Factors Influencing the Design Life of Old Steel Bridges

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Abstract. When deciding about the specific design life for bridge structures, care should be taken to ensure that the structure fulfils all the fundamental requirements of structural reliability in terms of robustness, safety and serviceability in order to achieve the service life of the bridge. Load and material properties, cross-section and system geometry are the basic variables or parameters that are being used when considering the design life of bridge structures. The design life of bridge structures as specified in the Eurocodes and British Standards is 100 and 120 years respectively while in AASHTO-LRFD it is for 75 years. However, in all these codes of practice, there are no specifications or provision or guidelines related to sustaining the service life of bridge structures (Bartholomew, 2007 & 2009). This is because the service life depends on the durability of the structure which is heavily influenced by several factors such as fatigue, corrosion and changes in superimposed loads. Therefore, with an increase in traffic loading together with climate changes, the demand to ensure service life is very acute because of the importance of bridge structures in the infrastructure network, which is especially true for long span bridges. Also, with different load applications of heavy good vehicles (HGVs) occurring during the design life, this will affect the structural integrity of old steel bridges. For example, old bridges in many countries including Malaysia were designed using British Standard compliances. Since the Eurocodes have been widely practising nowadays, the estimation of loading applied might be different with the old code of practice. Plus, with the traffic increasing every year without any control, the maintenance for the old bridges especially should be more frequent as these bridges may not have benefited to the remedial measures to improve the fatigue performances. Therefore, the actual service life may not reach the expected service life as the actual service life depends on the exposure condition of the structure, quality of materials, design and construction and also the level of maintenance performance. In addition, with the increase in traffic load and frequency, this could seriously jeopardise the integrity of old bridges to meet their actual service life. This paper discusses the issue concerning the design and remaining service life estimation used when designing and appraising steel bridges.

1. Instructions

Bridges in many countries including Malaysia were designed using British Standard compliances. There are more than 7000 bridges in Malaysia excluding Borneo island. Nearly 10 percent of these bridges used steel as the primary material. This total numbers of bridges do not include modern, long-span bridges, such as the Penang Bridges, Langkawi Sky Bridges, and the Muar Second Bridge. For a
steel bridge, the demand to ensure its service life is achieved is very acute especially for long-span bridges due to their importance in the infrastructure network. This is because steel bridges are prone to fatigue damage due to cyclic loading caused by moving traffic loading. According to Miki (2004), there are a large number of steel bridges that have been suffering fatigue cracking and several of them have collapsed due to fatigue damage since 1960. A recent incident related to the Fourth Road Bridge in Scotland where it was closed to traffic in December 2015 because of a sudden element failure due to fatigue. For all bridges, the actual service life may not reach the expected design life. This is because the actual life depends on the exposure condition of the structure, the quality of the materials used, the design and construction and also the level of maintenance received. On the contrary, the expected service life is based on the owner’s desires and expectations through the design life. As specified in the British Standards and Eurocodes, the design life for bridges is 120 and 100 years respectively while for AASHTO – LFRD it is only 75 years. However, where the number of years has been set as ‘mandatory’ for bridges to fulfil their service life, this does not necessarily mean that the structure is able to complete its expected service life especially with the status of the current highway system which is both overloaded and overcrowded?

1.1. Fatigue phenomena
Traffic volume can expose individual members and connections in structures to fatigue damage. According to Gabler (2015), increasing traffic volume is one of the major factors contributing to fatigue crack initiation. This phenomenon can deteriorate the performance of a structure as it may result in unexpected failure. Defects or damage due to fatigue failure may occur and develop progressively in the structure but it may not be detected visually. Fatigue usually causes a very fine crack to develop in the steel member. This initial crack is then followed by crack propagation leading to complete fracture of the member after the structure undergoes a sufficient number of critical load cycles. These critical load cycles for a bridge may occur from traffic and/or wind loading. Also, a breakdown of the coating system used for the structural system will increase the damage level. This increases the phase propagation of any cracks. A deterioration mechanism such as chloride and carbonation attack can dissipate into the structure and induces corrosion to the structure. In addition, if corrosive water leaks into the structure, fatigue corrosion may also occur and this speeds up crack propagation. For example, leaks at the cable anchorage points in a cable-stayed bridge during the grouting process contribute to premature fatigue and fatigue corrosion due to the infiltration of moisture (Tabatabai et al., 1995).

1.2. Codes of practice used
Starting from March 2010, it was recommended that all newly-procured structural design work in the UK complied with the Eurocodes (ADEPT, 2010). Therefore, the assessment of fatigue for bridges should also be based on the guidance given in the Eurocodes. However, the Eurocodes use a slightly different approach for the applied loading compared with the British Standard. Furthermore, the increase in traffic loading each year could affect the loading application that has been used in designing bridges using British Standards. Also, both the British Standards and the Eurocodes do not state clearly the traffic load increments to use which results in the designer using their own experience and knowledge to overcome this shortcoming. However, the Eurocodes introduce five different types of fatigue load namely, fatigue load model 1 (FLM 1) to fatigue load model 5 (FLM 5) which gives the opportunity for engineers to design the bridge using an accurate traffic model (BS EN 1991-2). These fatigue load models are used under different conditions and consist of a set of equivalent lorries that simulate the traffic, producing fatigue damage which is equivalent to that due to the actual traffic. Engineers prefer to use FLM 3 in bridge design because it reduces some of the calculation procedure, yet it is accompanied with some shortcomings (Muhammad Khairussaleh, 2016). In addition, the concept and application of this practice are easy and nearly similar to the standard fatigue vehicle (SFV) given in the British Standard BS5400 Part 10. On the contrary, two other types of the fatigue load models given in the Eurocodes; FLM 4 and FLM 5 provide the more realistic condition of typical
traffic. This is because the determination of the stress history can be achieved with the application of these fatigue load models as they simulate the actual traffic flow or by taking the exact data from the traffic movement on the bridge (in situ traffic). The complex stress ranges can be generated which are needed in the fatigue assessment through the use of the appropriate S-N curve. Therefore, FLM 4 and FLM 5 are the most accurate basis for predicting fatigue life.

2. Estimation fatigue damage using different codes of practice

Each code of practice uses its own distinctive heavy good vehicle (HGV) and conditions of traffic flow. There are two types of traffic load conditions recommended in the Eurocodes, FLM 4, namely traffic alone and traffic in a convoy. These types of load model simulate traffic where each lorry is expected to transit both alone and in a convoy along the bridge. In addition, this model classifies traffic loading into three types of traffic namely: long distance, medium distance and local traffic (BS EN 1991-2). Although there is no specific explanation in the Eurocodes but the Institution of Highway & Transportation, Department of Transport (1987) proposed the magnitude of these traffic loading should be chosen based on the design needs on the bridge one by one. The total load of the smallest lorry allowed to cross in the traffic alone condition is the lorry with two axles weighing 200kN (20 tonnes) while the total load of the heaviest lorry with a set of five equivalent axle loads is 490kN (49 tonnes). For the traffic in convoy condition, five equivalent lorries should run in convoy with a 40m spacing between each vehicle. This causes a total load of a set equivalent axle loads running in convoy to be equal to 1840kN (184 tonnes). This is quite a large traffic load being applied on the bridge for the fatigue assessment. In the AASHTO-LFRD code, the loading for the fatigue assessment is based on a single design truck with a total axle load of 320.3kN (72kip). This total axle load is 169.7kN lower compared to the biggest equivalent lorry load used with the traffic alone condition given in the Eurocodes and nearly five sixth lower than the total equivalent axle loads that are used with the traffic convoy condition in the Eurocodes. Table 1 shows the summary of the critical loading of HGV applied on a bridge from different codes of practice used for fatigue appraisal.

| Traffic Condition | EUROCODES FLM 4 | EUROCODES FLM 3 | BS 5400 | AASHTO LFRD |
|-------------------|------------------|------------------|---------|-------------|
| Traffic Alone     | 490              | 480              | 320     | 320.3       |
| Traffic in Convoy | 1840             | 516              | -       | -           |

Based on Table 1, it is evident that for the traffic alone model the Eurocodes provide nearly 53% or 1.5 times greater load compared with the AASHTO load value. Furthermore, in the convoy traffic condition, the load in Eurocodes is 5.75 times greater compared to the AASHTO traffic alone condition. However, BS 5400 provides the same magnitude of axle load as that given by AASHTO. Therefore, it is expected that the stresses that will be generated from AASHTO and BS 5400 will be substantially lower and the potential for fatigue damage to occur in the vicinity that is vulnerable to fatigue using these two codes of practice will be less than that expected from using the Eurocodes loading. It is important to remember that each code of practice also recommended a different service design life. That is why in order to achieve this sustained fatigue design working life up to 120 years, the Eurocodes provide a more stringent procedure compared to that adopted by the AASHTO code.

3. Potential factors affecting design life

Fatigue actions are determined according to the requirements of the fatigue assessment based on the statistical evaluation presented in S-N curves but fatigue appraisals are different from the ultimate and serviceability limits state verifications (Bijlaard, 2007). Therefore, any fatigue crack that develops
during the service life does not necessarily mean the end of the service life for the structure because the crack would be repaired (Bijlaard, 2007). This is to avoid any fatigue crack introducing more severe notch conditions that could shorten the service life of the structure even further.

3.1. Residual stress in the welded structure
The problems with welded structures are the presence of high tensile residual stresses occurring as a result of the welding process. This is because with high-stress concentrations forming in the vicinity of the connections and the occurrence of tensile residual stresses influences the fatigue life of welded structures. Therefore, the possibility of fatigue damage occurring in a welded structure is high even with an applied external compressive stress. Steel bridges generally contain a large number of welds and consequently high tensile residual stresses which may govern the fatigue life of the bridge. Although studies have been carried out to investigate ways of improving fatigue performance for weld connections (Lee et al., 2012; Wang et al., 2012; Fu et al., 2014; Zhao et al., 2016) old steel bridges can still be exposed to these high tensile residual stress fields. This is because only relatively modern bridges have benefited from these techniques used to improve the fatigue performance of the connections but not for old bridges. Therefore, steel bridges which have achieved a service life of more than 50 years should have a more frequent inspection for fatigue damage.

3.2. Traffic flow increasing and type of traffic characteristic
In December 2015, the Fourth Road Bridge in Scotland had major repair work undertaken after a crack was found in one of the support brackets for the truss carrying the bridge carriageway. The 20mm wide crack was not found during the last regular six months of maintenance regime work. Fatigue and overloading occurring over the bridge’s 51-year lifespan have been reported to be the reason for the crack problem (BBC, 2015). According to Russell (2016), this structure carries 78,000 vehicles per day with 9% of them are heavy goods vehicles (HGVs). In addition, the occurrence of the fatigue crack was identified to be partly due to the increasing traffic load, which was not fully anticipated in the initial design. Most codes of practice do not indicate the level of traffic increment expected for bridge design. Gabler (2015) also mentioned that the weight of the average truck, loaded to its full capacity, has also increased steadily along with the general increase in traffic density. Therefore, both of these events will adversely affect the strength of the bridge and will generate bigger stress ranges at the critical or vulnerable areas prone to fatigue damage. Hence, old steel bridges are also exposed to the same problems if there is any traffic load increment crossing over these bridges. Thus, in order to determine the condition of the structural elements prone to fatigue, cycle loading analysis has been conducted for a cable anchorage block on a cable-stayed bridge by comparing the traffic characteristic applied as shown in Figure 1.

![Figure 1: A 3D model of the anchorage block with the pressure applied to the bearing plate.](image)

Table 2 present the total fatigue damage that could occur on the cable anchorage block with different types of traffic characteristics.
Table 2: Total damage occurring at the cable anchorage block with a different type of traffic characteristic

| Type of traffic characteristic | Top Gusset (a) | Bottom Gusset (b) |
|--------------------------------|---------------|------------------|
| Local Traffic                  | 0.325         | 0.035            |
| Medium Distance                | 0.847         | 0.087            |
| Long Distance                  | 1.270         | 0.113            |

From the top gusset and bottom gusset of a particular cable anchorage block have different fatigue damage with reference to the type of traffic characteristic. Based on Miner’s rule, the damage accumulation ($D_{\text{max}}$) should not exceed 1.0 (BS EN 1993-1-9; Palmgren – Miners’ Rule). This value means that with proper maintenance, the bridge can sustain to its design service life. However, the International Institute of Welding (IIW, 2008) and BS 7608 (2014) recommended the fatigue damage accumulation should not exceed 0.5 due to a large number of uncertainties present in the fatigue phenomenon. According to Nussbaumer et al. (2011), the use of $D_{\text{max}} = 0.5$ is needed for verifications under both proportional and non-proportional multi-axial stress cases. Therefore, based on this fatigue guidance rule, it reveals that both gussets will not have an expected fatigue failure during their service design life but only for local traffic and medium distance traffic. However, the top gusset will experience fatigue failure if long distance is used as the traffic characteristic in the analysis.

3.3 The hot-spot stress approach

Using the hot-spot stress method (Maddox, 2011) for steel bridges for fatigue evaluation has become important nowadays. This is because the actual stress value used in estimating fatigue damage can be difficult to determine. Note that fatigue is a local failure and assessing the local stress concentrations occurring at the critical location is very critical. It is very important to determine high-stress intensities resulting at any particular connection detail. Therefore, the concept of a hot-spot stress has been introduced in order to provide an appropriate stress for fatigue assessment. This approach has been used successfully for welded details especially in offshore structures but it is still limited for bridge design engineering (Bhargava, 2009). Determining the hot-spot stress cannot be directly acquired from a finite element analysis. The derivation and guidance on how to determine the hot-spot stress have been outlined in the codes of practice (Niemi, 1995; Niemi et al., 2006; DNV - RP-C203; 2011; IIW, 2008; BS 7608, 2014). Basically, two stresses values located at a defined distance from the critical point will be used to extrapolate the hot-spot stress. This extrapolation is based on the finite element mesh density, element type, size, and reference point to the highest node stress concentration all of which has been recommended in the codes of practice. Even though this method has a high accuracy in assessing fatigue, the use of this method by bridge designers is still not widely adopted.

4. Conclusion

Bridges in Germany have to accommodate increasing loads due to the large increase in truck traffic and this was one of the primary factors in the initiation of a crack in the main girder of the Leverkusen Bridge (Gabler, 2015). The same factor was the cause of fatigue damage to the Fourth Road Bridge in Scotland causing it to close in December 2015 for a major repair. This increase in traffic density and vehicle loads which recently caused problems in two different countries alarming engineers as a significant increase in traffic volume over the past 50 years has not been properly considered in bridge design. This enormous growth in both traffic volume and weight will influence greatly the design life of old bridges. Moreover, with the different codes that have been used in the past for design, have resulted in different values for bridge service life. In addition, with the general lack of detailed information particularly regarding welded details on old bridges and the type of traffic characteristic used during design, old steel bridges are likely to have a high exposure to fatigue failure which will consequently be reflected in a shortened design life. Therefore, the maintenance schedule for old
bridges should be clear in identifying critical details and these need to be appraised on a frequent basis to ensure a continuity structural integrity of the structures.

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