Evaluating the residual life of aged railway bridges

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Evaluating the Residual Life of Aged Railway Bridges

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Abstract

The United Kingdom is home to a very expansive railway network. The network includes a significant number of bridges that were constructed in the Victorian era. The aim of this study is to estimate the remaining lifespan of a unique aged railway bridge, Windsor Railway Bridge in the UK. This research encompassed several steps: analysis of past and current traffic, prediction of future traffic trends, fatigue life analysis, estimation of lifespan consumption and estimation of remaining fatigue life. The FEA results showed that the highest stressed members in the structure were the Arch Stringer and Arch Vertical Bracing. By using finite element method together with cumulative fatigue theory, these members are predicted to have failed in 5 to 7 years' time, depending on the future traffic trends. Under a less conservative design class, some members are shown to have already failed sometime in the 1920s. It is found that a number of major conservative design assumptions were made. The failure mode and mechanism of the aged railway bridge has been highlighted in this paper.

Author keywords: computational mechanics, fatigue, history, mathematical modelling, railway systems, service life, steel structures

List of notation

S – Stress, usually measured in MPa

N (N_i) - Number of cycles until failure

K – Fraction of total cycles consumed

m – Quantity of stresses

n_i - Number of cycles experienced

Glossary of Terms/List of Abbreviation

FEA – Finite Element Analysis

WLR – Windsor Link Railway

LEFM – Linear Elastic Fracture Mechanics

DMU – Diesel Multiple Unit

1. Introduction

The British railway network is made up of a massive amount of infrastructure, including 40,000 railway bridges (Network Rail, 2017). Many of these bridges are considerably aged; some of the earliest were constructed in the 19th century. These bridges were not designed to the comprehensive standards that are in use today. Many of them were not designed with a design service life in mind. Also, these bridges were not taken into account the increased in severe loading conditions and harsh environment that happen nowadays. For new bridges, the more durable and reliable structures have been produced according to the improvement of bridge design standards and construction technologies (Das et.al, 2001). This means that there is a need to perform analyses to estimate the remaining service life of each one. As bridges are subjected to regular cyclic loading, fatigue life assessment is an alternative method to calculate the remaining service life. Various methods are available for estimating the fatigue life of both highway and railway bridges. However, the load conditions are different. The stress-life methods, including nominal stress, hot spot stress, effective notch stress, have been widely used and proved to have more accurate effective than strain-life and fracture mechanics approach (Ye et. al., 2014). A highway bridge is usually subject to low-magnitude; on the other hand, high-frequency traffics are what a railway bridge experiences, vice versa. Highway traffic is as random whereas railway traffic can be quantified to a certain extent. These differences mean that the stress response of a highway bridge is fundamentally different from that of a railway bridge. The overall aim of this study is to demonstrate a modified procedure where the remaining lifespan of a railway bridge can be appropriately estimated. In order to achieve this, the study identifies the past and current rail traffics, obtains the future traffic demand statistics, then analyses fatigue life, estimates lifespan consumption and finally quantifies the remaining life of the ageing railway bridge.

It is important to note that two main structural design concepts have been adopted over a century. The first one is 'permissible stress design' method, which was adopted for the railway bridge design in the past. The recently new on is 'limit states design' concept, which is now existed in Eurocodes. Correlation between two methods requires reliability analysis and appropriate technique to justify the risk of failure and load actions (Kaewunruen et al., 2012; 2014; 2015; 2016). This paper thus highlights a modified procedure to quantify the remaining life of ageing railway bridges. The insight into the quantification will help structural and railway engineers to enable better life cycle management by improving the structural condition prediction and later by retrofitting and maintaining the bridge timely. The better planning for railway bridge maintenance and renewal based on predicted remaining life will improve safety and reliability of the railway networks.

2. Literature Review

There are several approaches to investigate the remaining lifespan of bridges and other structures. A number of these methods have been classified into theoretical and field-based quantifications.

2.1. *Theoretical methods*

Miner (1945) developed a cumulative damage model which is known for its ease of use. The rule works on the presumption that each individual stress a component experiences will consume a portion of its lifespan. The Transportation Research Board (1987) provides a basic processed which estimates the remaining mean life and safe life of a highway bridge. It is a very bare-bones process which can be performed from values given in the document. It is only valid for highway bridges which only see truck usage (Transportation Research Board, 1987). It uses the Fatigue Truck model from the AASHTO Manual for Maintenance Inspection of Bridges (American Association of State Highway and Transportation Officials, 1983). Sieniawska and Sniady (1990) developed a method for estimating the remaining lifespan of a highway bridge based off the traffic it experiences. It takes into account the variable traffic throughout the day by use of a non-stationary Poisson process (Seiniawska and Sniady, 1990). Keller et al. (1995) performed a fatigue life analysis of a bridge using Linear Elastic Fracture Mechanics. Unlike Miner's (1945) method, the method used by Keller et al. (1995) estimates how long it would take a crack to reach critical depth. The method was performed with a bridge that had recorded historical traffic data so the results can be considered accurate (Keller et al., 1995; Kaewunruen and Kimani, 2018). This method was further developed by Rocha and Brühwiler (2012). Their results are in the form of a crack-growth curve which will display how many loading cycles the structure can experience before fracture (Rocha and Brühwiler, 2012). Like the process performed by Keller et al., it assumes Linear Elastic Fracture

Mechanics (LEFM), but admits a possible flaw by stating that the initial defect has to be a minimum depth for LEFM to be valid (Rocha and Brühwiler, 2012). The fatigue assessment code of practice NR/GN/CIV/025 provided the simplified method to estimate the load carrying capacity and fatigue life of aged underbridges and culverts (Network Rail, 2006). Limit state principle was taken into account for wrought iron bridge. Finite element analysis has been recommended as an alternative approach when the bridge does not meet the criteria in NR/GN/CIV/025. Imam et al. (2004) carried out Finite Element Analysis on a plate girder railway bridge and utilised Miner's rule to estimate the fatigue life. Aflatooni et al. (2014) developed a rating system that can be used in bridge networks to display which bridges should be prioritised in terms of maintenance. It does not output any information about the condition of the bridges, only which ones should be prioritized (Aflatooni et al., 2014). Jin-song et al. (2015) developed a process that investigates the effect of corrosion on the lifespan of a pre-stressed concrete bridge in coastal environments. It uses the same Fatigue Truck Model as used by the Transportation Research Board (1987). The nonlinear finite element analysis was considered as an assessment method for U-type wrought iron railway bridges (Canning and Kashani, 2016). However, there were many limitations due to the difficulties details and lacks of information. The elastic buckling and modal analysis were proposed to analyse the behaviour of the cast-iron arch bridges (Zymla, Zielichowski-Haber and Majka, 2016). However, it was not considered the cyclic loads and future traffic demands. You et al. (2017) evaluated a design method for pre-stressed concrete sleepers. It is performed by first principles, as a steel sleeper is a much more simplistic structure than a bridge (You et. al, 2017; Mirza and Kaewunruen, 2018).

2.2. *Methods based off field data*

Keßler et al. (2013) developed a method to update the service life of a reinforced concrete structure based off chloride-induced corrosion using data from potential mapping. The method uses field data to update an existing service life that may have been obtained by other means. As for steel bridges, Ye, Su and Han (2014) summarised the various methods for assessing the fatigue life. This report analyses numerical methods such as those using Miner's rule, and also methods using field data. Lee at al. (2017) displayed a process that estimates the fatigue life of a structure taking into account previous inspection results. This method will output a graph and the remaining fatigue life can be found from it. It can also be performed without inspection data for a bare-bones result.

3. Methodology

3.1. *Fatigue method*

There are three methods available to estimate fatigue life: stress-time, strain-time and LEFM. The stress-time method has been considered due to the following aspects:

1. The bridge of interest is likely to experience cyclic stresses less than yield so the stress-time method is appropriate.
2. The stress-time method is simpler to perform than LEFM. In this study, the stress responses can be obtained using STRAND 7 Finite Element Analysis software (STRAND7, 2018a).

3.2. *Bridge Choice*

The analysis will be performed for the Windsor Railway Bridge in the UK. It was designed by Isambard Kingdom Brunel. It was opened in 1849 and is the world's oldest wrought iron bridge that still in service and sees frequent use (Transport Trust, 2017). It is a bow-and-string bridge, a common type of bridge found in the UK. Other examples of this type of bridge are the Barmouth Bridge in Wales and the Braunstone Gate Bridge in Leicester. As it is also the oldest wrought iron bridge still in active service, there may be a degree of uncertainty to its remaining service life, so estimating the fatigue life will be beneficial to the railway industry. The location of the bridge can be seen in Figure 1.

Figure 1 – Location of the Windsor Railway Bridge (OpenStreetMap, 2018)

Figure 2 - The Windsor Railway Bridge (Poole, 2015)

3.3. *STRAND7 Model*

The Windsor Railway Bridge has a span of 61.5 metres, a rise of 7.6 metres and a deck width of 10.7 metres (Structurae, 2018). The STRAND7 model can be seen in Figure 3.

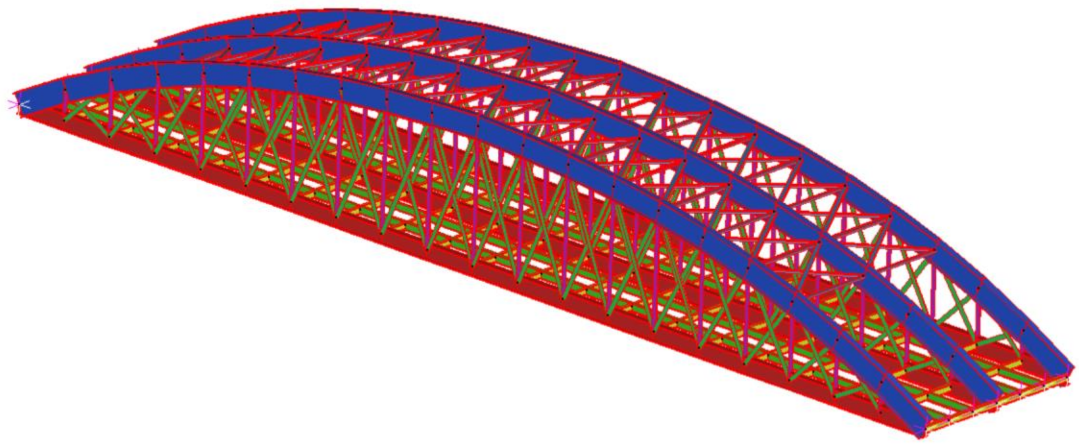


Figure 3 - Windsor Railway Bridge STRAND7 model

Table 1 shows the geometry details for the sections that form the model. Some details on the upgraded section sizes used in the bridge have been estimated. All dimensions are given in metres. The dimensions given in the table refer to the dimensions seen in Figure 4.

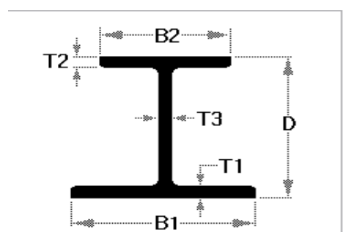


Figure 4 - Section dimension profile (STRAND7, 2018a)

Property	B1	B2	D	T1	T2	T3
Arch stringer	0.4	0.4	1.3	0.03	0.03	0.03
Floor primary stringer	0.4	0.4	0.9	0.05	0.05	0.05
Floor secondary stringer	0.1919	0.1919	0.4634	0.04	0.04	0.04
Floor interconnecting stringer	0.1461	0.1461	0.4	0.05	0.05	0.05
Arch lateral bracing	0.1019	0.1019	0.2572	0.02	0.02	0.02
Vertical arch-to-stringer	0.02	N/A	0.2572	N/A	N/A	N/A
Vertical arch bracing	0.03	N/A	0.2694	N/A	N/A	N/A

Table 1 - Geometry details for STRAND7 model

Although, there is no data available on the actual properties and sections of the Windsor Railway Bridge, the response is needed to be checked to ensure the conservative of structure. In lieu of this, the author will check the vertical deflections with basic design checks. As it has been believed that the structural designer mostly used high factor of safety. Thus, the responses are checked with the allowable limit.

Each stringer is 3075mm. The author will assume a serviceability limit for deflection of L/360. This results in a maximum allowable deflection of 8.54mm. The maximum vertical deflection of the bridge from the model is 3.66mm, so this is acceptable.

The bridge length is 61,500mm. The author will assume a serviceability limit for deflection of L/2600 (European Commission, 2012). The use of such a high factor is to ensure the bridge has adequate stiffness, as to reduce the need for excessive track maintenance (European Commission, 2012). This results in a maximum allowable deflection of 29.29mm. The maximum vertical deflection of the bridge from the model is 3.66mm, so this is acceptable.

3.4. *Finite Element Analysis parameters*

All connections will be taken to be as a rigid connection (except for the hinged supports). Imam et al. (2004) concluded that fully fixed or rigid connections result in a conservative fatigue life. Figure 5 shows the property details that will be assumed for wrought iron.

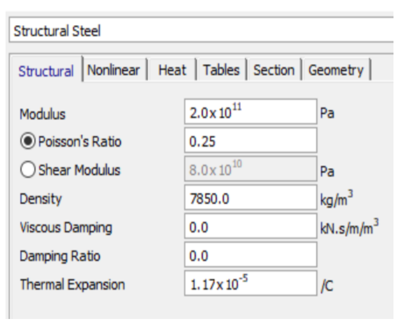


Figure 5 – Assumed properties for wrought iron (STRAND7, 2018a)

4. Past/Current Traffic Analysis (M1)

The traffic over the Windsor Railway Bridge will be calculated by inspecting past and current timetables. The Windsor Railway Bridge lies on the Slough to Windsor branch line. Because it is a branch line it is reasonable to assume that passenger timetables will provide an accurate reflection of all the traffic on the line. The author will presume the bridge has been used exclusively for passenger rail services since construction (no use as a road, bridleway etc.). The traffic model will be created from 1840 to 2020.

The timetable provided by Timetable World shows the services in place for 1949. This shows 42 trains from Slough to Windsor & Eton and 38 in the other direction. In 1965, it shows 39 trains from Slough to Windsor and Eton and 44 in the other direction. In 2017, there are typically 72 trains from Slough to Windsor & Eton and 84 in the other direction (Trainline.com, 2017). This gives a reasonable idea of passenger traffic. No other historical timetables are publicly available for the line. However, the information is reasonably sufficient for analysis.

It will be assumed that passenger trains are running for 51 weeks of the year. All services shown in timetables are assumed to have run every day. In 1949 the Sunday service only had about half the trains of the weekday service. In 1965 and 2017 the number of Sunday services is roughly the same as the weekday service. It will be assumed that the line is only used for passenger services as there are no freight depots on the line. It could be the case that in the past the line was used for a couple of freight services with goods being loaded and unloaded at stations, but the affect from this would be negligible.

From this information the following approximations have been made for the quantity of passenger trains crossing the bridge for the relevant years.

1949	1965	2017
26,520	29,631	55,692

Table 2 - Approximated number of train crossings

This still leaves a large gap from 1849 to 1949 with no information. As there is no data on the amount of rail services for this line before 1949, historical UK passenger data will be used to fill the gap. Figure 6 shows the estimated data on the number of passenger journeys from 1830 to 2010.

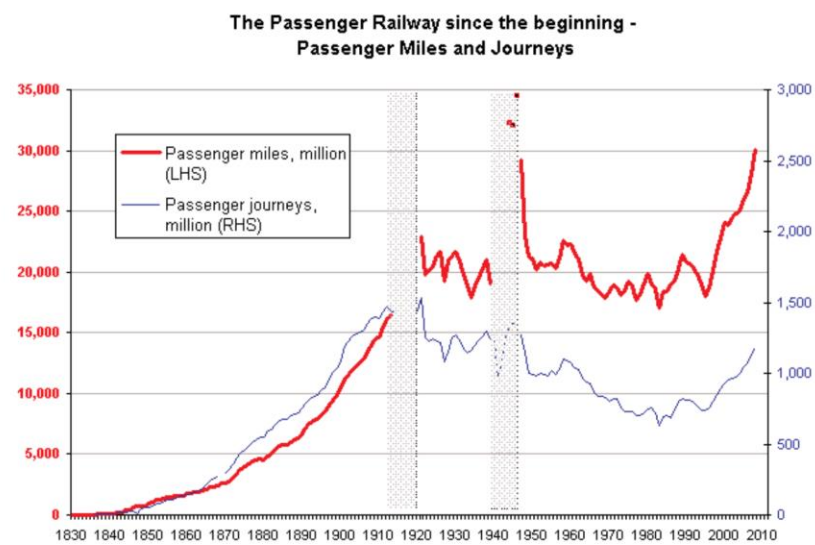


Figure 6 - Passenger Miles and Journeys since 1830 (ATOC, 2008)

General data on the UK railway industry was difficult for ATOC to obtain for years prior to 1947. So assistance was sought from Tim Leunig, a travel historian, to help generate the missing data (ATOC, 2008). This data for pre-1949 will be used to estimate the use of the Windsor Railway Bridge for this time period. From this data the following graph has been established:

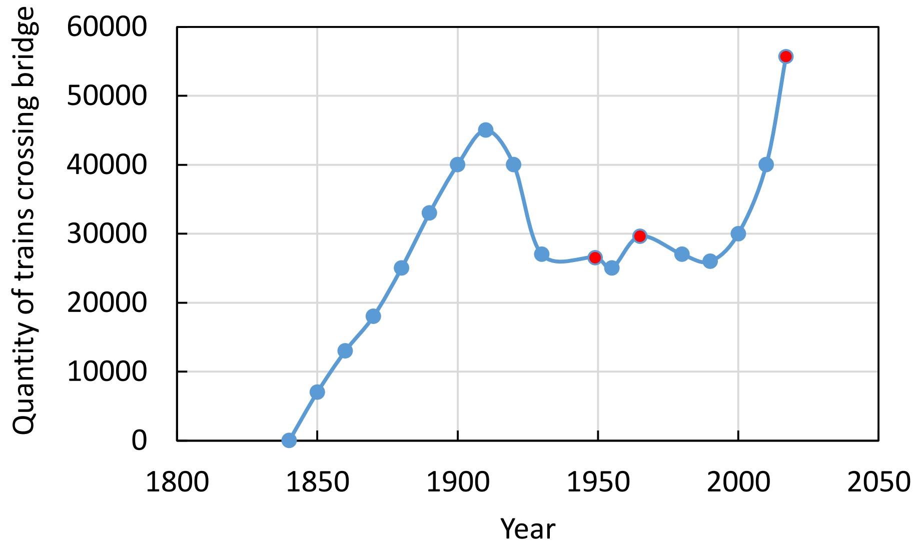


Figure 7 - Estimated yearly train crossings for the Windsor Railway Bridge

The highlighted data points indicate the known information (Data from 1949, 1965 and 2017) .The rest of the graph has been generated in accordance with the passenger data from Figure 6. Based on this data, the estimated cumulative train crossings since 1840 is 4,936,185.

5. Future Traffic Prediction

There is currently a project, called Windsor Link Railway (WLR), being surveyed and planned by independent consortium. The part of planned project is to link new railway line between Slough and Staines, running through Windsor. According to the survey made by Copper Consultancy in 2018, it was shown that the businesses and commuters in and around Windsor proposed to have this service as there will be more benefits by creating more journeys and providing new jobs. It is a significant project and will likely result in an increase in the amount of traffic. According to the planning programme, it is set to open in 2022 (Windsor Link Railway, 2017). It is assumed that the local services between Slough and Windor & Eton will remain but there will be an additional 60 trains each direction per day, which are using the line as part of long distance journeys (This is based off 5 trains per hour for 12 hours). This would result in 98,532 passenger trains crossing the bridge per year. These additional services will be assumed to be the same rolling stock as the local services (Class 165 DMU). Two analyses will be performed: one assuming the traffic increases as a result of the WLR, and one assuming the traffic stays as it is (as of 2018).

6. Finite Element Analysis

6.1. *Load cases*

Two load cases will be used, based on the different weights of various vehicles that may have used the bridge. The different load cases will represent the change in the type of traffic on the route. The load cases have been used as they can reasonably give an accurate representation of traffic that uses/used the line. Only two load cases will be used for simplicity. For a more accurate result more load cases should be used as it is likely that more than two types of rolling stock have been on the line.

The branch line is currently operated by First Great Western who utilise Class 165 DMUs (Angel Trains, 2017). The 165 is available as a 2-car unit or a 3-car unit (Angel Trains, 2017; Setsobhonkul et al., 2017). To obtain a conservative fatigue life estimation, it is assumed that all train formations have been 3-car units. Each car has slightly different specifications resulting in subtle weight differences, but reasonably each car weighs 38 tonnes (Angel Trains, 2017). This will be the load case for 1950-present day. For the purposes of simplicity each axle will be assumed to act as a single point load. Figure 8 shows an illustration of the point loads for the Class 165.

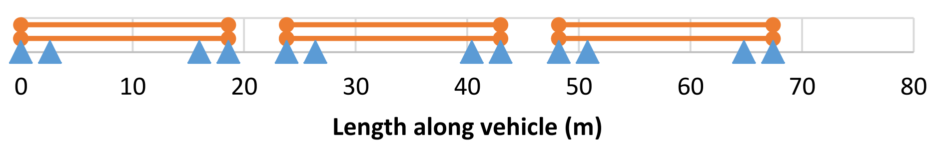


Figure 8 - Class 165 point loads

It can be assumed that for 1840-1950 the line saw primarily passenger usage. One of the main differences between this time period and modern times is the method of locomotion. Nowadays DMUs are used whilst in the early 20th century steam locomotives were used. Generally, each passenger carriage weighs around 20 tonnes, and that each train was made up of one locomotive and three carriages. The locomotive will be assumed to be a LMS Hughes Crab unit, which weigh 66 tonnes (Brown, 2014). They have a 2-6-0 axle configuration, and it can be assumed that 60 tonnes acts on the primary 6 wheels and the remaining 6 acts on the front 2 wheels. The coal car will be assumed to weigh 5 tonnes and that load is carried by 6 wheels. Figure 9 shows an illustration of the point loads for the locomotive passenger unit.

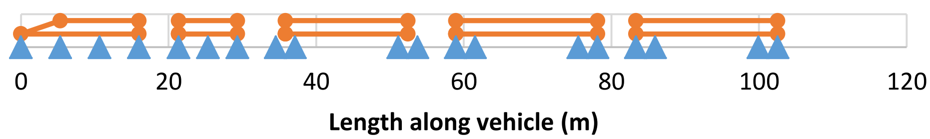


Figure 9 - Locomotive passenger unit point loads

6.2. *Analysis Method*

There are two methods that can be used for Finite Element Analysis: static and dynamic. Static analysis is the most basic form of the analysis, the loads are applied, remain constant and time independent. With dynamic analysis the loads can be time-dependent taking strain rate into account. Considering the scope of this study, quasi-static analysis using load factors have been carried out to identify the design actions and responses. This is because:

1. Train movement is constant and usually does not vary over time. Therefore no forces are imposed on the bridge as a result of acceleration/deceleration.
2. The bridge carries tangent track, so no lateral centripetal forces will be imposed on the bridge.
3. The track support has filtered out most intense dynamic load content via resilient layers. The load action on the bridge is rather quasi-static.

The first stage of the analysis is to perform a preliminary static analysis on the bridge without the applied loads. This will calculate the base deformations and stresses under dead load. This state is then saved and used as a 'zero' point for analysis of the applied loads. If this process were not performed the model would experience an initial sudden loading due to gravity. This can result in the solution being 'dominated by transient behaviour as the bridge bounces due to the gravity load' (STRAND7, 2018b).

For each load cases the analysis was performed twenty times. Each time the forces imposed by the vehicles are stepped over the bridge in increments of L/20, with L being the length of the bridge.

6.3. *Results*

The table below shows the members of each group that experience the highest stresses:

Beam #	Description	Passenger loco Max Stress (MPa)	Class 165 Max Stress (MPa)
78	Arch Stringer	-30.61	-24.38
81	Primary Floor Stringer	-8.56	-6.78
141	Secondary Floor Stringer	-0.52	-0.46
210	Interconnecting Floor Stringer	0.00	0.18
385	Arch Lateral Bracing	5.99	4.64
449	Vertical Arch to Stringer	6.44	6.53
578	Arch Vertical Bracing	-29.31	-24.14

Table 3 - Table of members experiencing the highest stresses

Table 3 shows which members are likely to fail first due to fatigue loading, and thus the point at which the bridge can be considered to be unsafe for use. The Arch Stringer and the Arch Vertical Bracing experience the highest stresses (members 78 and 578 respectively), so these two members will be analysed further. Figure 10 displays the location of members 78 and 578. The rest of the members have maximum stresses which are much lower than those of the arch stringer and vertical bracing, so it is not practical to do the full analyses for these members.

The Arch Stringer (member 78) will fail first, resulting in a severely increased loading on the nearby vertical bracings. When the Vertical Bracing (member 578) fails this will place increased strain on the diagonal bracings.

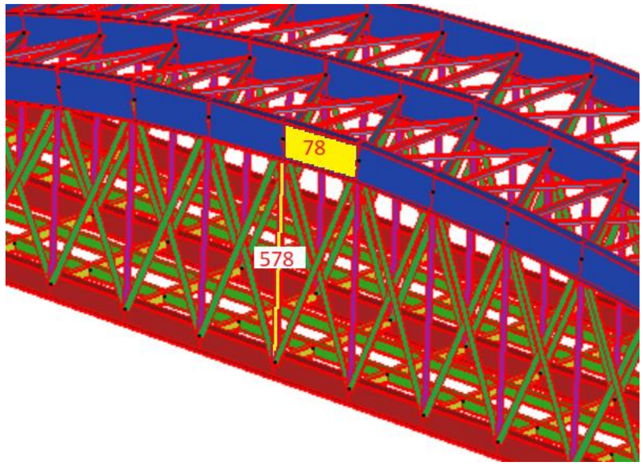
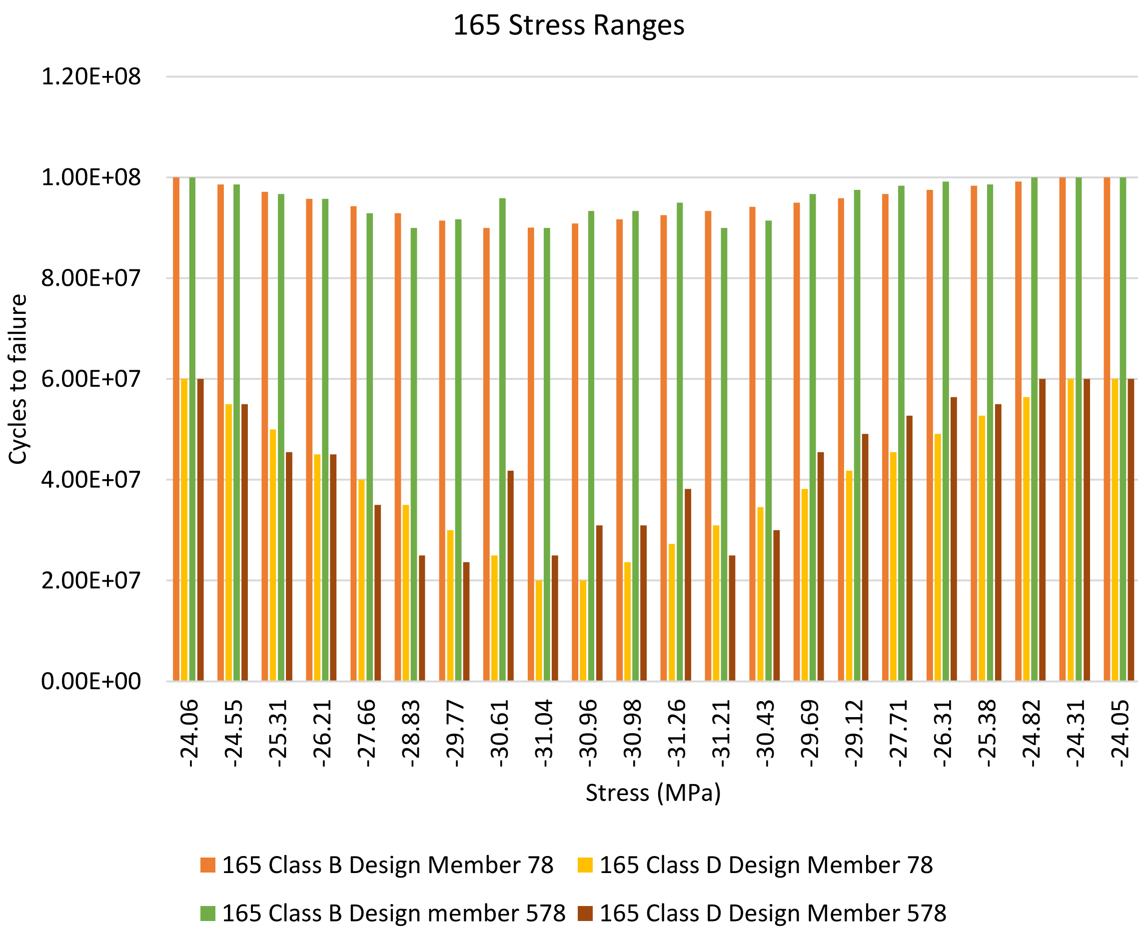
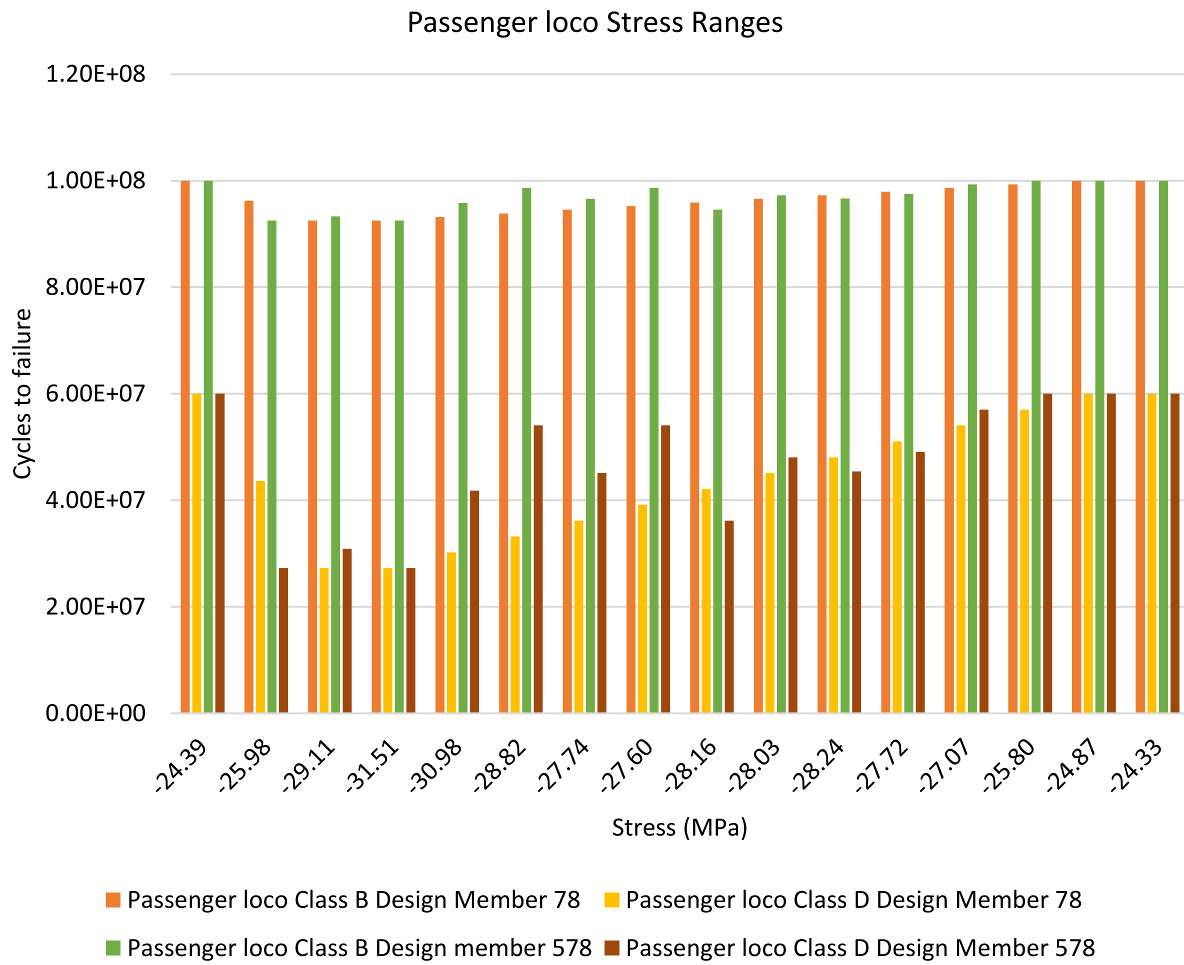


Figure 10 - Location of members 78 and 578



a) Class 165 stress ranges for members 78 and 578



b) Passenger loco stress ranges for members 78 and 578

Figure 11 - Critical states of stresses of members 78 and 578

The stress ranges for the Class 165 DUM for members 78 and 578, and for passenger locomotive can be found in Figure 11.

7. Lifespan consumption and remaining fatigue life

BS 5400 is a standard for steel, concrete and composite bridges (British Standards Institute, 1980). It defines nine S-N classes: B, C, S, D, E, F, F2, G and W. Class B gives the best performance (i.e. highest number of cycles to failure) while class W gives the lowest. The S-N chart shows the number of cycles to failure for the *mean* stress for each class. To obtain the number of cycles for the *design* stress, the curve used should be two standard deviations below the one being investigated. A standard deviation is a single S-N slope. This means that the design class B will be read off the graph as class D, and design class D will be read off the graph as class F. The different classes can be used to represent different characteristics of the bridge connections. For instance Imam et al. (2004) used Class B to represent the connections having a low clamp force, while class D is used to represent connections with a high clamp force, such as spliced connections. This project will use the same two classes and compare the results.

Components in the bridge will experience a range of stresses. Each individual stress will consume a fraction of the components lifespan. The S-N curves will be used to convert the stress ranges to the number of cycles to failure. These can then be used with Miner's Rule (1945) to calculate the total lifespan consumption of the component, as shown in Equation 1.

$$K = \sum_{i=1}^{m} \frac{n_i}{N_i} \quad (1)$$

Where K is the fraction of cycles consumed, m is the quantity of stresses, n_i is the number of cycles experienced and N_i is the cycles to failure.

A value of 0 for K indicates no damage and a value of 1.0 indicates failure (i.e. the component has gone beyond its Ultimate Limit State). The remaining fatigue life can be estimated by simply increasing the number of cycles the bridge has experienced (using future traffic estimations) until K exceeds 1.0.

Based off the data from Figure 10 and Figure 11, Miner's rule has been used to estimate the lifespan consumption and remaining fatigue life for each Design Class and each member. These results can be seen in Table 4. As mentioned previously the estimated lifespan is calculated twice: once for predicting values and once assuming the future traffic will remain as it is now. The future traffic model assumes that today's traffic trends remain in place until 2022 (WLR opening) and then the traffic increases to the authors prediction.

Design Class	B	D

Lifespan consumed (%)	**Member 78**	96	237
	Member 578	95	222
Estimated lifespan (authors estimation) (yrs)	**Member 78**	5.57	FAILED
	Member 578	5.80	FAILED
Estimated lifespan (current traffic trend) (yrs)	**Member 78**	7.05	FAILED
	Member 578	7.08	FAILED

Table 4 - Lifespan estimations for members 78 and 578

In fact, due to the irregularities of either wheel or rail, the load acting on track is amplified causing a high magnitude dynamic force in a very short duration (Ngamkhanong et al. 2018). A Dynamic Amplification Factor should be used to represent this additional load. However, upon looking at the results in Table 4, it becomes obvious that the use of a Dynamic Amplification Factor will simply result in both members exceeding their total lifespan and failing. Therefore the author will not investigate the effect of a Dynamic Amplification Factor as nothing will be gained from the results. Imam et al. (2004) investigated Dynamic Amplification Factors and concluded that whilst they may have a significant effect on the results, it is a very simplified estimation as in practice individual stress ranges would be associated with different factors.

Further to the results above, the author has generated the cumulative damage experienced by the two members for the two design cases. Under class D Design assumptions both members are predicted to have failed sometime in the 1920s. Figure 12 shows the cumulative damage. It is clear that the vertical members (member 78 and 578) will demonstrate a sign of failure first.

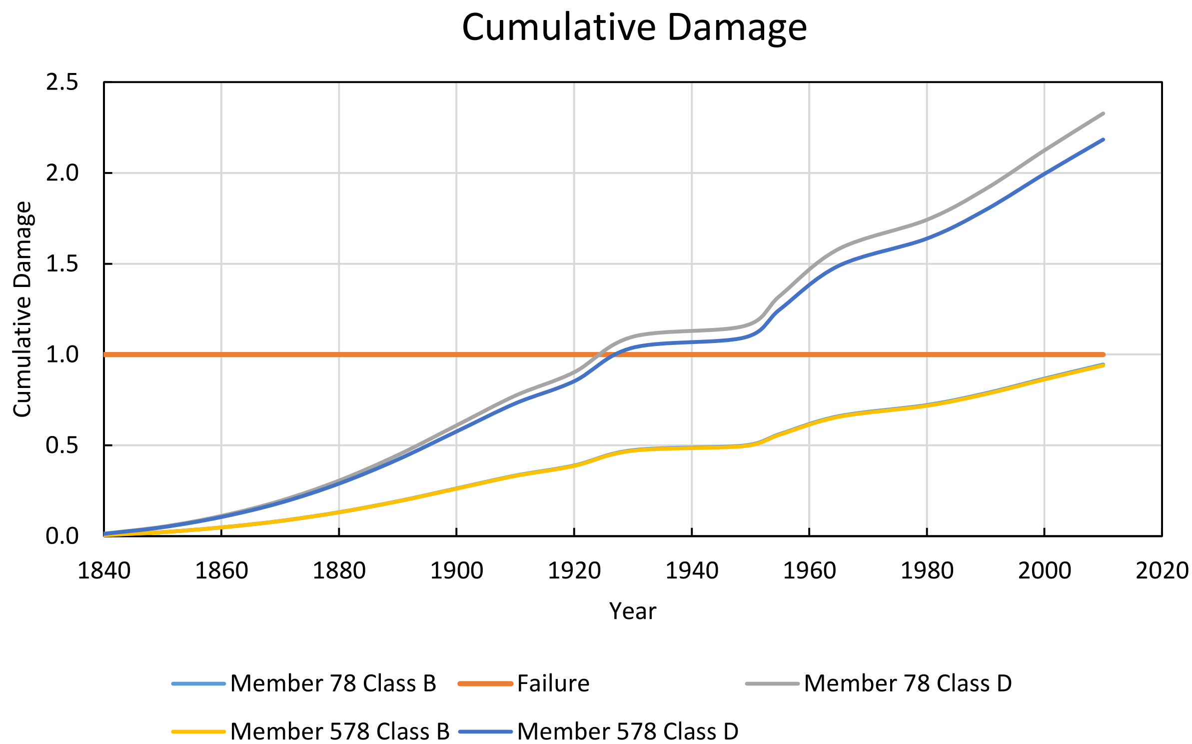


Figure 12 - Cumulative damage of bridge components

8. Discussion

The most noticeable result from the analysis is that under Class D Design assumptions both members are shown to have already failed, and are predicted to have failed much earlier. The potential implications are that the critical members are maintained regularly to extend the bridges lifespan. It is also evident that:

- The latest maintenance performed on the bridge was from October 2013 to June 2014 (Carter, 2014). The bulletin stated that the maintenance involved 'steel work repair'. Depending on the scale of the works this repair work could have a significant impact on the remaining fatigue life of the bridge, but that falls outside the scope of this document. This is the only documented repair work that the author was able to locate. It is very likely that additional repair work has been performed on the bridge during its life.

- For both of the future traffic models the highest stressed members on the bridge are both expected to fail within 10 years. The latest data-point on the traffic model is from 2017 so the results imply that both of the investigated members will fail after 2022 according to the traffic estimation, and after 2024 assuming current traffic. The latter result is likely to be more accurate as if the traffic over the bridge were to increase, it would do so in a gradual manner as extra services are slowly introduced. This insight is essential to help rail engineers maintain and renew the bridge accordingly.
- It is noted that when Imam et al. performed their FEA on a basic plate girder bridge the highest stressed member on their model was expected to fail in 89 years due to fatigue, and had consumed around 35% of its total lifespan (2004). However the bridge was assumed to be put into service in 1900 and the traffic analysis goes up to 2004. The Windsor Railway Bridge was opened in 1849 and the traffic analysis was performed up to 2017. Imam et al. had not made their traffic model available so any direct comparisons between their data and the authors cannot be relied upon. Although as the results seem to be in the same magnitude it could suggest that there may be some level of accuracy, but with the assumptions that were made it cannot be expected for certainty.

9. Conclusion

This study presents a new investigation into the remaining life of aged railway bridges. With over 40,000 ageing railway bridges in the UK only, this paper will provide a pathway for better operation and maintenance of railway bridges. Winsor railway bridge has been chosen for the investigation since the actual inspection data of the bridge can be accessed. Finite element analysis has been used to yield the action and responses for remaining life prediction using Milner’s fatigue theory. The FEA results showed that the highest stressed members in the structure were the Arch Stringer and Arch Vertical Bracing. Based on the increasing operational trend, these members are predicted to have failed in 5 to 7 years’ time, depending on the future traffic patterns. It is also found that maintenance details have not been adequately recorded. Note that some members should have already failed sometime in the 1920s without renewal or maintenance. However, due to the limited structural details and inspection data, the conservative assumptions are taken into account. This study highlights the remaining life prediction that could enable better planning for railway bridge maintenance and renewal. The established algorithm can be used to estimate the effect of future demand on the life span of existing bridges. The insight can be very beneficial to condition-based and predictive maintenance strategy for railway bridges.

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Figure Captions

Figure 1 – Location of the Windsor Railway Bridge (OpenStreetMap, 2018)

Figure 2 - The Windsor Railway Bridge (Poole, 2015)

Figure 3 - Windsor Railway Bridge STRAND7 model

Figure 4 - Section dimension profile

Figure 5 – Assumed properties for wrought iron

Figure 6 - Passenger Miles and Journeys since 1830

Figure 7 - Estimated yearly train crossings for the Windsor Railway Bridge

Figure 8 - Class 165 point loads

Figure 9 - Locomotive passenger unit point loads

Figure 10 - Location of members 78 and 578

Figure 11 – Critical states of stresses of members 78 and 578

Figure 12 – Cumulative damage of bridge components