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SIMULATION-BASED RELIABILITY ANALYSIS OF STEEL GIRDER RAILWAY BRIDGES

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Abstract. Bridges are an essential component of the transportation system and safety and sustainability of bridges are critical for the efficient operation thereof. Due to scarcity of resources, an economical way should be determined to design and maintain bridges and the transportation system in general. Reliability indexes are widely used in the analysis of these concepts within a semi-probabilistic approach. However, advances in computer technology allow implementing a fully-probabilistic approach. This study represents a simulation-based reliability analysis of steel girder bridges in the railway lines. Statistical parameters of the bridges are determined both analysing the existing body of knowledge available in the literature and conducting specimen tests. The Bayesian approach is used to update the statistical properties of the steel material. Basic Monte Carlo Simulation (MCS) is used to simulate the load and resistance of the bridge. The reliability of the bridges is determined according to their ultimate limit states and statistical load distribution. By using simulation, the consistency of the log-normal marginal distribution obtained is analysed herein.

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Introduction

Sustainable economic development and social welfare in any country depend on the continuity of the structural and infrastructural systems. There are numerous bridges that have almost reached the end of their service life in many developed countries, therefore, the performance of these bridges needs to be assessed to sustain the transportation system safety. Bridges being the most vulnerable component of the transportation systems are exposed to many different deterioration phenomena during their service life. Corrosion spills in concrete sections, fatigue, and cracks can be considered examples of these damages and deterioration processes (Ellingwood, 2005). Harmful chemicals, freeze-thaw effect, global warming, increased carbon emission rate, unexpected accidents, floods, landslides, and earthquakes are among the main reasons causing these damages. The effects of aging and the change in bridge performance over time should be determined to ensure the continuity of transportation systems in the economical and reliable way. Mathematical descriptions of the physical mechanisms underlying the deterioration process are generally not available, therefore, empirical formulas are obtained with the help of probabilistic methods (Apostolopoulos & Papadakis, 2008; Biondini et al., 2006).

The reliability analysis determines the failure probability of a bridge under service loads. Consideration of the structural and external uncertainties in the analysis is the most important advantage of this approach. Monte Carlo simulation (MCS) is the most commonly used probabilistic method to determine the failure probability of a bridge (Wu et al., 2020). However, these methods are computationally expensive, so alternative approaches are proposed to minimize the analysis time, such as using deep learning (Nabian & Meidani, 2018), important sampling methods (Cheng et al., 2005) and Latin hypercube sampling methods (Novák et al., 2014).

Theoretically, there are three approaches to evaluate the reliability index: deterministic, semi-probabilistic, and probabilistic. The existing codes and specifications are used the semi-probabilistic approach to determine the bridge reliability index (Akgül & Frangopol, 2004a). AASHTO LRFD design concept was created for girder, truss, and arc steel bridges in 1994. The LRFD design concept increases the load while decreasing the resistance using a constant that satisfies the reliability index of 3.5. These coefficients are determined conducting the probabilistic analysis of different existing bridge structures.

AASHTO design concepts are also used in many different countries, and the LRFD reliability-based approach is used to design new bridge structures. The LRFD design concept allows considering the material and load distribution in reliability analysis adopting a simplified approach. However, according to new statistical data, re-calibration of the load and resistant factor is required (Leblouba et al., 2020). The computer technology investigations and scientific studies about the reliability index allow implementing a full-probabilistic approach in new bridge design and performance evaluation of the existing bridges (Kuroda & Nishio, 2020; Nguyen, 2020; Sommer et al., 1993; Tabsh & Nowak, 1992).

UIC (2006) allowable stress design approach is used in Turkey to perform reliability analysis of the existing steel railway bridges (UIC-71, 2006). According to the specifications and design guides, the existing semi-probabilistic methods need to be replaced by probabilistic approaches over time (Akgül & Frangopol, 2004a). Within the probabilistic approach, the load and strength parameters are simulated by considering their statistical distributions (Biondini & Frangopol, 2016, 2019). This approach also allows ensuring a safe and economically viable service life of the bridges.

In this study, safety index of a steel girder railway bridge situated in Turkey was evaluated with a fully probabilistic simulation approach considering the capacity of the main girders. The buckling analysis for the compression part of the main girders was performed by the incremental load method, the bending capacity and the plastic moment capacity of the bridge were determined as well. Bridge reliability analysis was conducted using the MCS considering the uncertainties of the bridge girders, materials, and loads. Statistical steel material properties were determined based on the information available in the literature and then updated with the Bayesian approach using the results of specimen tests.

1. Ultimate capacity of the bridge

1.1. Description of the bridge

Steel girder bridges are an important element of the railway networks in Turkey. These bridges are easy to construct; they usually have a ballasted deck. The selected bridge is part of the Manisa-Afyon railway line in the west of the Turkey. The length of the bridge span is 21 m, it consists of two main steel girders, cross beams, and stringers. Retainer plates of the ballasted deck prevent falling of the ballast. The height of the main girder, which has built-up sections made of flange plates, a web plate and four angle sections, is 1820 mm. The height of the ballast layer is approximately 45 cm. Stringers with IPN300 and UPN240 sections are placed at 1.5 m intervals transversely. Cross beams with IPN450 section also are spaced at 1.5 m. Schematic and general views of the bridge are given in Figures 1 and 2.

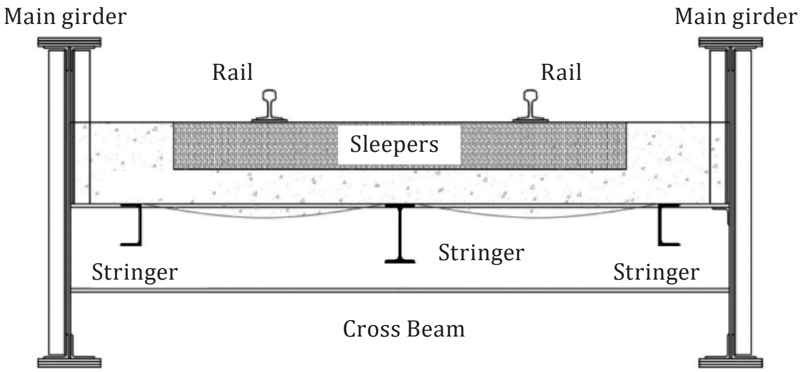
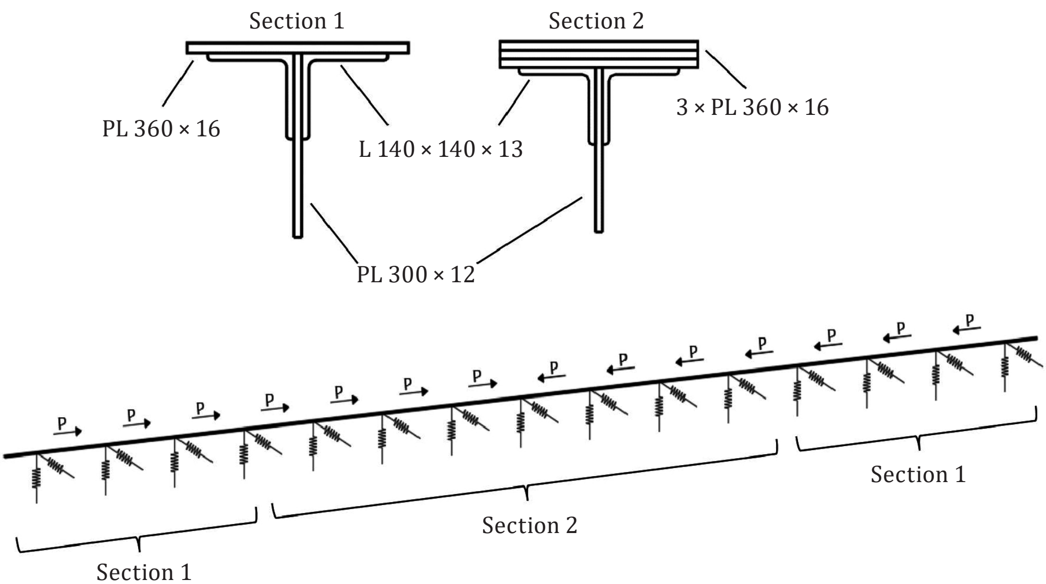


Figure 1. Cross-sectional view of the bridge

Figure 2. General view of the bridge

1.2. Determination of the bending moment capacity of the bridge

The bending moment capacity of the bridge was determined considering the capacity of two main beams. Since the length of the clean span of the bridge was 21 m, it was necessary to check whether the lateral-torsional buckling should be considered or not. In the original design drawings, it was observed that each cross beam connected to the main beams was strengthened with a stiffener plate up to the compression top part of the cross-section of the main girder. It is clear that these stiffener plates increase the shear force capacity of the main beam and prevent lateral buckling of the compression zone of the main girder section in the area where the cross beam is connected. In this study, the effect caused by the stiffener plates on the buckling of the compression zone was considered with the help of the springs defined in the finite element model of the bridge. A finite element model was created for the section shown in Figure 1 to determine these spring coefficients. A lateral unit load was applied to the compression part of the main girder, and the resultant displacement values were determined. For the compression zone of the main girder (see Figure 3), the axial compressive load was imposed in proportion to the moment distribution.



Note: Units in mm

Figure 3. Section details and FE model's schematic view of the compression part of the main girder

In the finite element model, an incremental loading analysis was carried out by considering the material and geometrical nonlinearities. The variable pressure load P was gradually increased until the system became unstable. The maximum of this incremental load gave the collapse load of the compression bar of the section of the main girder.

FE results showed that the collapse of the compression bars occurred after Section 1 failed to carry the compression load. Section 2 did not reach its ultimate capacity, but an influence on the yielding capacity was observed. The collapse stresses for Section 1 and Section 2 were determined as 323.29 N/mm^2 and 268.92 N/mm^2, respectively. Figure 4 shows the axial load distribution in the collapse state of the compression zone and the plastic hinges formed in the cross-section. The plastic joint colors are magenta, blue, and red, which correspond to slight, moderate and collapse damage. It was observed that the yield stress was exceeded before buckling for both sections. The main girder reached its plastic moment capacity without showing any lateral-torsional buckling. Therefore, the plastic moment capacity of the main girders was used to determine the bending capacity of the bridge.

Since in the riveted girders the plates are held by the riveted connections at the ends of the plate, the edges cannot move towards each other, so the axial stresses occur along the plate. This behavior causes reduction of the bending stresses and the corresponding deflection, thus a higher load bearing capacity of the plate is achieved. However, if the riveted connection can resist the effects of this axial force, the plate gains additional capacity for the plastic behavior (Imam & Collins, 2013). Considering that this axial force moves across the width of the plate and is transferred through the riveted connection, the study performed by Cremona et al. (2013) stated that if an I-girder has riveted flange and web plates as considered in the presented study, then the limitations for cross-section classification can be used considering Eurocode No. 4 (CEN 1994). For Class 1, the upper limits for spacing of rivets on plates subjected to compression supported by web and angles are defined as 10 tε for outstand flange and 40 tε for interior flange in transverse to direction of the compressive stress, with additional 12 tε for outstand

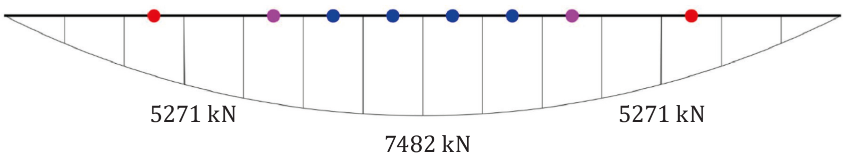


Figure 4. Axial load distribution in the compression section for the collapse load

and interior flange in the direction of the compressive stress, where t is the thickness of the compression part and ε is equal to $\sqrt{235 / f_y}$ (Cremona et al, 2013). Cremona et. al. (2013) stated that it is expedient to use plastic hinge analysis for a girder with all cross-sections belonging to Class 1. Since the riveted I-girders considered in this study were classified as belonging to Class 1 in terms of spacing of the rivets on the plates, plastic hinge concept was appropriately used for the limit analysis of the main girders.

1.3. Limit analysis of the bridge

Rigid perfectly-plastic constitutive laws and equal energy assumptions form the basics of the limit analysis. The internal and external energy must be equal. With regard to the yielding criterion, the starting point of plastic flow and flow rule, the plastic strain increases in the correlation to the stress state. The limit analysis determines the load-carrying capacity of the structure with a load multiplier. There are two basic approaches to performing the limit analysis: the kinematic approach (upper bound) and the static approach (lower bound) (Biondini, 2000). The kinematic approach assumes that the considered materials are perfectly plastic and geometry changes are insignificant at the limit load (He et al., 2012). Using this method, all possible collapse mechanisms were examined and the minimum load multiplier gave the critical limit load. By comparing the bending moment diagrams for the collapse load, it was checked whether plastic moment capacity was exceeded in any section. If not, the selected mechanism gave the critical collapse mechanism. In the static approach (lower bound), bending moment diagrams were generalized and the loads were increased until the first plastic hinge formation was reached. Then by changing the structural model with the new plastic hinge formation and bending moment diagram of the new model, analyses were repeated

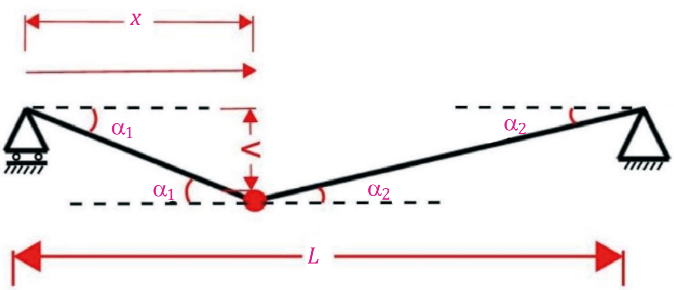


Figure 5. Collapse mechanism for the main girder of the bridge

by increasing the load to examine the process of second plastic hinge formation until the system became unstable. At this point, the critical load multiplier was determined.

Plastic hinge formation as the collapse mechanism of the selected steel girder is shown in Figure 5. The virtual work equation is derived for the distributed load, as shown in Equations (1)–(3). The work done by internal forces can be expressed as the multiplication of moment capacity of the section and the corresponding rotation. The rotation of plastic hinges is obtained considering the position of plastic hinges (x), as given in Equation (1). The work done by the exterior loads is expressed as the multiplication of exterior load derivation and the corresponding displacement calculated considering (x) as given in Equation (2). The load multiplier (ϕ) is calculated by Equation (4). Position of the plastic hinges can be determined by minimizing of the load multiplier (ϕ) for position of plastic hinges (x) varying from 0 to L.

$$\alpha_1 = \frac{\nu}{x}, \quad \alpha_2 = \frac{\nu}{L-x}, \tag{1}$$

where α_1 and α_2 are the rotation at the end supports, ν is the vertical displacement at the point of plastic formation, x is the position of the plastic hinge and L is the length of the bridge span.

Work done by the external load

The left side of the plastic hinge　　The right side of the plastic hinge

$$\sum_{i=1}^{n} \frac{q_i \frac{\nu \cdot x_i}{x}}{2} + \sum_{j=1}^{n} P_j \frac{\nu \cdot x_i}{x} \qquad \sum_{i=1}^{n} \frac{q_i \frac{\nu \cdot x_i}{(L-x)}}{2} + \sum_{j=1}^{n} P_j \frac{\nu \cdot x_i}{(L-x)}$$

$$+G\frac{\nu \cdot L}{2}, \tag{2}$$

where q and P are the external distributed and point loads, respectively, and G is the self-weight per-unit length of the main girder.

Work done by plastic deformation

$$M_p(\alpha_1 + \alpha_2)$$

$$M_p\left(\frac{\nu}{x} + \frac{\nu}{L-x}\right) \rightarrow M_p \frac{\nu}{(L-x) \cdot \zeta} \rightarrow \zeta = \frac{x}{L}, \tag{3}$$

where M_p is the plastic moment capacity of the main girder and ξ is the relation between x and L.

Calculating the load multiplier

$$\phi = \frac{M_p \frac{\nu}{(L-x) \cdot \zeta} - G\frac{\nu \cdot L}{2}}{\sum_{i=1}^{n} \frac{q_i \frac{\nu \cdot x_i}{x}}{2} + \sum_{j=1}^{n} P_j \frac{\nu \cdot x_i}{x} + \sum_{i=1}^{n} \frac{q_i \frac{\nu \cdot x_i}{L-x}}{2} + \sum_{j=1}^{n} P_j \frac{\nu \cdot x_i}{(L-x)}} \tag{4}$$

2. Determination of uncertainties

Determination of statistical and deterministic properties of the bridge are critical for estimation of its reliability. Akgül & Frangopol (2004b) determined random variables (i.e., steel yield strength, the weight of the concrete, dead and live load moment distributions, section modules and the cross-sectional area of the steel girder) having log-normal distribution for probabilistic reliability analysis of steel girder bridges. Statistical definition of material probability and geometry of the bridge structure can be defined based on the available information in the literature and experimental data from the characterization tests. These statistic definitions can be updated if additional information becomes available with the help of the Bayesian approach (Moreira et al., 2016). The probabilistic methods include two main steps. The first one is estimation of the critical model parameter based on the numerical model and experimental data. Expected values of the material, geometrical and mechanical properties used in the reliability analysis of the structure are also investigated (Matos et al., 2016).

Load and strength uncertainties were simulated in these studies. The weight, ballast, sleeper and rail weights of the steel bridge were considered as the dead load, and train load was considered the live load. The statistical distribution of the train load and wagon loads was determined by applying different long-term measurements (Imam et al., 2008; O'Connor et al., 2009). The obtained histograms were categorized to define the empty, half-loaded, and fully-loaded wagons and then log-normal mean and standard deviation of the parameters were determined. The existing material test of a similar steel railway bridge in Turkey was used to update the statistical distribution of the steel material. Bridge bending moment capacities were simulated considering the log-normal distribution of the steel yielding strength and the geometrical properties of the steel section. Random variables for steel girder bridges obtained from the literature review are given in Table 1.

Table 1. Random variables for steel girder bridges

Variable	Mean μ	Cov	Distribution	Reference
Thickness of Plate (*t*)	–	0.04	Normal	(Guedes Soares, 1988)
Length of Plate (*b*)	–	0.01	Normal	
Yield Strength (*Fy*)	257.2 MPa	0.068	Lognormal	(Hess et al., 2002)
	252.56 MPa	0.12	Lognormal	(Akgül & Frangopol, 2004)
Dead Load, kN/m	–	0.1	Lognormal	(Moreira et al., 2016)
Train Load	–	0.1–0.3	Lognormal	(Imam et al., 2008)
	–	~0.1	Lognormal	(O'Connor et al., 2009)

2.1. Bayesian update of the yielding strength of steel

Statistical distributions of bridges are determined considering both the existing information and experimental data. The Bayesian approach is a promising approach that allows using both types of information to determine the probabilistic properties of the bridge.

Initially, the possible values of parameter θ with a prior relative likelihood $p_i = P(\Theta = \theta_i)$ exist. With the available additional information, the parameter's prior assumption θ is modified through the Bayes Theorem (Ang & Tang, 2007) given in Equation (5).

$$P(\Theta=\theta_i|\varepsilon)=\frac{P(\varepsilon|\Theta=\theta_i)P(\Theta=\theta_i)}{\sum_{i=1}^{k}P(\varepsilon|\Theta=\theta_i)P(\Theta=\theta_i)},\ i=1,2,...,k \tag{5}$$

Limited experimental information can be obtained from the existing bridge and in many cases conjugate distributions of these properties are not available. Therefore, the Bayesian update of these expectation probabilities does not give an explicit posterior distribution. The simulation-based approach allows generating a posterior distribution without conjugate prior and experimental distribution (Okasha & Frangopol, 2012). Metropolis-Hastings (MH) algorithm can be used to generate any model even if direct sampling is difficult and a description of the algorithm exists (MathWorks, 2009).

In this study, probabilistic distribution of steel yielding was determined based on the results of literature review. A test of the available specimen was conducted to update this information. Four specimens taken from a steel girder bridge integrated in the Turkish railway line were used to determine the yielding and ultimate strength of the steel material. The yielding strengths of these specimen were measured as 212.6, 239.9, 239.7, and 264.2 N/mm^2. While existing information was used as a prior distribution, the specimens were used to update the steel yielding probability with the help of the MH algorithm in MathWorks (2009). Figure 6 shows the prior, experimental, and posterior distribution, and Table 2 gives the corresponding statistical means and standard deviations. The standard deviation of the

Table 2. Mean and standard deviation of the statistical distribution of the yielding properties of steel

	Prior	Experimental	Posterior
Mean μ, N/mm^2	252.56	238.1	239.61
Std σ, N/mm^2	30.5	21.69	10.71

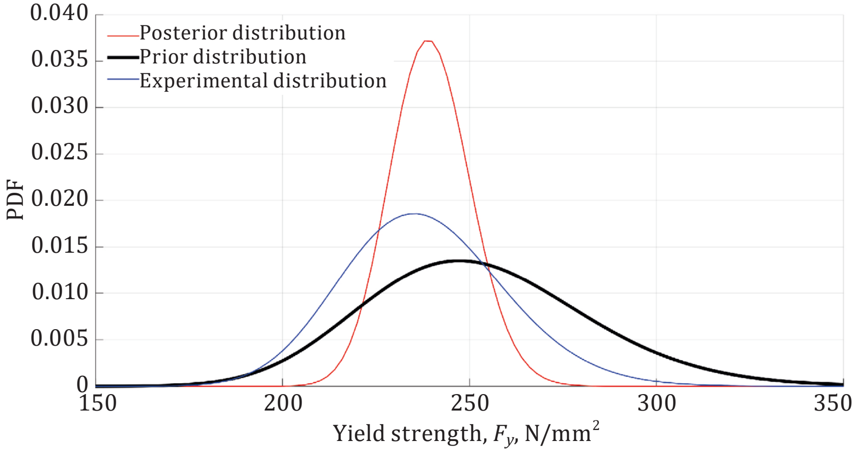
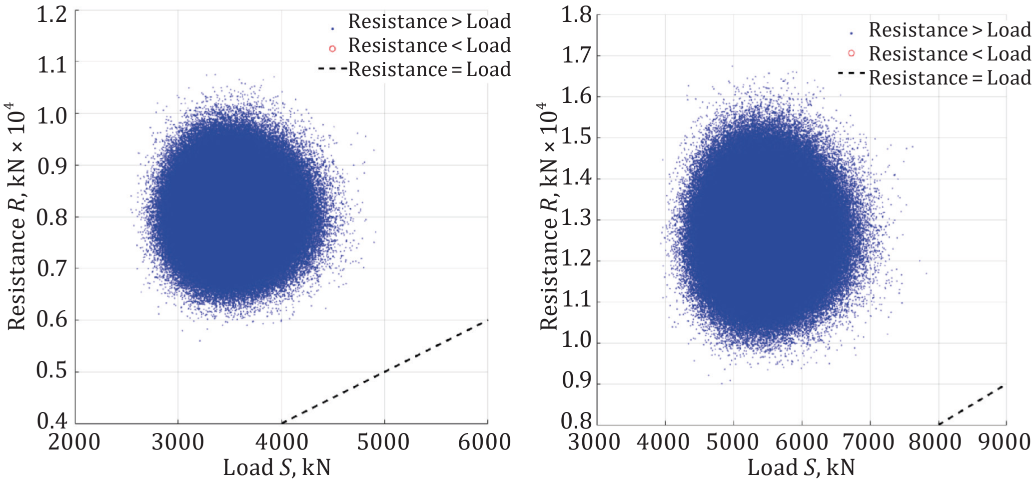


Figure 6. Prior, experimental and posterior distribution

posterior distribution is much smaller than the prior and experimental distribution. So, the Bayesian update approach decreases the epistemic and aleatory distribution.

3. Monte Carlo simulation (MCS)

The behavior of an existing or newly designed system is reproduced using the simulation process. This simulation allows the engineer to understand and manage the behavior of a structural system better. One other significant advantage is that simulation helps understand how the essential components of the system respond and behave under different statistical conditions. A transfer function is used to generate output distribution according to the input file. The analytical calculation of the output is only available for simple transfer functions that otherwise are not possible because of different uncertainties and complex actions. In this case, the number of simulations becomes critical in terms of the output results. Increasing the number of simulations would give a more realistic outcome but would also increase computational effort. Therefore, an adequate number of simulations should be determined. The scatter plot of the simulated resistance and load distributions obtained in this study is shown in Figure 7.



a) for Section 1 **b)** for Section 2

Figure 7. Scatter plot of resistance and load distributions

4. Reliability analysis of the bridge

Based on the structural reliability theory, probability of failure is determined by $P_f[R/S] = (R\text{-}S) < 0$ where R and S are capacity and load, respectively. Reliability index can be expressed as $\beta = \Phi^{-1}(P_f)$. According to the normal distribution assumption, the reliability index is determined using the following expression (Ang & Tang, 2007),

$$\beta = \frac{\mu_R - \mu_S}{\sqrt{{\sigma_R}^2 + {\sigma_S}^2}}, \tag{6}$$

where μ_R and μ_S are mean values of resistance and load, respectively, σ_R and σ_S are standard deviation of resistance and load, respectively.

In Monte Carlo simulation, basic random variables $X = \left[X_1, X_2, ..., X_n\right]^T$ generated in accordance with their $i = 1,2,...,n$, marginal density function $f_{X_i}\left(x_i\right)$ and results of a number of analyses are considered to determine the outcomes (Biondini, 2008). Resisting R and Load S distribution of both parameters can be obtained in the simulations. Considering the difference of the two parameters for all simulation samples, the number of failures can be obtained. Using the number of failures and the total number of simulations, probability of failure is calculated, as shown in Equation (7).

$$P_F = \Phi(-\beta) \cong \frac{N_{\text{fail}}}{N_{\text{simulation}}}, \beta \cong -\Phi^{-1}\left(\frac{N_{\text{fail}}}{N_{\text{simulation}}}\right) = \Phi^{-1}\left(1 - \frac{N_{\text{fail}}}{N_{\text{simulation}}}\right). \quad (7)$$

If the number of simulations goes to infinite, β shall approach the exact solution.

4.1. Simulating load multiplier

The bridge superstructure was loaded with the UIC train load model, as shown in Figure 8. The horizontal loads were neglected, because only the main girder of the bridge is cantilevered and the vertical load plays the dominant role. Moreover, neglecting the horizontal load helps simplify the probabilistic approach. Mine transportation makes an essential part of railway transportation industry in Turkey, where wagon load can by 50% exceed the design load. Therefore, the wagon loads were increased in the limit analysis. The train load was increased until the bridge collapsed. Thus, the multiplication load that caused the bridge to collapse could be obtained. The limit analysis approach described in Section 1.3 was used to determine the load multiplier. The probabilistic distributions of the structural parameters were defined in Section 3. The load multiplier was calculated based on the results of the probabilistic simulation of the bridge strength and load. If the load multiplier is less than 1, the structure fails. It was aimed to determine this probability of failure by performing the reliability analysis of the bridge. The reliability index β shows whether the bridge can safely carry the design loads. Different β values are determined as threshold values by bridge owners and specifications. Turkish Railways Administration (TCDD) use $\beta = 3$ as a safety threshold that corresponds to 4.4 ‰ probability of collapse.

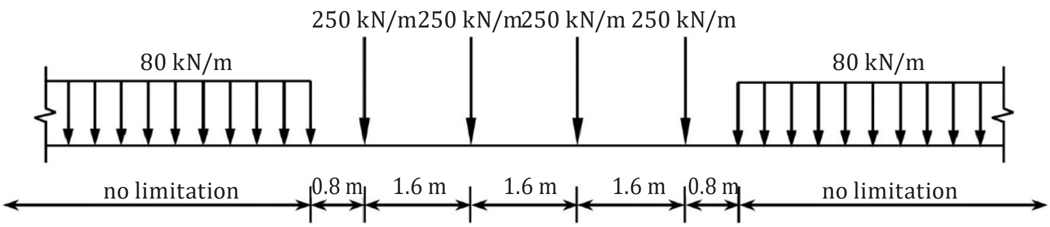
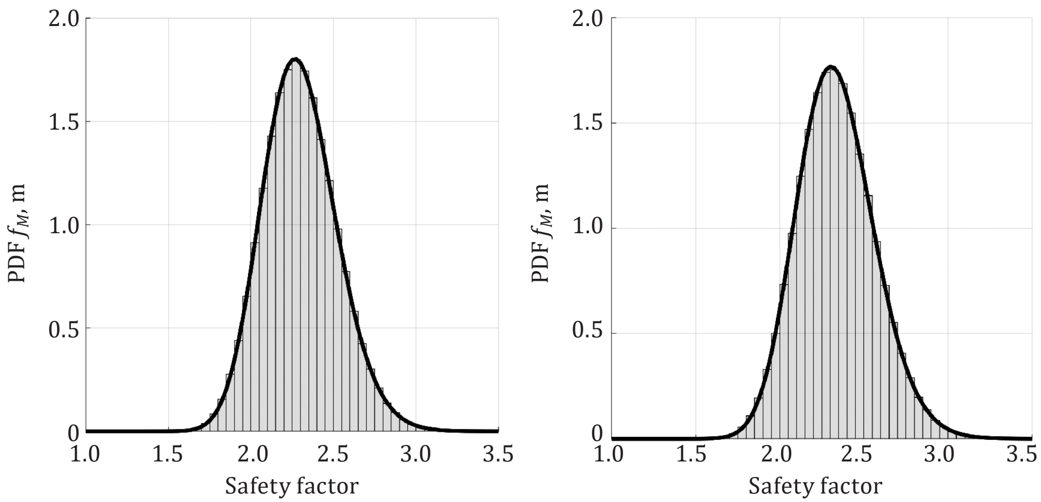


Figure 8. Live load model UIC-71 (UIC 776-1, 2006)

Limited number of simulations can be used in the MCS, and initially, the number of simulations should be determined. 10^6 simulations had been found sufficient in many studies (Biondini, 2008). Classical MSC calculates the probability of failure by dividing the number of failures by the number of simulations. It is possible to determine the bridge safety index even when the failure cannot be observed in the simulations. For this, log-normal mean and standard deviation values of load multipliers were determined. In this case, the safety index was determined by dividing log-normal mean by log-normal standard deviations. Figure 9 shows histograms of load multiplier and log-normal distribution for two bridge sections. As seen in Figure 9, the log-normal distribution represents the load coefficients with great convergence.

4.2. Determining reliability (safety) index of the bridge

The safety index was determined using the log-normal distribution approach. Besides, to determine whether the number of simulations was sufficient or not, the safety index obtained depending on the number of simulations is presented in Figure 10 for both sections. It may be seen that the safety index value converges to a constant value after 10^4 samples and does not depart from this value as the number of analyses increases. Thus, it was found that 10^4 number of simulations was sufficient to determine this bridge safety index by using the log-normal



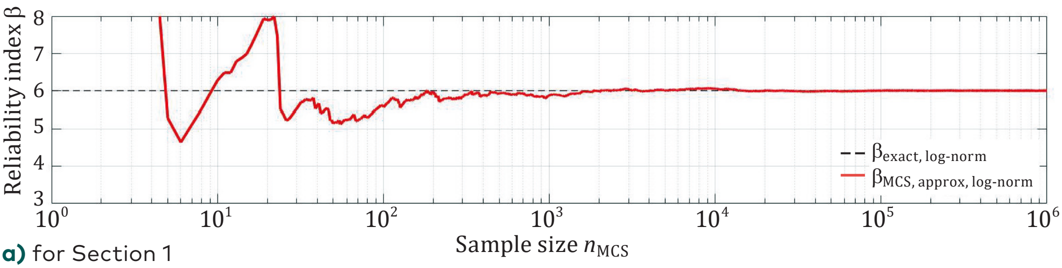
a) for Section 1 **b)** for Section 2

Figure 9. Histogram and log-normal distribution of load multiplier

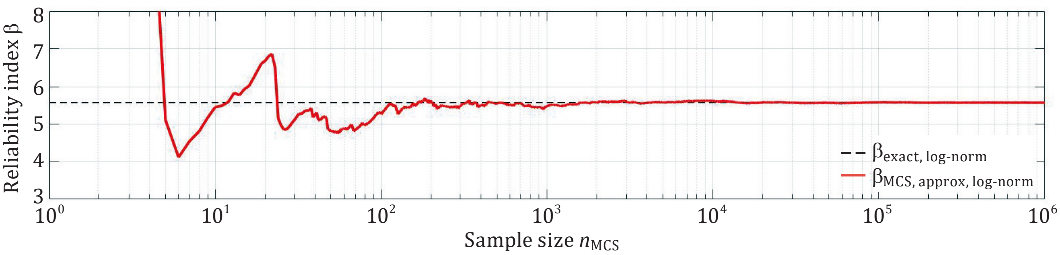
approach. The compatibility of the load multiplier with the log-normal approach was vital in determining the safety index. Reliability index for Section 1 and 2 were determined as $\beta_{\text{section }1} = 6.00$ and $\beta_{\text{section }2} = 5.57$, respectively.

5. Life-cycle reliability analysis of the bridge

Steel bridges are exposed to many different aging phenomena that decrease their structural strength and reliability. To sustain the serviceability of the bridge and transportation system in general, life-cycle reliability analysis of the bridge should be performed. In the literature it has been widely discussed that steel structures are exposed to time dependent deterioration phenomena caused by multiple factors, such as corrosion and fatigue. Corrosion reduces the thickness of the steel section and can be predicted adopting the empirical approach. Fatigue also reduces the structural integrity during the service life of a structure, as it promotes crack propagation. Similar to corrosion, fatigue-related problems are examined by using fracture mechanics theorems, fatigue failure has not been considered in this study.



a) for Section 1



b) for Section 2

Figure 10. Reliability index distribution versus the simulation number

5.1. Corrosion damage

Corrosion-caused deterioration of steel occurs as a result of its exposure to the salt water and atmospheric corrosive environment exposure (Biondini & Frangopol, 2016). Different corrosion forms have been distinguished depending on how corrosion affects the metal. Eight different corrosion forms, such as uniform corrosion, stress corrosion and pitting corrosion, were distinguished by (Fontana, 1986). Within the scope of this study, a life cycle analysis simulating the uniform corrosion in the bridge girders was performed.

In this context, the progression rate of corrosion and the initial time should be determined. A protective paint layer was applied on the steel elements to prevent corrosion. Thus, it was assumed that the onset of corrosion on steel elements was delayed for a period of time. This period was determined as 5 years in the areas with high corrosion effects near the sea and 15 years in the rural areas, depending on the environmental factors (Czarnecki & Nowak, 2008).

A suitable time-variant deterioration model is required to consider properly the life-cycle analysis of a structure. However, a deterministic expression of the deterioration process is unavailable. In this case, empirical models can be successfully adopted (Ellingwood, 2005).

$$\delta(t) = k(t - t_i)^{\eta}, \tag{8}$$

where $t_i = t_{cr}$ is initiation time of aggressive chemical, k and η are parameters determined by the regression analysis of available limited data (Akgül & Frangopol, 2004a; Czarnecki & Nowak, 2008; Kayser & Nowak, 1989; Sharifi & Paik, 2011).

The empirical expression and probabilistic approach to determine the damage function of the structure totally depends on the existing experimental data. Three different corrosion parameters were determined considering the environmental condition, such as Rural, Urban and Marine. Rural areas imply the smallest corrosion risk while the Marine areas have the highest corrosion risk (Czarnecki & Nowak, 2008). Change of corrosion rate over time was derived for three different environmental conditions as shown in Figure 11 using Equation (8) and regression parameters given in by Sharifi & Paik (2011) and (Biondini & Frangopol, 2016).

The area of concentration of high corrosion on a steel girder bridge was determined by field survey as shown in Figure 12. Corrosion was observed over the web near the support and ¼ of the web at the middle of the span with the bottom flange of the section (Kayser & Nowak, 1989). Section loss in the web of the steel girder was higher in the support, which was critical to shear strength of the girder. Therefore, reliability of the bridge was determined both considering shear at the supports and bending over the girders.

5.2. Life-cycle reliability analysis

Structural damages can be assumed as progressive deterioration of materials and components. The member level deterioration is specified by means of time-variant damage indices $\delta = \delta(t) \in [0;1]$. $\delta = 0$ represents no damage while $\delta = 1$ represents the full damage. Damage index d is related to the deterioration parameter, which represents corrosion penetration in the metal and mass loss of the girder. Geometrical properties of the damaged cross-section area are determined considering the damage index versus time.

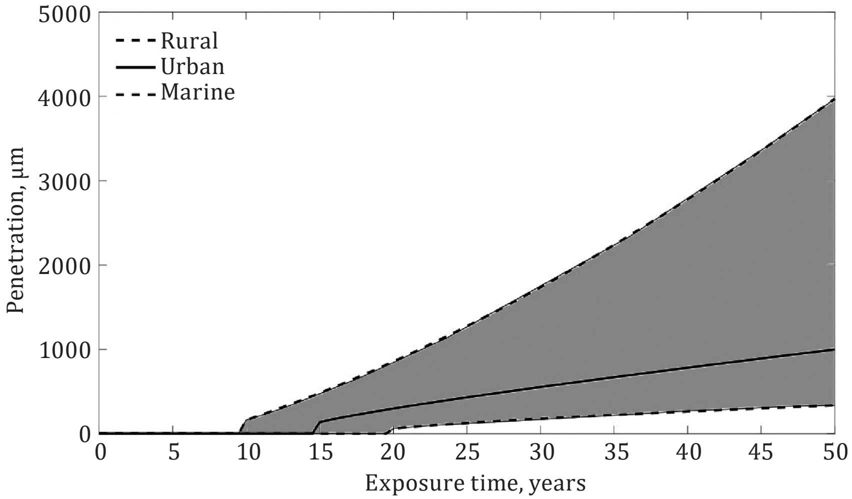


Figure 11. Corrosion penetration versus exposure time

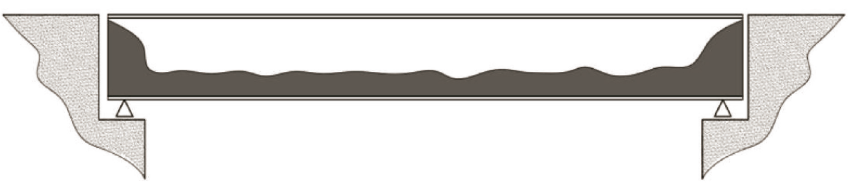
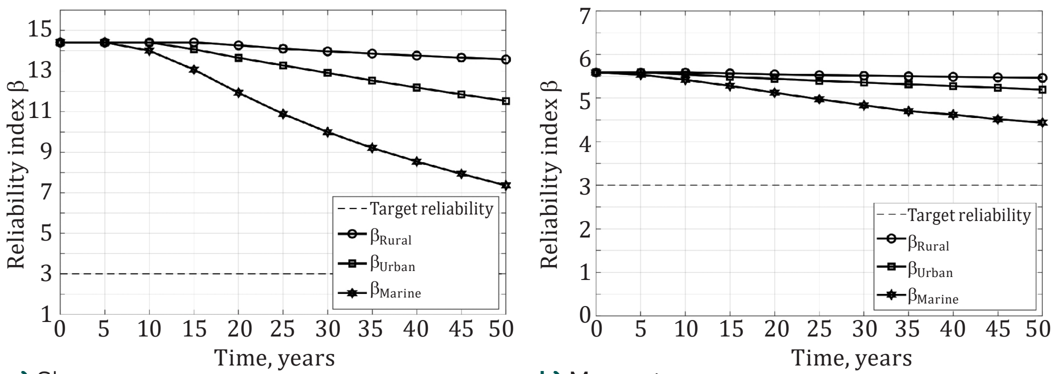


Figure 12. Typical location of corrosion on steel girder bridge (Kayser & Nowak, 1989)

In this study, the life-cycle reliability of the bridge considered was determined for both bending along the girder and shear at the girder supports (see Figure 13). Significant decrease in the shear strength capacity of the bridge was observed over time, as the girder web was more affected by corrosion. Since deflection controls are effective in the main girder design, the section heights increase significantly and the shear capacity of the main girder becomes much higher than the required shear capacity. For this reason, the reliability index value of the main girder under the effect of shear force was calculated far above the limit value. However, due to the concentration of corrosion losses in the support area, this reliability index value suffers significant losses throughout the service life of the structure in corrosion-affected regions. It was observed that also the bending capacity of the steel girder bridge decreased significantly in the marine environment. In the study it was assumed that no painting and maintenance activities were performed after the bridge was installed. The results also showed the importance of the maintenance activity for steel girder bridges, especially in the high corrosive environments.



a) Shear **b)** Moment

Figure 13. Life-cycle reliability analysis of the bridge

Conclusions

A simulation-based reliability analysis for main plate girders of steel railway bridges has been presented in this study. Bending moment capacity of the main girders of the bridge was determined considering statistical material and cross-sectional properties, as well as lateral torsional buckling. Dead and train loads were simulated and finally reliability of the bridge was determined. For this purpose,

- FE model of the compression part of bridge girders was created, and incremental compression load was applied proportional to moment load distribution on the girder to determine the buckling load of the compression sections. Sections experienced the yielding stress without buckling. Therefore, load-carrying capacity of the main girders of the bridge was taken to be equal to their plastic moment capacity.
- The statistical properties of the steel material were determined based on the existing information and specimen test results using the Bayesian approach. Furthermore, decreases of the posterior distribution standard deviation increased the reliability index of the bridge.
- The reliability of the bridge was determined considering the limit analysis of the main girders based on the positions of plastic hinges. A very similar load multiplier distribution and reliability index were simulated for sections of the main girders.
- As a result of the sensitivity analysis, it was concluded that one thousand simulation was enough to determine the reliability of the bridge with reasonable accuracy.
- The reliability index of the bridge was determined as $\beta = 5.57$, which was greater than $\beta = 3.0$ calculated according to UIC 2006 allowable stress design approach.
- Life-cycle reliability analysis of the bridge was performed under bending and shear loads. The results showed that deterioration velocity of the corrosion increases over time and becomes more critical while the bridge reaches the end of its service life.

As a result of the analysis it was observed that corrosion significantly reduced the bending capacity of the main girder of the steel bridge over time. However, significant differences in the corrosion rate due to environmental effects show the necessity of conducting site-specific studies to make the bridge maintenance and repair activities more economical. Determining the strength losses that each bridge will be exposed to separately will allow carrying out more economical and reliable maintenance and repair activities. In addition, it has been demonstrated that consideration of the potential environmental effects during the design of the bridges is essential to ensure economical and safe service of the bridge throughout its lifetime.

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