$$

$$\approx 1501\ \text{kN/m length}$$

The compression within the arch shall be checked under these loading conditions.

To determine the compressive force the horizontal and vertical forces at the springing point must be calculated and a resultant formulated. L is the span and f is the height of the arch.

$$V = \frac{w \times L}{2} = 6863\ \text{kN}$$

$$H = \frac{w \times L^2}{8 \times f} = 3432\ \text{kN}$$

The resultant force comes to equal 7674 kN. The stress can now be calculated in the arch by dividing this force by the arches cross-sectional area shown in the following equation.

$$\sigma = \frac{7674 \times 10^3}{(9.144 \times 0.457) \times 10^6}$$

$$= 1.8\ \text{N/mm}^2$$

The compressive strength of brickwork can be as low as 25N/mm^2, therefore even with low strength brickwork the arches of the bridge would be able to take this loading, and most probably be able to take derailment loads and additional wind loading.

5 Construction

Due to this viaduct being constructed nearly 160 years ago there is very little information to be found on the exact construction process. However as many similar bridges were constructed in this era during the development of 'The railway age' there are many bridges that can be assumed to have been constructed in a similar manner.

One such bridges is the Brackley Viaduct in the South-West of Northamptonshire. This viaduct is very similar to the Ouse Valley Viaduct as it consists of multiple semi-circular arches and tapered piers all made from brickwork and has its construction explained in Ref. [6].

Firstly the areas underneath the multiple piers must be excavated to accommodate for the foundations. Each pier has two courses of footings, which together were just over 1m deep and constructed of brickwork. Then the piers would be built up from ground level to the springing line. The abutments and end wing walls would also be constructed at this time.

Once all the piers have been completed, falsework would be built and positioned between two piers at one end that would form the shape of the arch and provide support during its construction. This formwork would usually be made from timber as it can be cut and shaped on site easily. This is shown in figure 9.

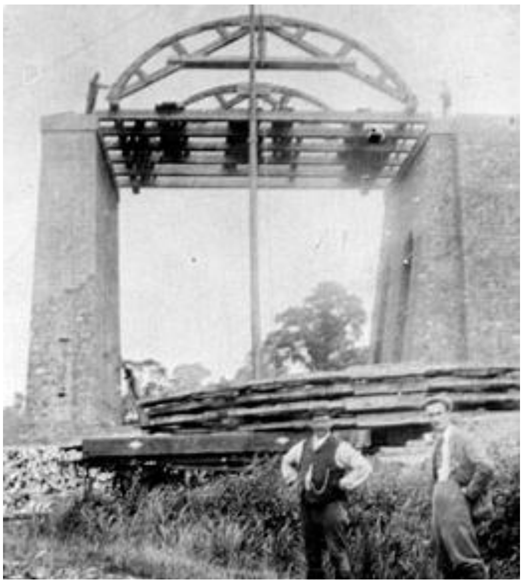

Figure 9: Typical arch timber falsework

The arches would be constructed from each pier to meet at the middle at the crown of the arch. Once the arch had been completed the exterior spandrel walls would then be constructed and the formwork could be removed from under the arch. This was done between adjacent piers one at a time before the next arch began to be built, demonstrated in figure 10.

Figure 10: Sequential arch construction

Once all the arches were built and falsework removed, the bridge is seen to be substantially self-stable. The interior, between external spandrel walls, would then be filled with its infill (puddle clay) to a desirable level. This is most likely to be done manually. On top of this the interior spandrel walls would then be constructed above the piers between adjacent arches, as previously shown in figure 6.

The rest of the puddle clay would then be laid, before the parapets and pavilions would have been built. The limestone for their construction was brought by barge via the river Ouse, much like the bricks. Finally the ballast, sleepers and rails would have been placed to complete the construction process.

6 Foundations and geotechnics

Each pier has two courses of footings, which taken together are in total just over 1m deep. These footings are inclined, as shown in figure 11. This is to increase the area that the vertical force from the pier acts on, which greatly reduces the chance of any differential settlements.

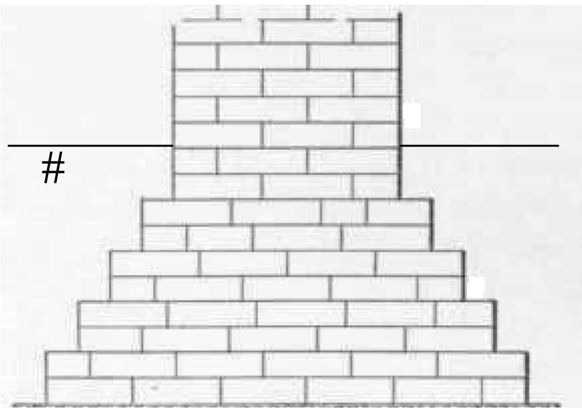

Figure 11: Spread brickwork footing

No evidence can be found to suggest the precise area of the base of the footings; however there have been no real settlement issues to this date proving these footings are substantial for the ground conditions on the site.

7 Serviceability

Arches perform best under uniformly distributed loads but are known to deflect and deform under point loads, specifically at the quarter span position.

Railway bridges can have point loads distributed through axles onto the tracks, however all structural elements are deep enough below the rails that these point loads are treated as uniformly distributed over an effective length. This is predominantly the issue for a single-span arch bridge or if a single arch was loaded in a multi-span arch bridge, however deflections and deformations could also occur if the bridge was loaded asymmetrically, shown in figure 12.

Figure 12: Exaggerated deformation

To analyse this properly a computer finite element package would be used due to the complexity of the issue.

8 Durability

8.1 Chemical attack

The limestone that the parapets and pavilions are constructed from is particularly susceptible to chemical attack. Acidic gases emitted into the air from industrial areas, agricultural pesticides or from cleaning agents, ironically used to clean the stone, attack the surface of the limestone as a gas, or could dissolve into precipitation and form acid rain which can be harmful to the whole structure.

8.2 Frost attack

The destructive effect of frost is due to the 9% increase in volume that occurs when water turns from a liquid to ice at 0°C.

When bricks and mortar are saturated full of water, expansion within their pores may set up internal stresses. The UK is one of the most prone areas for frost attack, due to its frequently fluctuating weather conditions, which gives rise to recurring freeze/thaw cycles.

Failure can be noticed through brick surfaces flaking, while the mortar joints may crumble. It is difficult to tell whether the cracking, flaking and crumbling of the brickwork that has occurred in this bridge before, is solely due to frost attack. However it can be assumed that it is one of the many factors that led to the materials failure.

8.3 Ring-separation

The arches in this structure are multi-ring construction, shown in figure 13. Each ring was originally bonded to the adjacent rings to ensure that the arch acts as one structural element and the rings act collectively. However stresses in the mortar between these rings can cause delamination to occur and this phenomenon known as ‘ring-separation’. The only force acing between different rings after this failure is friction.

This mode of failure is very common for structures of this type and can significantly reduce its carrying capacity. This has been seen to occur both in the main arches and piercings of the piers, which led to repair works to be carried out.

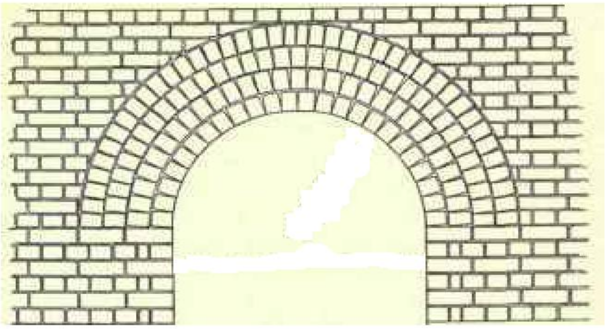

Figure 13: Multi-ring construction

9 Restoration, improvements and future changes

As mentioned at the start of this paper, major restorations were carried out on the viaduct between 1996 and 1999, without closing the bridge for train use. The major reason for concern was the state of the limestone parapets and plinths. They had been so badly weathered that sections were beginning to fall off. After much debate affected sections were replaced by another French limestone, Richemont Blanc, from a quarry near Bordeaux. It was chosen due to it being a close match in colour and texture to the original Caen limestone. Cores were made through the new and remaining stone and stainless steel anchors were inserted to tie both the new and remaining cornices in, to avoid future falls.

Due to its heritage status, any unnecessary changes will certainly be prevented by the appropriate authorities. However the structure has already outlived a design life of 120 years, so modifications and improvements are expected on such an old bridge. Network Rail, who own the viaduct, now perform visual checks every year on all railway bridges and physical checks every seven years to make sure they meet the high standards needed.

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