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OCENA ZMĘCZENIA 100-LETNIEGO STALOWEGO MOSTU KOLEJOWEGO W UJĘCIU NIEZAWODNOŚCI KONSTRUKCJI

FATIGUE RELIABILITY ASSESSMENT OF 100 YEAR OLD STEEL RAILWAY BRIDGE

Streszczenie Mosty kolejowe są podatne na uszkodzenia materiału w skutek zmęczenia w okresie ich użytkowania. Są one narażone na duże obciążenia cyklicznego wywołane przejeżdżającymi pociągami. W stanie granicznym zmęczenia, elementy i połączenia mogą ulec awarii, nawet gdy poziom naprężeń jest niższy od naprężeń dopuszczalnych. Na ocenę zmęczenia składa się wiele parametrów takich jak identyfikacja krytycznych komponentów, historia obciążeń, zakres naprężeń, liczba cykli, stopień degradacji i wielu innych. Większość z tych parametrów ma charakter losowy, dlatego też zalecane jest podejście probabilistyczne do dokładnego oszacowania trwałości zmęczeniowej. Artykuł ten przedstawia analizę niezawodnościową typowego mostu stalowego z dźwigarami blachownicowymi, nitowanymi wykonaną na podstawie wyników uzyskanych z metody elementów skończonych (MES). Na podstawie historii obciążenia i zakładanego poziomu bezpieczeństwa oszacowano przewidywany okres użytkowania dla każdego krytycznego elementu i połączenia.

Abstract Railway bridges are vulnerable to fatigue damage during their service. They are exposed to cyclic high stresses due to the moving load. In the fatigue limit state, components and connection may lead to failure even when the stress level is lower than the allowable stresses. Fatigue evaluation consist many parameters such as identification of critical components, recent and past load history, stress range, number of cycles, degree of the deterioration and many others. Most of these parameters are random in nature; therefore, the probabilistic approach is recommended for accurate estimation of remaining fatigue life. In this study the through-plate girder, riveted railway bridge is analysed using results from Finite Element Method (FEM). Based on the load history and assumed safety level the predicted years of service is estimated for each critical component and connection.

1. Introduction

Railway bridges constitute a vital part of the transportation infrastructure system and they require special attention to provide safe and economical service. Consequences of stoppage of railway traffic can be severe, including impacts on the regional or even national economy.

Based on the characteristics of railway bridges in USA, over 60% of railway bridges were constructed before 1950. Those bridges are over 60 years old and they require special attention. According to data provided by Union Pacific about 50% of railway bridges are steel structures, about 40% are short bridges with a total length less than 50ft, and about 75% of railway bridges have span length less than 50ft, [1].

There is a growing need for efficient methods to evaluate the safety reserve in the railway bridges. The current methods are based on the deterministic approach and emperical equations.

The parameters which affect safety of railway bridges are random in nature. Therefore, probabilistic approach are more accurate for estimation of remaining fatigue life.

The objective of this study is to present a reliability model for railway bridges demonstrated on typical through-plate girder structure. The research work is based on the identification of the basic load and resistance parameters and modeling of structural behavior. Based on structural analysis performed using FEM programs, [2], the live load effect for the bridge components was developed by Rakoczy and Nowak [3]. The calculation of effective stress and number of cycles are calculated. The statistical parameters of fatigue resistance is based on the previous study, [4]. Finally, the calculated reliability index for individual components and connections are presented and the predicted years of service is estimated.

2. Structural analysis of typical railway bridge

The investigated bridge is a through-plate girder, riveted, open deck railway bridge. It was designed according to AREA, [5], and built in 1894. The structure is located on the main railway line connecting Bangkok to the north and northeast of Thailand, [6]. The overall inspection shows that the structure is in good condition with minor loss of sections due to corrosion. The bridge has a one simply supported span which is 32 ft. 9 in. (10 m) long with the floor system presented in Figures 1 and 2.

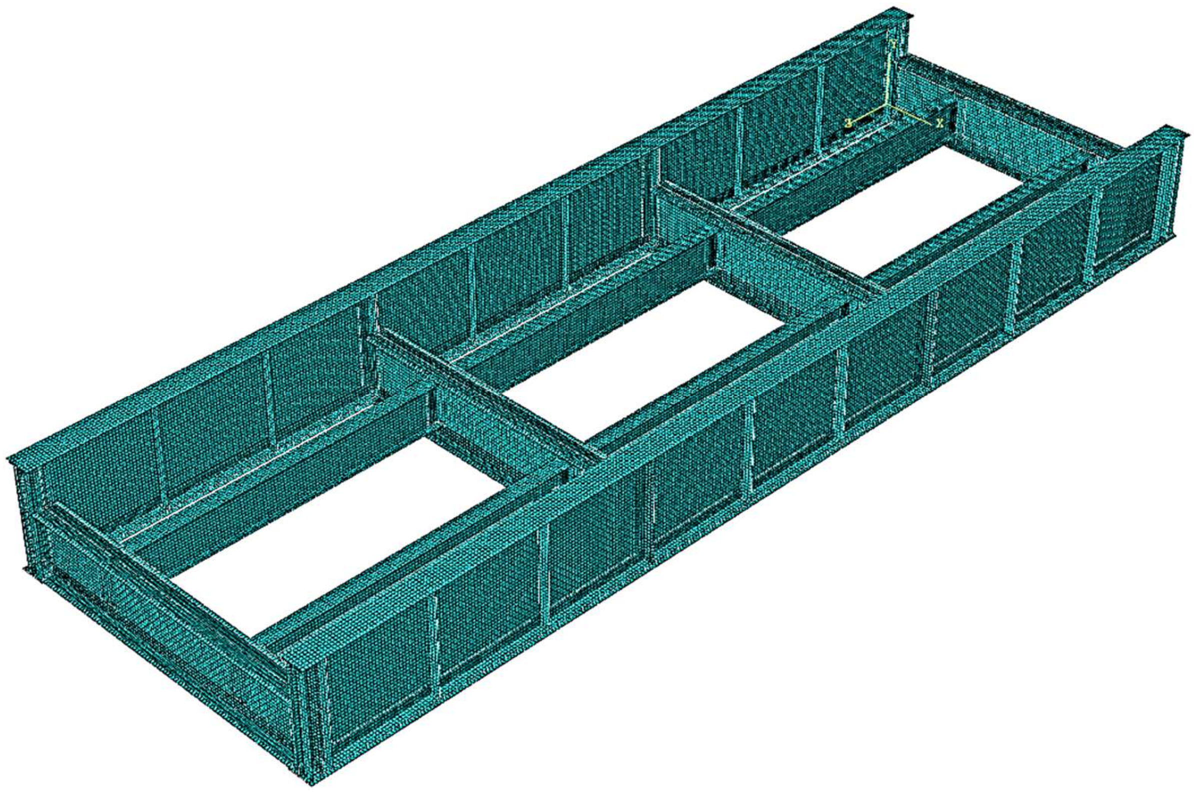

Fig. 1. The floor beam of through-plate girder bridge

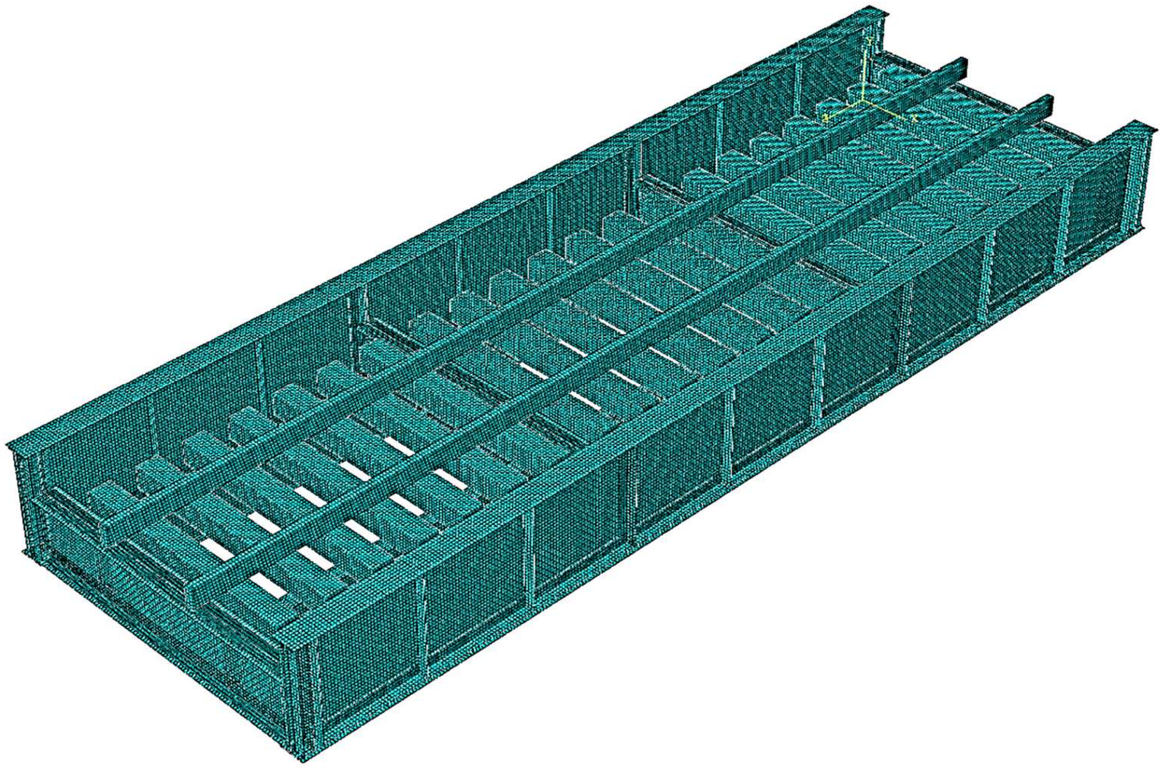

Fig. 2. The floor beam of through-plate girder bridge with rail ties and rails.

The main structural components include two main plate girders and a floor system of floor-beams and stringers. The girders are spaced transversely at 10 ft. 2 in. (3.1 m) from center to center, the floor beams are spaced 10 ft. 11 in. (3.33 m) in the longitudinal direction, and the stringers are spaced transversely at 4 ft. 11 in. (1.6 m). The details about the dimentions and drawings of connections is presented in the previous, [1, 7].

The FEM model was used to investigate behavior and performance of the bridge under moving load. In the FEM analysis, the concentrated load representing unit force was placed at each 0.1 ft and moved over the bridge. Using this approach, an influence line for each member of the bridge was developed. The FEM analysis showed that the most critical points of the bridge remain in elastic stage under the design load, [3]. It is expected that the loading spectra under current operating conditions do not exceed the design load. Therefore, for further analysis the principal of super position could be applied.

The response spectra for each component of the bridge were obtained under the statistical load model described in previous research by Rakoczy and Nowak, [3], and using developed algorithm in Mat Lab software. The scheme for the algorithm was based on the research of Tobias et al. [8]. It includes train simulation and calculation of stress history. Based on the developed stress history it is possible to calculate number of cycles and effective stress range. The details abut model of the bridge, properties of the components and material characteristic is given in the previous research, [1].

3. Fatigue analysis

For variable stress history, the rain-flow cycle counting is a method recommended by ASTM. This method counts the number of fully reversal cycles as well as half cycles and their range amplitude for a given load time history. A fully reversal cycle is when a cycle range goes up to its peak and back to the starting position. A half cycle goes only in one direction, from the "valley" to the "peak" or from the "peak" to the "valley", [9].

When the number of cycles of stress range is determined, Miner's rule may be applied. Generally, Miner's law is proposed to find the relationship between variable-amplitude fatigue behavior and constant-amplitude behavior. According to the Palmgren-Miner's rule, fatigue damage due to a variable-amplitude loading is expressed by the equation shown in Eq. 1.

$$D = \sum_i \frac{n_i}{N_i} \tag{1}$$

Where D is the accumulated damage; ni is the number of cycles at ith stress range magnitude; and N_i is the corresponding N value from S-N curve at i^{th} stress range magnitude, [10]. Theoretical failure occurs when the sum of the incremental damage equals or exceeds 1. In practice, a value of D less than unity indicates failure.

Miner's rule can be rearranged to develop an equivalent constant amplitude cycling loading. The equivalent constant stress produces the same fatigue damage as a variable amplitude load for the same number of cycles, [11]. This theory is based on the exponential model of stress range life relationship presented in Eq. 2, [12]:

$$N = AS^{-n} \tag{2}$$

where N is number of cycles to failure, S is the nominal stress range, A is a constant for a given detail and n is the slope constant. After short derivation and assumption that the number of cycles at ith stress range magnitude ni, is a product of the probability of occurrence of cycle with amplitude S_i and the total number of cycles N_T, the equivalent stress range is:

$$S_e = \sqrt[n]{\sum_i p_i S_i^n} \tag{3}$$

where S_e is the equivalent stress for a constant amplitude. The exponent n for most structural details is 3 and, therefore, the final equation for equivalent stress is referred as a Root Mean Cube (RMC) of the stress distribution Eq. 4.

$$S_e = \sqrt[3]{\sum_i p_i S_i^3} = \sqrt[3]{\sum_i \frac{n_i}{N_T} S_i^3} \tag{4}$$

Based on this general algorithm, the simulation of unit train is repeated 5000 times and the cumulative distribution function (CDF) of the accumulated damage, $(S^3N)(1/3)$, are plotted on the normal probability paper for each component of the bridge. Then, the statistical parameters of load are derived. The calculation was performed for described previously bridge. The results of the analysis are presented in the Figures 3 through 6.

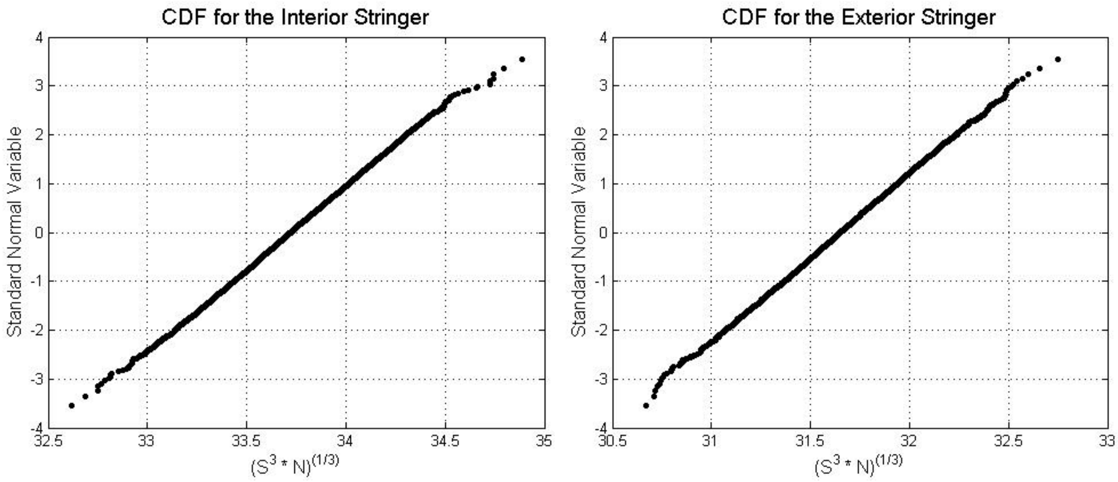

Figure 3. CDF of accumulated damage, $(S^3N)(1/3)$, for stingers, bridge #1

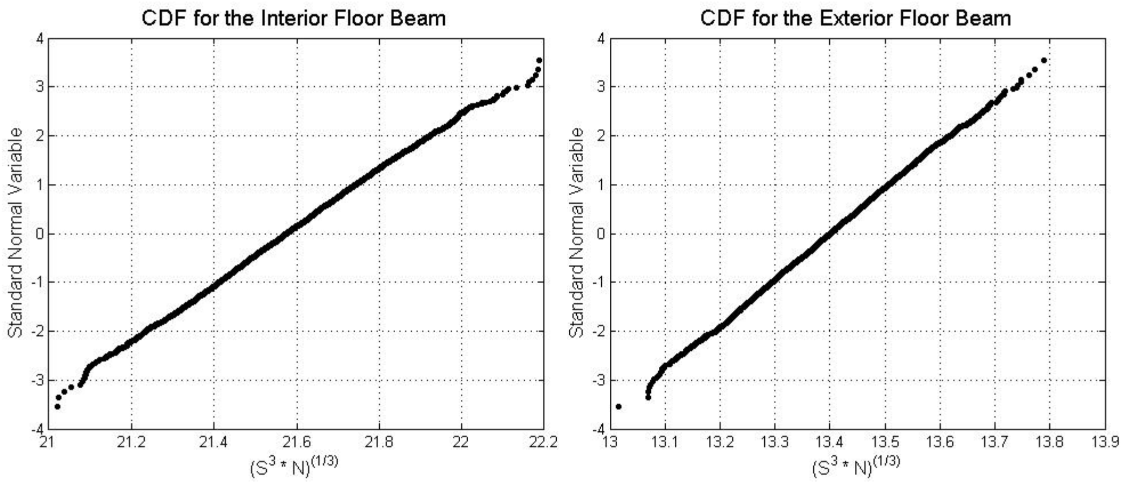

Figure 4. CDF of accumulated damage, $(S^3N)(1/3)$, for floor beams, bridge #1

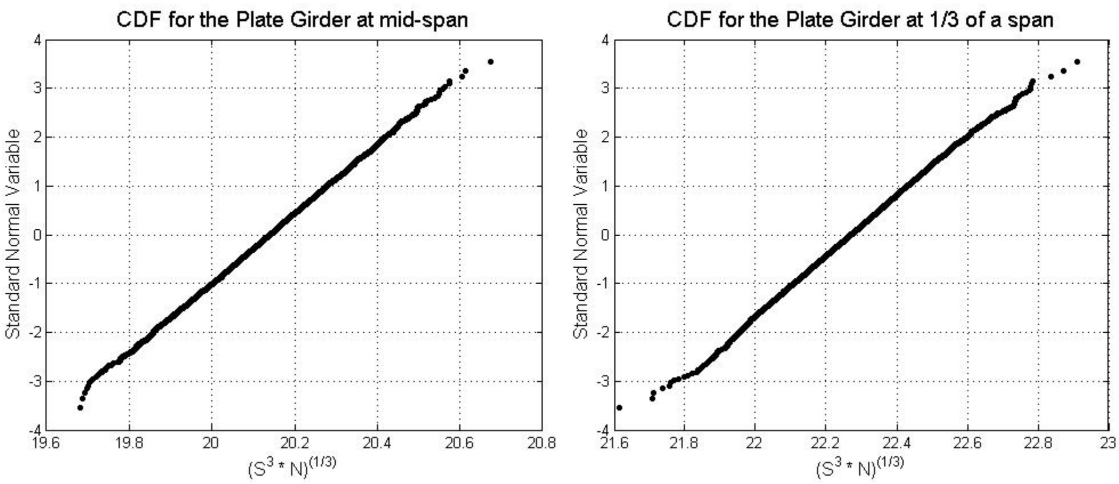

Figure 5. CDF of accumulated damage, $(S^3N)(1/3)$, for plate girder, bridge #1

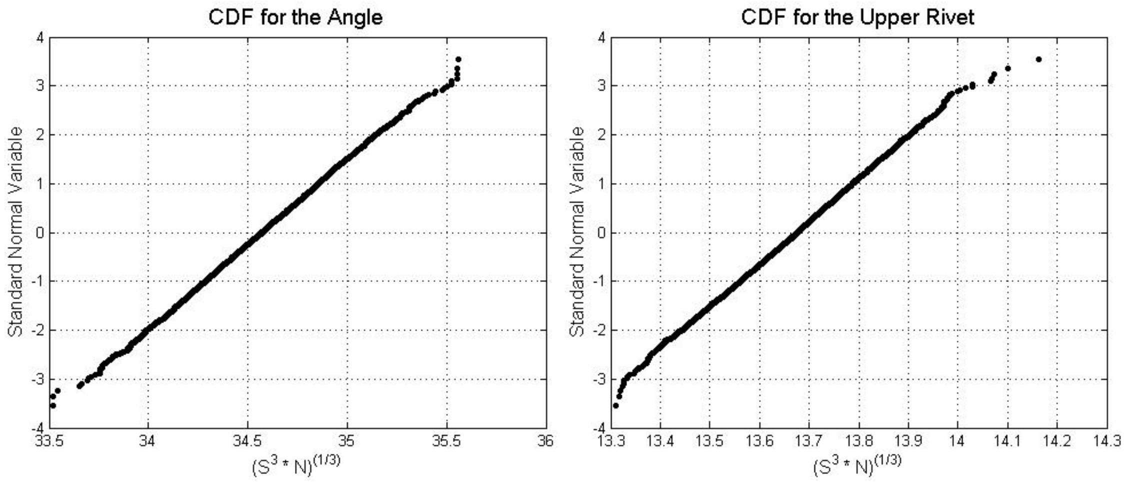

Figure 6 CDF of accumulated damage, (S^3N)(1/3), for stringer-to-floor-beam connections, bridge #1

The results of fatigue analysis presented on the normal probability paper indicate that the accumulated damage for each component and connection is close to the straight line. If the curve is close to a straight line, then the variable can be considered as a normal random variable, [13]. Therefore, the statistical parameters are determined directly from the graph and they are presented in the table 1.

Table 1. The statistical parameters of the fatigue load for bridge #1

Member	**# of cycles per train**		**Equivalent stress**		**(S3N)(1/3)**	
	Mean, μ	**CoV, V**	**Mean, μ**	**CoV, V**	**Mean, μ**	**CoV, V**
Interior Stringer	764	0.003	3.69	0.008	33.72	0.0084
Exterior Stringer	718	0.004	3.53	0.009	31.65	0.0089
Interior Floor Beam	370	0.008	3.01	0.008	21.58	0.0076
Exterior Floor Beam	807	0.004	1.44	0.008	13.40	0.0079
Plate girder, center	316	0.006	2.96	0.007	20.14	0.0069
Plate girder, 1/3 L	316	0.003	3.27	0.007	22.27	0.0073
Connection - Angle	593	0.013	4.12	0.009	34.57	0.0082
Connection - Rivet	481	0.010	1.75	0.009	13.68	0.0084

4. Reliability analysis

The load and the resistance model for fatigue limit state contain many uncertainties. For that reason, evaluation of bridge performance needs to be analyzed by using probabilistic methods. There are several procedures of reliability analysis available for the structural performance in ultimate limit state; however, fatigue evaluation in terms of reliability is not well developed.

The limit state function for fatigue in through-plate girder railway bridges can be expressed in terms of the damage ratio, as seen in Eq. 5.

$$S_e = \sqrt[3]{\sum_i p_i S_i^3} = \sqrt[3]{\sum_i \frac{n_i}{N_T} S_i^3} \tag{5}$$

If we replace the nominator by a Q and denominator by R we can obtain the simple limit state function presented in the Chapter 2.3, as seen in Eq. 6.

$$g\left(Q,R\right)=\frac{\sqrt[3]{\sum_{i} S_{Qi}^{\ 3}\cdot N_{Qi}}}{\sqrt[3]{\sum_{i} S_{Ri}^{\ 3}\cdot N_{Ri}}}=\frac{Q}{R}=1 \tag{6}$$

Since the statistical parameters of load and resistance were developed, the reliability index can be calculated using a simple formula. Both variables, *Q* and *R*, demonstrated characteristics of normal distribution. Therefore, the basic statistical parameters which are required for reliability analysis are mean value, μ, standard deviation, σ, and coefficient of variation, *V*. For special cases, such as a case of two normal distributed, uncorrelated random variables, *R* and *Q*, reliability index is given by Eq. 7.

$$\beta=\frac{\mu_R-\mu_Q}{\sqrt{\sigma_R^{\ 2}+\sigma_Q^{\ 2}}} \tag{7}$$

To calculate reliability index we must specify fatigue category and total load on the bridge. The through-plate girder contains mainly two categories of details: the riveted connections, such as riveted cover plates, and the double angle connection. Therefore, for Interior and Exterior Stringers, the Category A will be used, while for Floor Beams, Plate Girders and Stringer-to-Floor-Beam Connections Category D will be used. The statistical parameters of all Categories are presented in the table 2, [1].

Table 2. The statistical parameters of the fatigue resistance

Category	**A**	**B**	**B’**	**C**	**C’**	**D**	**E**	**E’**
Mean value, μ	4205	2980	2280	2430	2050	1810	1200	1150
Standard deviation, σ	835	425	250	480	370	250	140	240
Coefficient of variation, V	20%	14%	11%	20%	18%	14%	12%	21%

Whereas the load on the railway bridges is defined in terms of million gross metric tons per year, the statistical parameters for the accumulated damage were developed based on the average unit train which contains 200 cars. To find a gross weight of 1 MGMT, the multiple unit trains were used. Since a simulation was done for 5000 trains, the total gross weight was about 50 MGMT. Therefore, it was possible to distinguish different ranges of load of 1 MGMT, 5 MGMT, and 10 MGMT, and obtain the statistical parameters. The summary of statistical parameters for both bridges is presented in table 3.

Table 3. Statistical parameters of the accumulated damage, $(S^3N)^{(1/3)}$, for unit train and GW equal 1, 5, and 10 MGMT.

Member	**Mean value of $(S^3N)^{(1/3)}$**				**CoV, V**
	Unit train	**1 MGMT**	**5 MGMT**	**10 MGMT**	
Interior Stringer	33.72	191.04	326.68	411.59	0.0084
Exterior Stringer	31.65	179.37	306.72	386.44	0.0089
Interior Floor Beam	21.58	122.30	209.13	263.49	0.0076
Exterior Floor Beam	13.40	75.97	129.90	163.66	0.0079
Plate girder, center	20.14	114.13	195.15	245.88	0.0069
Plate girder, 1/3 L	22.27	126.19	215.79	271.88	0.0073
Connection – Angle	34.57	196.01	335.17	422.29	0.0082
Connection – Rivet	13.68	77.50	132.52	166.96	0.0084

The calculations of predicted years of service were carried out. Three cases of load were considered: 1, 5, and 10 MGMT per year. The reliability indices were fixed and were equal 0, 0.5, 1.0, 1.35 and 1.75. Recently, many researchers use $\beta = 0$ in the fatigue analysis of railway bridges (Tobias et al. 1997; Imam 2005; Imam 2008). Even if the reliability index for fatigue evaluations can be relatively low, $\beta = 0$ is too low. For the evaluation of existing highway bridges, the target beta is $\beta T = 1.35$ for redundant and $\beta T = 1.75$ for non-redundant members according to AASHTO Guide Specifications for Fatigue Evaluation of Existing Steel Bridges, [14]. Therefore, the reliability index for railway bridges also should be retained higher than 0. The results of this analysis are shown on the Figures 7 to 9.

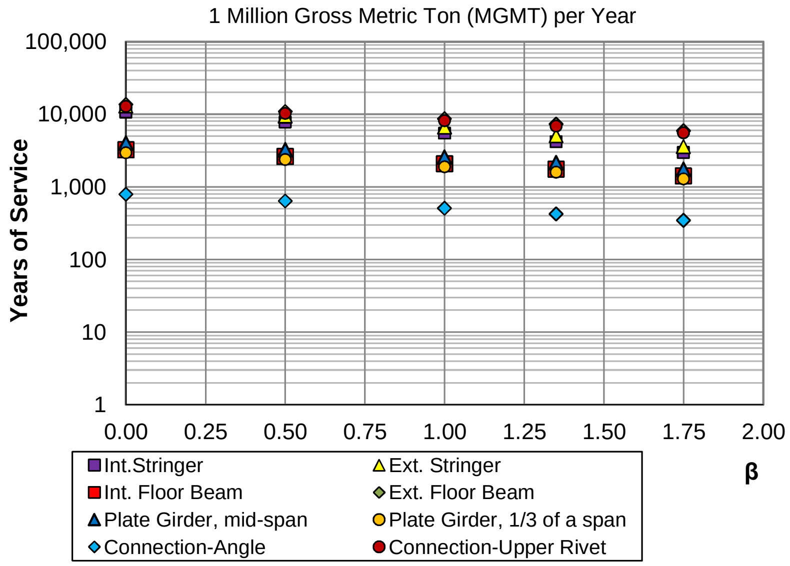

Figure 7. Predicted years of service for Bridge #1 subjected to 1 MGMT per year

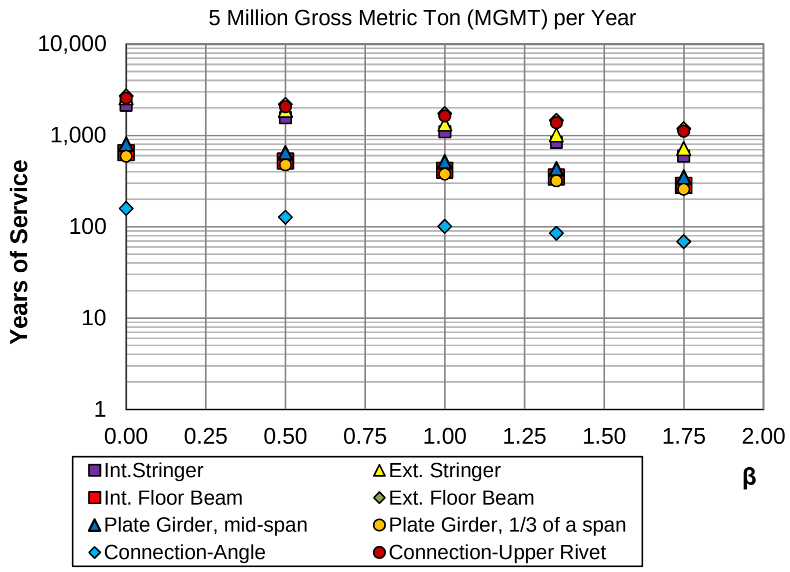

Figure 8. Predicted years of service for Bridge #1 subjected to 5 MGMT per year

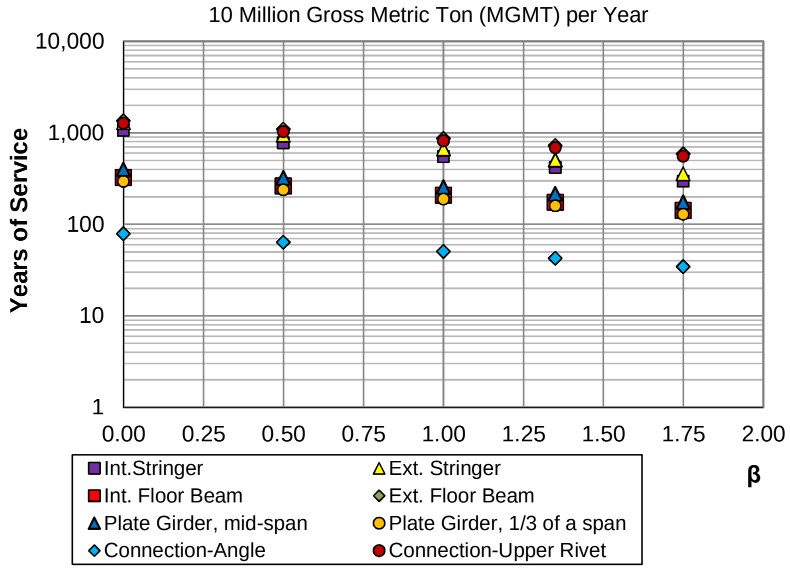

Figure 9. Predicted years of service for Bridge #1 subjected to 10 MGMT per year

The results shows that bridge is able to carry a load equal 1 MGMT per year for more than 300 years with β = 1.75. This means that the components and connections have very small probability of occurrence damage due to fatigue in these periods of time. Reliability index β = 2 corresponds to 2.0% of probability of failure, β = 1 corresponds to P_f = 15.0%, and β = 0 corresponds to P_f = 50.0%. For 5 MGMT per year, bridge still has a high probability that will not have a damage caused by fatigue; whereas, for the last of the case, in which the load is 10 MGMT per year, the connection reached only 30 years with β = 1.75. In each considered cases of load, the lowest predicted years of service were achieved for the angle in the Stringer-to-Floor-Beam connection. This analysis confirms that the weakest link in the bridge system is the connections.

5. Summary and Conclusions

The fatigue life of structural elements was estimated based on the S-N curves, which present the number of cycles to failure as a function of the constant stress amplitude. The S-N fatigue data, created in a laboratory, contains a considerable amount of scatter, even when standard specimens made from the same material are used [1].

In the reliability analysis, both loading and strength were treated as random variables. The loading side was classified through the gross weight of train traffic per year. The response of the bridge components and connection were simulated using influence lines developed in the FEM and algorithm written in the Mat Lab. The probability of failure for fatigue was calculated by using damage ratio as a limit state function and the distribution of load and resistance. The fatigue was considered in eight critical places on the bridge: mid-span of interior and exterior stringers, mid-span of interior and exterior floor beams, the plate girder in center and quarter of the span, angle and rivet in the stinger-to-floor-beam connections. Total damage in the components and the connections were calculated under the statistical load model for freight and passenger trains. This study give a broad view of the potential remaining fatigue lives of typical railway bridges subjected to unit train loadings.

The currently acceptable reliability index for fatigue in older bridges is 0. However, for the design of new bridges it is recommended to increase the reliability index to 1.5. During service of the bridge the accumulated fatigue damage is increasing in time at different rates, depending on tonnage per year and train type. The reliability approach is the reasonable way to evaluate performance of the railway bridges due to high degree of uncertainty in the fatigue strength of riveted details and loading conditions.

The reliability analysis for the fatigue limit state was presented for various safety levels and through three cases of operating conditions. In each of the considered cases of load, the lowest predicted years of service were achieved for the angle in the stringer-to-floor-beam connection. This study has confirmed that riveted bridges are not likely to develop fatigue cracks in the primary members because the cyclic loads do not result in stress range levels that exceed the estimate fatigue limit for riveted members (Category D). However, the weakest link in the bridge system is the connection.

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