



Probabilistic failure assessment of steel eyebars by considering multi-phase nonlinear corrosion model

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ABSTRACT

Corrosion measurements were performed on the 175-years-old historical Széchenyi Chain Bridge within the last four decades; most lately under its reconstruction process in 2022. Measurement results are evaluated, and the findings encompass the structural corrosion loss of the chain elements (eyebars). A multi-phase nonlinear corrosion model, with time-dependent Weibull distribution parameters, is developed for bridges over rivers in urban environment. The model is capable of considering the time prior to corrosion initiation corresponding to the start of corrosion protection coating failure. Prediction of structural integrity and performance of chain elements is crucial to ensure reliable operation of such a historical structure. Therefore, multiple scenarios of possible corrosion progression are analysed with and without maintenance to estimate the evolution of corrosion damage over time. Probabilistic finite element calculations are carried out to predict the probability of failure of the chain elements subjected to pure tension. Partial safety factor of Eurocode, determined through Monte Carlo simulations with a response surface, ranges from 1.16 to 1.41 for corroded elements, assuming no renewal of the corrosion protection. Renewal of the coating significantly reduces probability, resulting in a partial safety factor of 1.17. Stochastic analysis indicates adequate load-bearing capacity of the chain elements for at least 20 years without significant renewal of the corrosion protection system.

1. Introduction

The historic Széchenyi Chain Bridge, a 175-year-old structure situated in Budapest, Hungary, holds a crucial position in the infrastructure of the capital city. The road and pedestrian bridge, spanning across the Danube River, handles significant daily traffic and serves as a primary connection between Buda and Pest. Its construction began in 1839 and finished in 1849, marking it as one of the most extensive chain bridges of its time, boasting a main span of 202.60 m (Fig. 1). Presently, only Hercílio Luz Bridge in Brazil (339 m) and Clifton Bridge in England (214 m) surpass its largest span.

Over the years, the historical bridge underwent several renovations, maintaining much of its original structural system. The original bridge, which lacked a stiffening girder, operated until 1913. Initially, a light-weight timber deck system caused noticeable aeroelastic vibration of the bridge deck. Subsequently, a redesigned suspension system (main girder) incorporated twenty-five carbon steel chain bars between each node. This alteration doubled the length of the chain bars, enhancing the load-bearing capacity and extending the distance between pins and

suspension bars from 1.8 m to 3.6 m. The total ironwork mass increased to 5200 tons with the addition of a new truss stiffening girder made from carbon steel with ultimate strength range of 480–560 MPa. Further enhancements included the installation of a reinforced concrete deck. Although the bridge was destroyed in 1945 during World War II, it was rebuilt without significant alterations to the structural system and was reopened for traffic in 1949. With more than 100-year-old chain elements and 70-year-old deck system, investigations and proof load tests were conducted in 2002 and 2018 by the Budapest University of Technology and Economics, Department of Structural Engineering to gauge the rotational capacity and structural behaviour of the pins between the chain elements. The bridge, without any structural changes since 1949, faced corrosion concerns, especially in the deck system, highlighting the need for a thorough reliability analysis. Notably, on-site corrosion measurements resulted in the development of a stochastic reliability assessment method in 2019, underscoring the need for urgent renovation of the deck system [1]. During the reconstruction between 2021 and 2023, while the deck system was replaced with an orthotropic steel deck, attention was given to the corrosion of chain bars, which can

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reduce tensile resistance and structural integrity, particularly in conjunction with reduced pin rotational capability.

The aim of the current paper is to develop a corrosion model based on measurements which can be used in subsequent numerical analysis to assess the probability of failure of eyebars considering different maintenance scenarios. The paper is organized as follows: Section 2 gives a review of state-of-the-art corrosion models. The strategy for predicting structural integrity and performance of chain elements consists of measuring corrosion losses of chain bars, which is presented in Section 3. It primarily showcases findings from corrosion measurements conducted in the last 40 years, most lately during the reconstruction process. In addition, a corrosion model is formulated in Section 4, to predict structural integrity and performance of chain elements, utilizing corrosion measurement data. Multiple scenarios of corrosion progression are discussed with and without maintenance in Section 5. The evolution of corrosion damage over time is estimated, while probabilistic finite element calculation is also carried out to predict the probability of failure of eyebars subjected to tension. Finally, Section 6 addresses further comments and discussion.

2. Literature review

The literature review introduces previously published results regarding corrosion measurements and models. Therefore, findings presented in this paper can be readily introduced and differentiated.

Metal loss due to corrosion is widely recognized as a significant issue affecting the durability of both new and existing steel structures during their service life. Adverse weather conditions, prolonged exposure to aggressive agents, insufficient maintenance, and paint film degradation contribute to corrosion in structural steel components. Corrosion degradation can potentially lead to sudden structural collapse. Numerous research dealt with the cause of corrosion development, and its consideration in design. Mainly, the effect of corrosion on material loss, material properties, and time-dependency need to be addressed within the static analysis. Disregarding corrosion in the analysis of structural integrity can significantly surpass the damage tolerance of affected steel structures, gradually deviating from their optimal design condition. This deviation can adversely affect their structural performance when not subject to regular maintenance or repair.

Numerous experimental studies have been undertaken to measure and quantify the damage caused by corrosive environments in the case of steel structures and characterizing the morphology of corrosion. Visual examination is the most often used form of corrosion inspection. However, it only detects surface corrosion. On the other hand, various non-destructive physical probing methods have been developed and implemented to describe and assess material loss and corrosion levels such as:

- ultrasonic thickness measurement method [2],
- gamma radiation [3],
- X-ray-based technologies [3,4],
- holographic interferometry [5],
- white light interferometry [6],
- thermography [7],

- scanning electron microscope stereo photogrammetry [8],
- optical three-dimensional scanning [9,10],
- atomic force microscopy [11].

Several laboratory experiments indicate a correlation between the mechanical properties of structural steel and corrosion-induced damage. It is attributed to local defects (notches) on corroded surfaces, leading to the initiation of fractures at lower stress levels. Characteristics of stress-strain curves heavily depend on corrosion damage, location, size and depth of dislocations [12]. According to Kim et al. [9], tensile tests indicate that yield strength and ultimate strength of structural steels remain relatively unaffected by corrosion when considering reductions in geometric dimensions. Conversely, the ultimate strain exhibits a nearly linear decline with the increase of corrosion damage. In addition, several research [9,13] pointed out the disappearance of yielding plateau, or reduction in its length, in the case of severe degradation due to extensive corrosion damage. Various corrosion models exist for the assessment of mechanical characteristics in corroded steel elements. On the other hand, corrosion models have an inevitable role in subsequent structural analysis. For instance, finite element analysis can be used for predicting the residual load-bearing capacity of corroded steel elements [1,10,14–16]. Corrosion models vary in complexity and have been tailored to different levels of analysis:

- effective thickness (t_{eff}) estimation methods for a specific corrosion state and time:
 - o Matsumoto et al. [17]:

$$t_{\text{eff}} = \min(t_{\text{mean}} - \text{width}) \quad (1)$$

- o Muranaka et al. [18]:

$$t_{\text{eff}} = t_{\text{mean}} - 0.7S \quad (2)$$

- o Appuhamy et al. [16]:

$$t_{\text{eff}} = 0.2 t_0 + 0.8 t_{\text{mean}} \quad (3)$$

- o Kim et al. [9]:

$$t_{\text{eff}} = t_{\text{mean}} - S \quad (4)$$

In the expressions, $\min(t_{\text{mean}} - \text{width})$ is minimum mean thickness, S is standard deviation, t_0 is initial thickness, and t_{mean} is mean thickness.

- time-dependent empirical corrosion models:
 - o linear model [19]:

$$d(t) = A + Bt \quad (5)$$

where d is average corrosion depth, t is time, A is initial corrosion loss, and B is to be determined from regression analysis of experimental data.

- o power model [19–21]:

$$d(t) = At^B \quad (6)$$

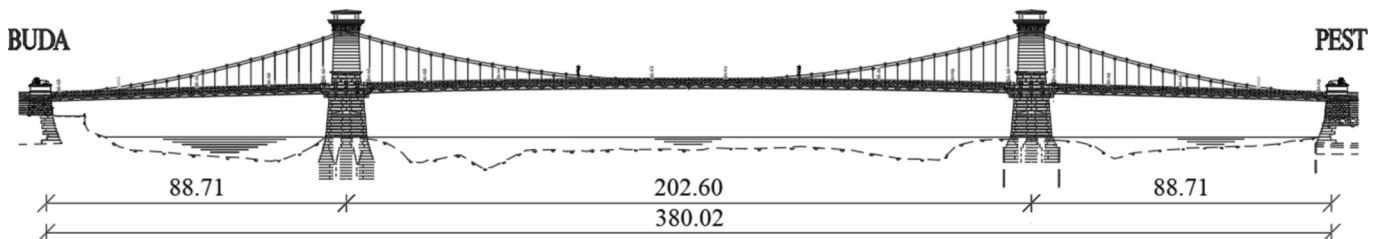


Fig. 1. Side view of the Széchenyi Chain Bridge in Budapest, Hungary with the main dimensions [m].

where d is average corrosion depth, t is time, A is corrosion loss after one year of exposure, and B is to be determined from regression analysis of experimental data.

- o 3-parameter Weibull function by Paik and Kim [22] for maritime environment:

$$d(Y_e) = \frac{\alpha}{\beta} \left(\frac{Y_e}{\beta} \right)^{\alpha-1} \exp \left[- \left(\frac{Y_e}{\beta} \right)^\alpha \right] \quad (7)$$

where d is corrosion depth or pit depth, α is shape parameter, β is scale parameter, while Y_e is exposure time in years after the breakdown of the coating and can be calculated as $Y_e = Y - Y_c$, Y is the age of the steel structure or specimen, and Y_c is the corrosion protective coating life. The formula assumes that there is no corrosion damage until the end of coating life. On the other hand, the model includes the change of corrosion depth and its probability density over time and in the function of actual corrosion state.

- o multi-phase nonlinear model by Garbatov et al. [23] for maritime environment:

$$d(t) = \begin{cases} 0, & t \leq \tau_c \\ d_\infty \left[1 - \exp \left[- \left(\frac{t - \tau_c}{\tau_t} \right) \right] \right], & t > \tau_c \end{cases} \quad (8)$$

where d_∞ is the long-term corrosion depth (or corrosion wastage), $d(t)$ is the corrosion wastage at time t , τ_c is the time prior to corrosion initiation (corresponding to the start of corrosion protection coating failure, if any); and τ_t is the so-called transition time duration.

- o modification of Garbatov model [21]:

$$d(t) = \begin{cases} 0, & t \leq \tau_c \\ d_\infty \left[1 - \exp \left[- \left(\frac{t - \tau_c}{\eta} \right)^\beta \right] \right], & t > \tau_c \end{cases} \quad (9)$$

where aforementioned parameters are the same, η and β are additional model parameters for more flexible curve-fitting capability. They could be determined using regression analysis.

- time-dependent phenomenological corrosion model by Melchers [19] which includes the following effects:
 - o initial corrosion,
 - o diffusion-limited corrosion controlled by corrosion products and microorganic growth,
 - o aerobic activity,
 - o and anaerobic activity.

Nevertheless, the aforementioned corrosion models were developed by considering some minor simplifications, and their application limits are quite specific. In practice, using regression analysis based on available measurement data remains essential for an accurate understanding of a given corrosive environment and structural configuration. In the context of historical steel bridges, the development of a corrosion model presents an opportunity for a significant academic contribution.

3. Corrosion measurements

The paper showcases plate thickness measurements conducted in 1986, 2002, and 2022 on the Széchenyi Chain Bridge, followed by a comparative statistical data evaluation. The mostly corroded chain elements are examined, and changes in corrosion rate over time are analysed. The most suitable density function is identified for the dataset and its parameters are determined to estimate the corrosion rate. On the other hand, it permits an estimation of corrosion state over the

upcoming years and decades, considering various maintenance scenarios.

3.1. Equipment

In 1986, a specialized measuring instrument, for gauging the thickness of the chain elements, was developed at the Budapest University of Technology and Economics (BME), Department of Structural Engineering. The gauge, equipped with seven sensors in its head, measures seven parallel lines along the surface of chain plates. Strain gauge resistors are arranged in a temperature-compensated bending configuration as a half-bridge, with two active strain gauge elements aligned to detect axial strain due to bending of the thin sheets with rollers. One strain gauge is mounted on the top sheet, and the other on the bottom sheet. As the thin plates with rollers expand, the strain gauges on the sheets experience bending. Each gauge factor, corresponding to a half-bridge sensor, is calibrated using reference plates to determine the thickness of eyebars. This unique instrument was employed to conduct the measurements, as depicted in Figs. 2–3, alongside the chain plates. The complete measurement setup, from sensors to software, consists of (i) uniaxial strain gauges with a nominal resistance of 350 Ω , (ii) an HBM Canhead amplifier system (Fig. 2), (iii) an HBM MGCplus data acquisition system, and (iv) a laptop for real-time visualization and data storage on a hard drive, using CatmanEasy software.

Measurements are conducted at eight locations (4 chains $\times$ 2, since top and bottom chains are measured as well) on chain plates after rust removal with sandblasting. The chain plates under analysis, comprising 11 measured eyebars, are located within the anchorage chambers, where the highest level of corrosion loss has been observed (Fig. 4). The measured areas span 1.5 to 2.0 m from the eyebar head along the parallel section of the chain elements. In prior measurements, the same chain elements were analysed, but the instrument could only offer data at specific points. During the most recent assessment, the newly integrated data acquisition system enabled continuous data collection along a designated line, providing uninterrupted measurements. The specific measurement zones and lines are shown in Fig. 5.

3.2. Measurement results

Fig. 6 shows the measurement results within the 2-m-long measurement zone for a particular chain element (Buda side, inflow side, lower chain element) comprising eleven eyebars. The plotted results encompass the measured thickness values from all seven measurement lines, which are also plotted in Fig. 7 for one specific eyebar. Measurements are taken in both forward and reverse directions along the chain elements resulting in two overlapping data series for each specific measurement line (Figs. 7–8). These measurements are deeply analysed and evaluated. Minimum, maximum and average thickness values are calculated and statistically assessed in all cross-sections. The corresponding results are presented in Fig. 8. Measurement results for all the $8 \times 11 = 88$ eyebars are evaluated similarly. The minimum and average thickness values are studied and compared between 1986 and 2022. Findings from the three thickness measurements are outlined below. The average thickness results are yielded as minimums of the recorded average thicknesses in each cross-section. Fig. 9 illustrates the measurement results, where the horizontal axis depicts the duration in years, and the vertical axis represents the percentage of the remaining corroded cross-sectional area (A_{corr}) compared to the nominal value (A_{nom}). Within the figure, the eight lines correspond to the eight chain elements, and abbreviations in the legend denote the specific terms used for identifying each chain element. The first letter in the three-letter abbreviations is either 'B' or 'P' which identifies the Buda or Pest side, respectively. The second letter denotes the inflow ('I') or outflow ('O') side of the Danube river, while the third letter identifies the top ('T') or bottom ('B') chain element (two independent chain elements are under each other on the bridge forming the whole chain system).

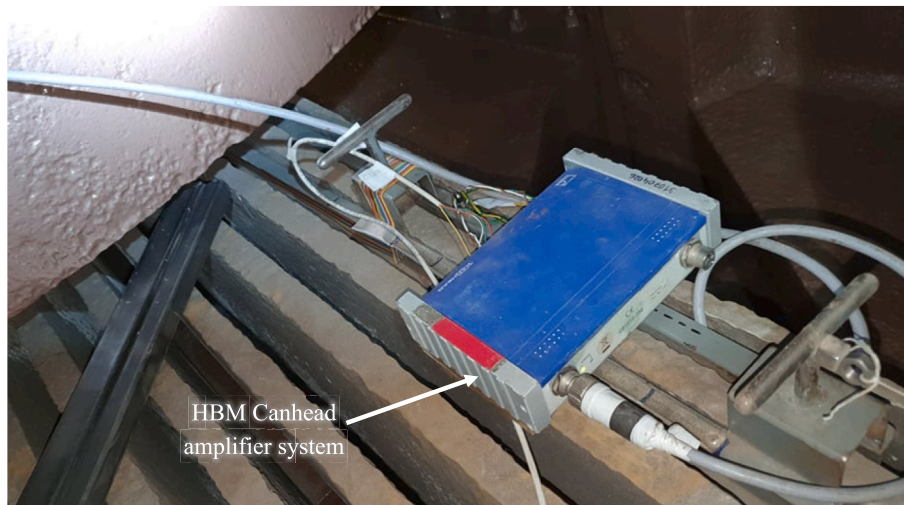


Fig. 2. Corroded chain plates and measuring instrument with amplifier system.

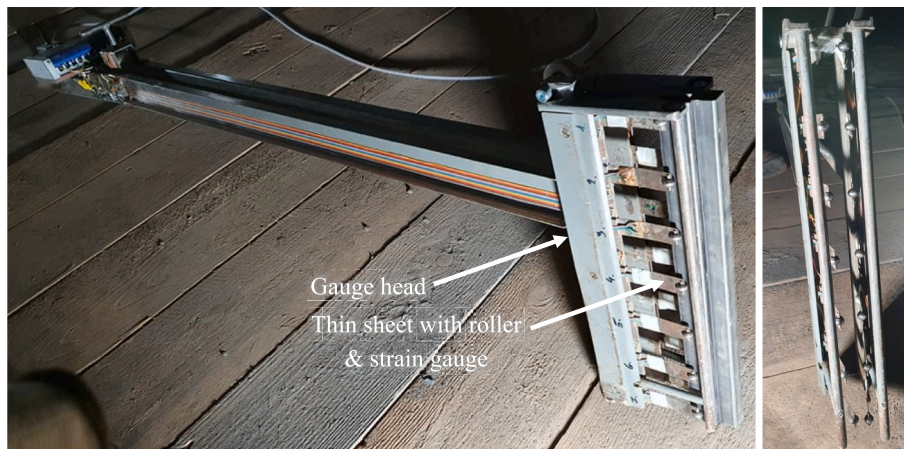


Fig. 3. Chain thickness measuring instrument developed by the BME Department of Structural Engineering.

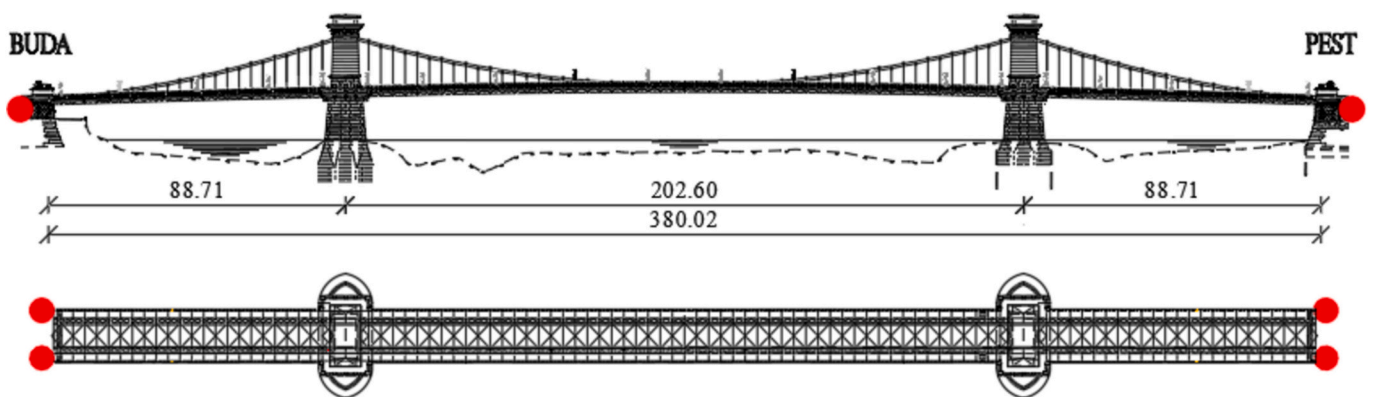


Fig. 4. Corrosion measurement locations.

The graph in Fig. 9 shows a consistent increase in corrosion within the plates over the years. The substantial loss of sections is particularly notable in the chain plates compared to the measurements in 2002. Based on the data obtained in 2022, the remaining cross-sectional area is less than 85 % of the nominal value for nearly all chain elements. The only exceptions are the top and bottom elements on the outflow side of the Buda side. The residual cross-sectional area to be taken into account

in the structural verification varies between 79 and 90 % (material loss is 10–21 %) for the eight chain elements measured in the vicinity of anchorage. Upon comparing the results, it is also evident that corrosion was least observed in 1986 and it is most pronounced in 2022. Related to all three measurements, the inflow side of the Buda side consistently displays the highest corrosion levels, whereas the outflow side of the same side exhibits the least corrosion. However, certain chain segments

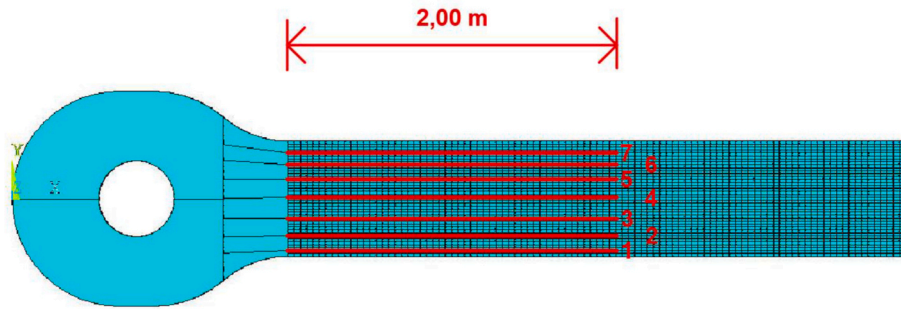


Fig. 5. Schematic of the measurement zone and measurement lines labelled from 1 to 7.

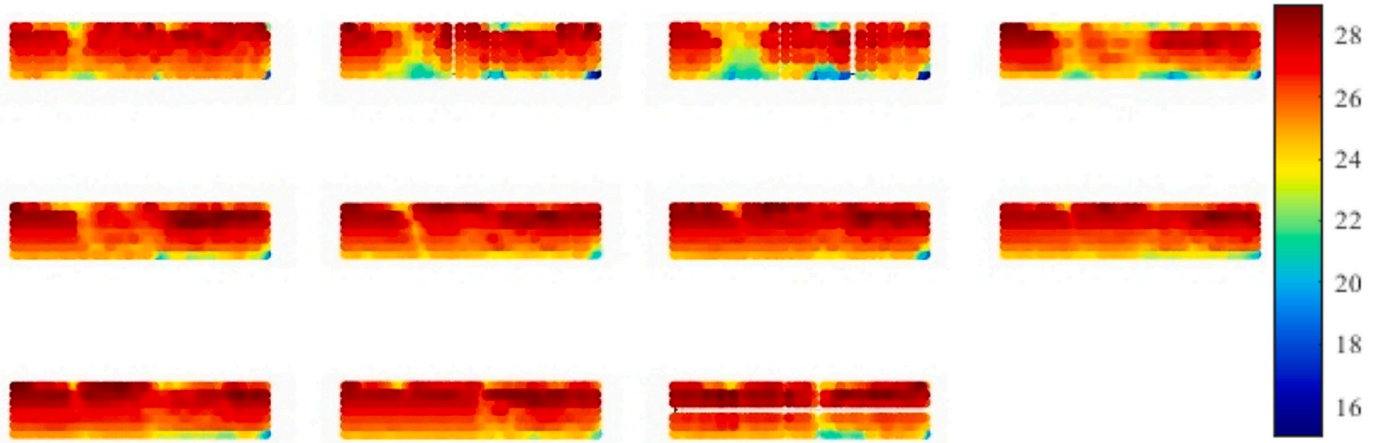


Fig. 6. Measured thickness fields [mm] for one particular chain element with eleven eyebars – Buda side, inflow side, lower chain element in 2022 (reference thickness is 29 mm).

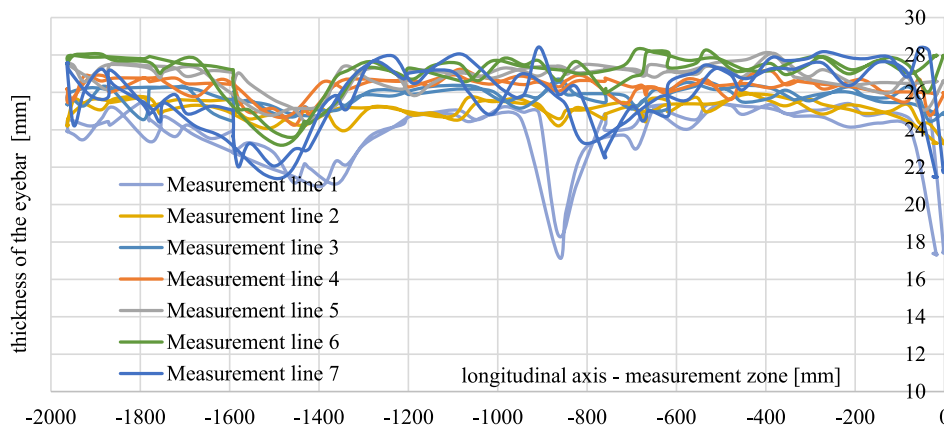


Fig. 7. Corrosion measurement results for one specific eyebar – Buda side, inflow side, lower chain element – first eyebar in 2022.

exhibit irregularities in their data trends. This divergence may stem from the differing measurement methodologies used. While prior tests in 1986 and 2002 focused on predefined points, the assessment in 2022 involves a continuous scan of the chain plate surfaces. Previous assessments might have overlooked the smallest cross-section within the interval, thus potentially affecting the accuracy of the calculated average values. Notwithstanding these uncertainties, the corrosion trends, as depicted in the plotted average values across all eight chain elements, are notably discernible. The observed trends from the comparison indicate that this dataset is suitable for statistical estimation of corrosion rates. In the initial stage of the statistical assessment, the measured data is utilized to derive the mean amplitude values for each chain element.

These amplitudes represent the disparity between the nominal thicknesses and the measured thicknesses. Considering the nominal thickness of a chain plate as 29 mm, the reduction in thickness over time delineates the diminished chain plate thicknesses attributed to corrosion. The statistical evaluation incorporated the average amplitude from all measured 88 plates, demonstrating the evolution in time and escalating corrosion rates. While in 1986, the majority of corrosion loss were below 1.5 mm, by 2002, most locations doubled in depth compared to 16 years prior. The last data from 2022 highlights the imperative for the reconstruction, exhibiting an average plate loss of 4–5 mm.

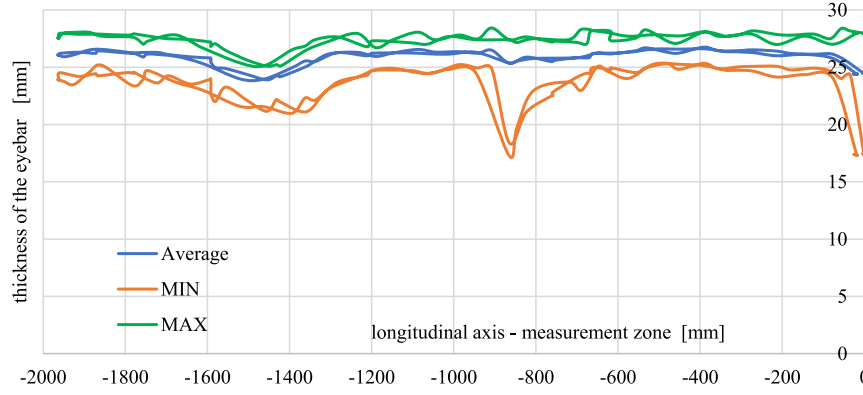


Fig. 8. Minimum, maximum and average thicknesses for one specific eyebar – Buda side, inflow side, lower chain element – first eyebar in 2022.

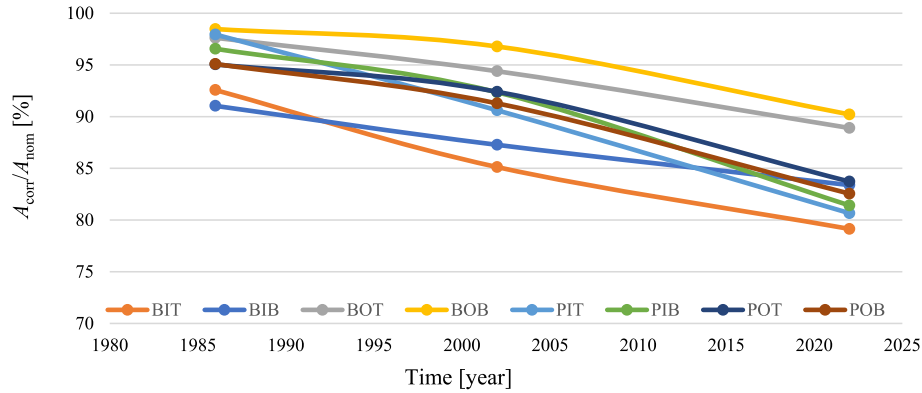


Fig. 9. Average values of reduced cross-sections due to corrosion compared to nominal values.

4. Novel corrosion model development

The subsequent objective was to employ a distribution function that represents the trend of the obtained graphs, aiding in a more systematic data evaluation. The EasyFit program is used to fit density functions to the data and subsequently rank them via the Kolmogorov-Smirnov test. Upon analysing the function fitted to the dataset, the Weibull distribution (type III extreme value distribution) exhibited the most favourable performance and demonstrated a robust fit based on the Kolmogorov-Smirnov test. It aligns well with the findings in existing literature [22] to model the time-dependent corrosion of steel plates. The Weibull probability density functions fitted to the histograms are shown in Fig. 10. The horizontal axis of the plots represents the values of the corrosion depths, i.e., overall corrosion losses measured on the two opposite surfaces of the eyebars, while the vertical axis represents the normalized probability distribution. Parameters of the fitted density functions are given in Table 1. Shape parameter α and scale parameter β are plotted as a function of time, and a function to each of the data series are fitted in Fig. 11a. The processed measurement data shows an accelerating trend of corrosion; therefore, exponential functions are used to examine the time-dependence of corrosion:

$$\alpha(t - \tau_c) = 1.162 \times \exp[0.032(t - \tau_c)] \quad (10)$$

$$\beta(t - \tau_c) = 1.3846 \times \exp[0.0373(t - \tau_c)] \quad (11)$$

where t is time in years and $\tau_c = 1986$ years is the time prior to corrosion initiation. In 2022, a new corrosion protection coating was applied on the steel structure. The atmospheric corrosivity category according to the EN ISO 12944-2 is C5 for the urban area with high humidity and aggressive atmosphere, almost permanent condensation and high pollution. The protective paint system consists of (i) a priming coat

containing aluminium pigment, (ii) epoxy binder as intermediate coat and (iii) a polyurethane topcoat. The nominal dry film thickness is 350 μm . The effect of renewing the coating is evaluated by assuming the corrosion rate starts from the same value as in 1986. Hence, for subsequent analysis of the chain system (Fig. 11b), τ_c is set to 2022 in this case. Additional terms $\alpha_{re} - \alpha_c$ and $\beta_{re} - \beta_c$ are introduced to the equations to ensure continuity, where α_c and β_c represent the fitted Weibull parameters at $\tau_c = 1986$ years, and α_{re} and β_{re} are the corresponding fitted parameters at the time of corrosion protection renewal (i.e., first, determining parameters by the equations above with the former τ_c is requisite in the case of corrosion protection renewal). Renewal of corrosion protection is also assumed in 2038 in Fig. 11b (with τ_c set to 2038).

Based on the measurement results and their statistical evaluation, the following multi-phase nonlinear corrosion model, with time-dependent Weibull distribution parameters, is developed for bridges in urban environment over rivers:

$$d(t - \tau_c) = \begin{cases} 0, & t \leq \tau_c \\ \frac{\alpha(t - \tau_c)}{\beta(t - \tau_c)} \left(\frac{t - \tau_c}{\beta(t - \tau_c)} \right)^{\alpha(t - \tau_c) - 1} \exp \left[- \left(\frac{t - \tau_c}{\beta(t - \tau_c)} \right)^{\alpha(t - \tau_c)} \right], & t > \tau_c \end{cases} \quad (12)$$

where d is corrosion depth or pit depth considering corrosion losses measured on the two opposite surfaces of the plates, t is time in years, τ_c is the time prior to corrosion initiation, while α and β are time-dependent shape and scale parameters, respectively. The formula assumes that there is no corrosion damage until the end of coating life τ_c .

It is emphasized that the limitations of the proposed corrosion model are the following:

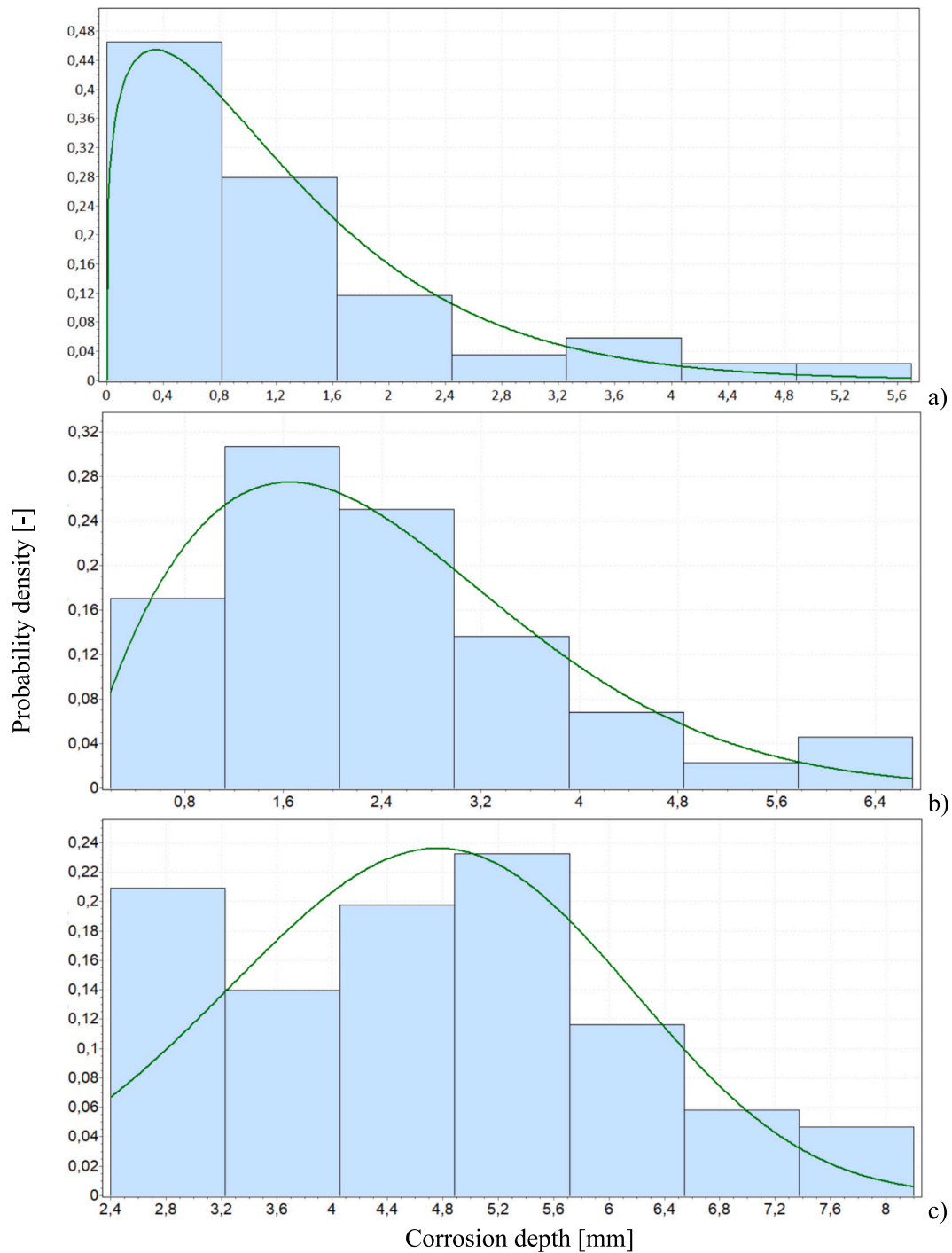


Fig. 10. Weibull density functions fitted to measurement data in year: a) 1986, b) 2002 and c) 2022.

Table 1

Parameters of the Weibull density function based on measurements.

Parameter/Year	1986	2002	2022
α	1.2302	1.7489	3.8445
β	1.3362	2.6801	5.1502

- the influence of expected variations in air temperature, relative humidity, pollutant concentrations, etc. due to climate change [24,25] are not considered in the long-term prediction of corrosion losses,

- the effect of coating renewal is evaluated by assuming that the corrosion rate is identical with the previously recorded rate from measurements (taken between 1986 and 2002 in this study),
- applicable to bridges located in industrial-urban areas with high humidity and aggressive atmospheric conditions, similar to those in Budapest, Hungary.

5. Maintenance scenarios and probabilistic failure assessment

During the investigation, the progression of corrosion impacting chain plates of the bridge is studied using two fundamental scenarios. Firstly, an assessment delves into the hypothetical corrosion damage

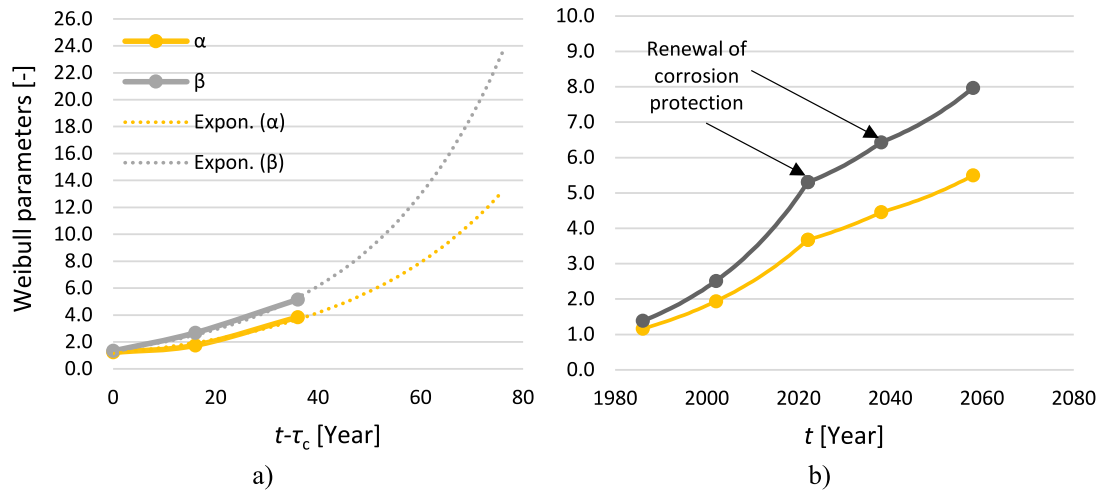


Fig. 11. a) Exponential functions fitted to Weibull distribution parameters (without additional protection), and b) two-phase functions considering renewal of corrosion protection in 2022 and 2038.

development whether the bridge had not been coated with new corrosion protection in 2022. Conversely, the second scenario investigates the present state, accounting for the application of new corrosion protection on the steel structure.

Density functions are produced based on Eqs. 10–11. The mode of fitted Weibull density function, representing the most frequent corrosion depth, is 4.8 mm for corrosion losses in 2022 (Fig. 12a), which is around ~17 % (4.8 mm/29 mm). It becomes 7.4 mm, 10.9 mm and 16.0 mm for 2032, 2042 and 2052, respectively. The nominal thickness of a chain plate is 29 mm. Therefore, the accelerated corrosion would have resulted in very high cross-sectional loss and the failure of chains without any renewal of corrosion protection. On the other hand, repetition of renewing the coating (Fig. 12b) every 16–20 years can effectively reduce corrosion rate and extend the life of the bridge. Renewal of coating in 2022 and 2038 and evaluated in 2038 and 2058 leads to most frequent corrosion depths of 6.1 mm and 7.7 mm; corresponding Weibull distribution parameters are shown in Fig. 11b.

It is crucial to highlight that chains are experiencing a decrease in their overall resistance due to corrosion. The developed corrosion model is utilized in probabilistic finite element calculations as well to predict the probability of failure of chain plates subjected to pure tension. The Probabilistic Design System (PDS) module of ANSYS [26], a general-purpose finite element software is applied for performing probabilistic calculations regarding load-bearing capacity. Probabilistic design variables and the corresponding distribution functions and parameters are summarized in Table 2 based on the recommendations of JCSS [27],

Table 2
Probabilistic design variables.

Probabilistic input variable	Distribution function	Distribution parameters
Initial plate thickness t	Gaussian	$t_{\text{mean}} = t_{\text{nom}} = 29 \text{ mm}$ $\sigma_t = 0.05 \times t_{\text{mean}}$
Plate width B	Gaussian	$B_{\text{mean}} = B_{\text{nom}} = 365 \text{ mm}$ $\sigma_B = 0.005 \times B_{\text{mean}}$
Corrosion loss (i.e., reduction of thickness)	Weibull	see Eqs. 10–12
Young's modulus E	Lognormal	$E_{\text{mean}} = 210,000 \text{ MPa}$ $\sigma_E = 0.03 \times E_{\text{mean}}$
Poisson's ratio ν	Lognormal	$\nu_{\text{mean}} = 0.3$ $\sigma_\nu = 0.03 \times \nu_{\text{mean}}$
Yield strength f_y	Lognormal	$f_{y,\text{mean}} = 295 \text{ MPa}$ $\sigma_{f_y} = 0.07 \times f_{y,\text{mean}}$
Ultimate strength f_u	Lognormal	$f_{u,\text{mean}} = 529 \text{ MPa}$ $\sigma_{f_u} = 0.04 \times f_{u,\text{mean}}$
Ultimate strain ϵ_u	Lognormal	$\epsilon_{u,\text{mean}} = 0.20$ $\sigma_{\epsilon_u} = 0.06 \times \epsilon_{u,\text{mean}}$

corrosion measurements and the developed corrosion model. In 2015, statistical evaluation of previous tensile tests, from 1913 and 1948, was carried out at the BME Department of Structural Engineering; the mean yield strength $f_{y,\text{mean}}$ and mean ultimate strength $f_{u,\text{mean}}$ were determined by laboratory tests, which are also considered in the current analysis. Geometrically and materially nonlinear analyses (GMNA) are carried out considering (i) large deflection effects, (ii) quad-linear

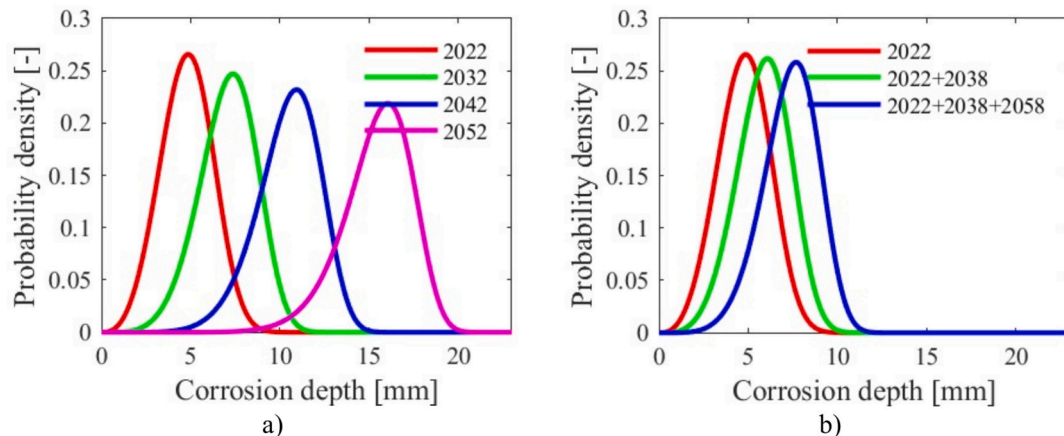


Fig. 12. Weibull density functions: a) assuming no corrosion protection, and b) renewal of corrosion protection coating in the denoted years.

kinematic hardening material model according prEN 1993-1-14 [28] to accurately capture yield plateau and strain-hardening behaviour of the steel material, and (iii) geometrical deviations, i.e., corrosion pits, on the chain element surfaces due to localized corrosion (Fig. 13) as indicated by the thickness reduction outlined in Table 2 based on the proposed corrosion model. Single chain plates, subjected to pure tension, are analysed using 8-node solid elements (SOLID185). Standard contact behaviour is modelled between the pin and the eyebear with TARGET170 and CONTA173 elements assuming friction coefficient of 0.2. Nominal thickness t is 29 mm, nominal width B is 365 mm at the parallel zone, while the pin diameter D is 240 mm. The total length L of the eyebear is 7999 mm. Half of an eyebear is modelled with corresponding boundary conditions representing plane of symmetry. A fine mesh is used with average element size of 15 mm ($\sim B/24$), while six elements are applied through the thickness of the plate. The finite element mesh is shown in Fig. 13; mesh scheme with 50,692 finite elements and 60,364 nodes is verified based on the simulated load bearing capacity. Typical normal stress distribution in the corroded eyebear and the failure mode are shown in Fig. 14.

The probabilistic analysis is performed using Monte Carlo simulation with random Latin hypercube sampling (MCS-LHS in Fig. 15) which method ensures to have independent samples without accumulation of clusters of samples by having a memory of previous simulations. On the other hand, extreme ends of distribution functions are also considered in sampling process. A total of n samples is used in the simulation dividing each random input variable into intervals of equal probability ($p = 1/n$), while random sampling is carried out within each interval. The quantity of simulation loops, i.e., samples, needed for a Monte Carlo simulation to model a quadratic response surface depends on the number of random input variables. The determined sample size for every simulated maintenance scenario is set at 81, aligned with the number of probabilistic design variables, adhering to the advanced analysis guide of the software. Response surfaces are fitted for the output variable, i.e., load amplification factor $\alpha_{ULS} = N_{FEA}/N_{Ed}$ for Ultimate Limit State (ULS), where N_{FEA} is the finite element analysis (FEA)-based resistance and $N_{Ed} = 1809$ kN is the design tensile force in ULS. Forward-stepwise-regression model with linear and quadratic terms including cross-terms is used with confidence level of 0.95 for filtering out irrelevant terms of the model. In addition, Monte Carlo simulations are performed with $N = 10^6$ simulation loops on the derived response surfaces (MCS-RS in Fig. 15) in MATLAB environment. It ensures fast calculation and evaluation of the α_{ULS} using approximation functions.

A total of seven scenarios are analysed: one without any corrosion as a reference, four without any maintenance and two with the renewal of corrosion protection. The latter are demonstrated in Fig. 12. Results of the stochastic analyses are summarized in Table 3. Characteristic and design load factors are determined for $P_f = 5\%$ ($\alpha_{ULS,k}$) and 1% ($\alpha_{ULS,d}$), respectively. The relevant values are determined from the probability density functions determined based on the Monte Carlo simulation results. After determining the probability of the α_{ULS} exceeding the specified limit, which is set as 1, the probability of failure P_f is computed as a measure of reliability. One-sample Kolmogorov-Smirnov tests are performed at the 5% significance level, and it turned out that MCS-RS data do not come from a standard normal distribution. Therefore, reliability

index $\beta = \alpha_{ULS,mean} / \alpha_{ULS,std}$ is not applicable in the analysed cases. Nevertheless, mean and standard deviation values of load amplification factors, $\alpha_{ULS,mean}$ and $\alpha_{ULS,std}$, respectively, are denoted in the table to describe the changes contributed to corrosion and renewal of protection coating.

Design value could be evaluated in general by using the characteristic value and $\gamma_{M0} = 1.0$ for tensile resistance of the cross-section (Fig. 14b) according to EN 1993-1-1. However, it is shown that partial factor associated with the uncertainty of the resistance model, the ratio of characteristic and design values, $\gamma_{Rd} = \alpha_{ULS,k} / \alpha_{ULS,d}$ varies between 1.16 and 1.41, assuming no corrosion protection, by statistically evaluating the MCS-RS-based sample data. Nevertheless, renewal of corrosion protection coating in the denoted years effectively reduces corrosion rate and probability of failure compared to those analysed cases where no corrosion protection is assumed between 2022 and 2052. Partial factor γ_{Rd} is only 1.17 when renewal of the coating is repeated in 2022 and 2038. The present findings highlight the significance of using different partial factors for heavily corroded steel elements; further investigations, measurements, and calculations are needed in enhancing the partial safety factor for corroded members. Thus, partial safety factor applicable for design γ_M^* (Fig. 16a) is evaluated using Eq. 13 as follows:

$$\frac{N_{Rk}}{\gamma_M^*} = N_{FEA,d} = \alpha_{ULS,d} \bullet N_{Ed} \rightarrow \gamma_M^* = \frac{N_{Rk}/N_{Ed}}{N_{FEA,d}/N_{Ed}} = \frac{A \bullet f_{yk} / N_{Ed}}{\alpha_{ULS,d}} \quad (13)$$

where $N_{Rk} = A \bullet f_{yk}$ is the characteristic resistance based on analytical calculation using the cross-sectional area of the uncorroded eyebear ($A = 29 \text{ mm} \times 365 \text{ mm}$) and the characteristic yield strength $f_{yk} = 262.3 \text{ MPa}$ is applied assuming lognormal distribution (Table 2). Partial safety factors applicable for design are demonstrated in Fig. 16b for all the analysed cases. The reference model, without corrosion, results in $\gamma_M^* = 1.01$, which is very close to $\gamma_{M0} = 1.0$. The figure unambiguously shows the impact of new corrosion protection layers by keeping γ_M^* close to the value evaluated in the year of reconstruction; however, a slight increase of $\sim 15\%$ until 2058 is evaluated. On the other hand, partial safety factor dramatically increases in scenarios without any corrosion protection.

Since the historic Széchenyi Chain Bridge holds a crucial position in the infrastructure of the capital city, reliability class RC3 and consequence class CC3 are required based on EN 1990 indicating target value $\beta = 5.2$ (target probability of failure $P_{ft} = 9.9 \cdot 10^{-8}$) regarding 1-year lifetime. The reference model, without any corrosion, is assumed to be statistically evaluated and checked for a 100-year reference period and the target probability of failure is determined using Eq. 14 based on Eq. (C.3) demonstrated in Annex C of EN 1990:

$$P_{ft} = \Phi(\beta_{t_{ref}}) \leq [\Phi(\beta_1)]^{t_{ref}} \quad (14)$$

where $\beta_{t_{ref}}$ is the reliability index for a reference period of t_{ref} years, β_1 is the reliability index for one year based on the reliability class, Φ is the cumulative distribution function of the standardised normal variable. The equation yields $P_{ft} = 9.9 \cdot 10^{-6}$ for a 100-year design lifetime; the reference model without any corrosion considered in the finite element analysis exhibited sufficient reliability ($P_f < 10^{-8}$) to withstand the

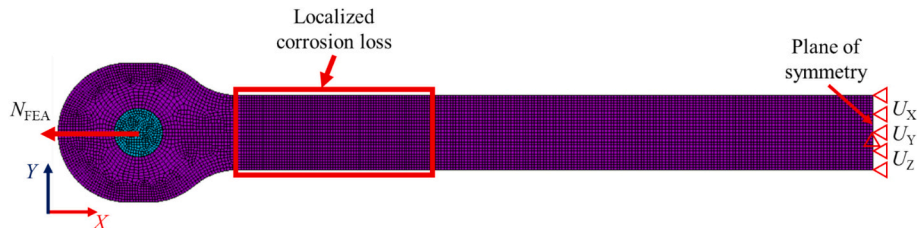


Fig. 13. Finite element model of a chain plate subjected to pure tension with localized corrosion.

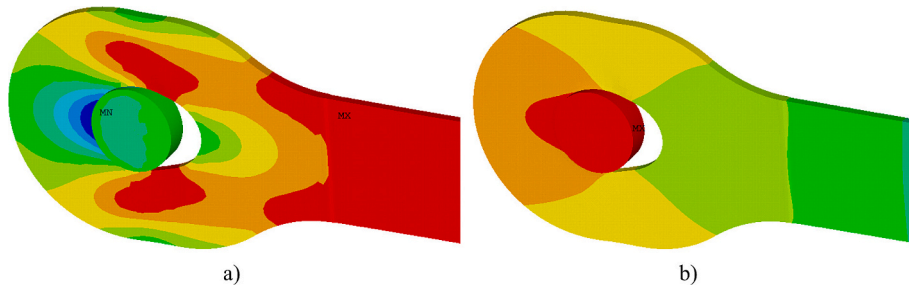


Fig. 14. Typical normal stress distribution, and b) failure mode in corroded chain plate using finite element analysis.

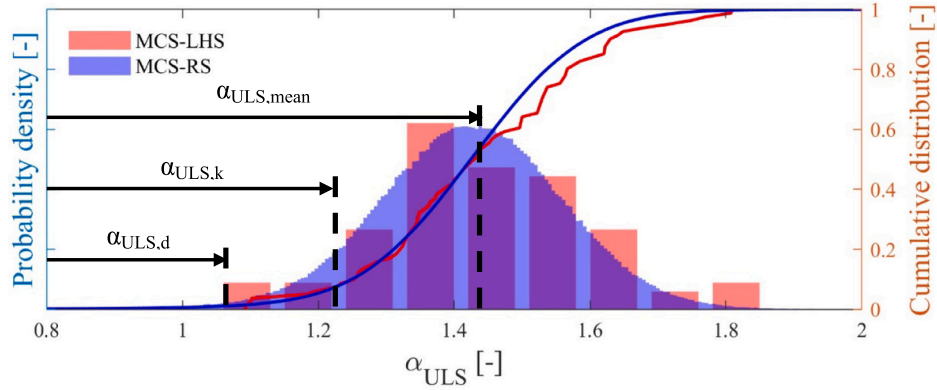


Fig. 15. Monte Carlo simulation-based empirical probability density functions and cumulative distribution functions using Latin hypercube sampling (MCS-LHS) and response surface (MCS-RS) – year 2022.

Table 3

Results of the probabilistic analysis based on MCS-RS.

Parameter/Year	Without corrosion (0)	No corrosion protection				Renewal of corrosion protection	
		2022	2032	2042	2052	2022 + 2038	2022 + 2038 + 2058
Characteristic value	1.7270	1.2372	1.0948	0.8854	0.5810	1.1741	1.0846
$\alpha_{ULS,k}$ (5 %)							
Design value	1.5171	1.0638	0.9279	0.7161	0.4113	1.0067	0.9290
$\alpha_{ULS,d}$ (1 ‰)							
$\alpha_{ULS,mean}$	1.7281	1.4427	1.3052	1.1028	0.8043	1.3773	1.2896
$\alpha_{ULS,std}$	0.0932	0.1255	0.1309	0.1381	0.1444	0.1254	0.1301
γ_{Rd}	1.14	1.16	1.18	1.24	1.41	1.17	1.17
γ_M^*	1.01	1.44	1.65	2.14	3.73	1.52	1.65
t_{ref} [year]	100	73	83	93	103	89	109
P_f [-]	$<10^{-8}$	0.00013	0.0072	0.2332	0.9068	0.00082	0.0077
P_{ft} [-]	$9.9 \cdot 10^{-6}$	0.0015	0.0017	0.0019	0.0021	0.0018	0.0022

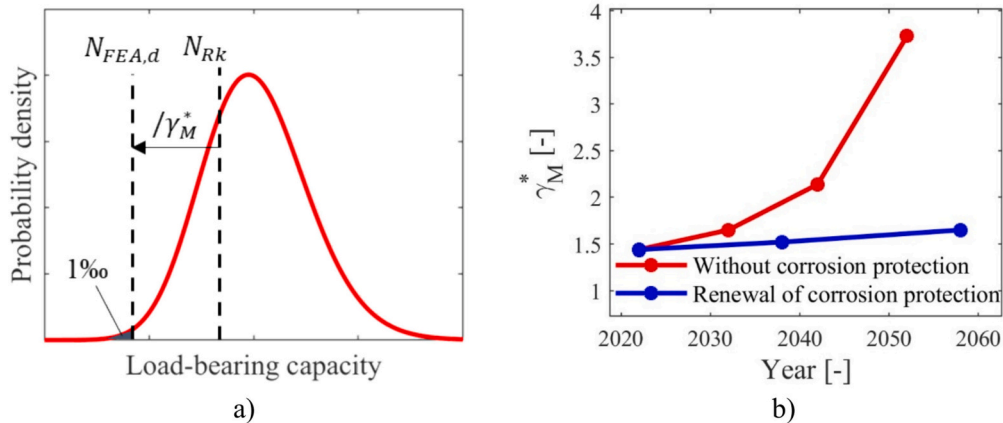


Fig. 16. a) Explanation of partial safety factor γ_M^* and b) results of the analysed scenarios.

applied loads. However, the standard is generally used for design, not for existing structures. According to Sykora et al. [29], ISO 2394 provides a more suitable reliability differentiation for existing structures by factoring the costs of safety measures. It suggests that the target level of reliability should be determined based on a balance between consequences of failure and costs associated with safety measures. In addition, Sykora et al. [30] demonstrated the concept of individual risk provided in ISO 2394 which yields the relationship between target probability of failure P_{ft} and the conditional probability of occupant fatality P_1 :

$$P_{ft}(\text{per year}) \leq 10^{-6} \text{ per year} / P_1 \quad (15)$$

where $P_1 = 0.001, 0.01$ and 0.05 for consequence classes CC1, CC2 and CC3, respectively; thus, $P_{ft} \leq 2 \cdot 10^{-5} t_{ref}$ is used for CC3 in the actual case with reference period t_{ref} in years.

Reference period is calculated as the lifetime of the structure considering 1949 as the year of reopening for traffic after rebuilding the structure. It can be stated that corrosion damage of chain plates governs the failure mode and ultimate load level. According to the numerical simulation results, the eyebar without corrosion protection would not have met the requirements by 2032. The probability of failure would have reached the target value between 2022 and 2032. On the other hand, repeating the renewal of corrosion protection can extend the lifetime of the chain elements; probability of failure can be kept below the target value in the future as well. It is worth noting, load is handled as a deterministic value in this study, which could be analysed as probabilistic variable later to ensure the target reliability. Nevertheless, the reliability assessment can be refined by using results of subsequent non-destructive testing [31], such as proof load test [32] or structural health monitoring system [33]. Thus, statistical indicators derived from measurements could be implemented into a performance-based framework using Bayesian updating [34].

6. Conclusion

The paper presents measurement results of the 175-years-old historical Széchenyi Chain Bridge of Budapest. The major aim of the measurements was the investigation of corrosion grade of the chain system. Corrosion can significantly affect the structural health of the bridge and has clear impact on the further lifetime of the structure which had to be assessed by the Designers. Corrosion loss measurements were executed on the chains at the most corroded part of the bridge. These plate thickness measurements were similarly conducted in 1986, 2002, and 2022, following by a comparative statistical data evaluation which made it possible to assess the corrosion rate change of the bridge over time and perform probabilistic failure assessment using different scenarios. Based on on-site measurements and probabilistic finite element analyses, the following findings can be distinguished:

- based on corrosion measurements in 2022, it has been determined that the most frequent material loss is 17 %, while the largest measured corrosion loss is 21 % in a specific cross-section;
- a multi-phase nonlinear corrosion model, with time-dependent Weibull distribution parameters, is developed for historical structures operating in urban environment based on measurements between 1986 and 2022;
- the developed corrosion model permits an estimation of corrosion state of the bridge over the upcoming years and decades considering various maintenance scenarios;
- based on the Monte Carlo simulation results partial safety factors are determined for the specific chain element resistance calculation, which is compared to the partial safety factor of the Eurocode; the impact of the corrosion grade and corrosion protection on the calculated partial safety factors are also highlighted and the methodology to calculate the relevant values are presented in the paper;

- partial safety factor applicable for design γ_M^* is evaluated for all the cases; the reference model, without corrosion, results in $\gamma_M^* = 1.01$, while it increases in time when corrosion protection degradation is considered;
- results also prove, the value of γ_M^* would dramatically increase in scenarios without any corrosion protection.

Based on the findings of the research, the aim of subsequent investigations is implementing the results of other non-destructive tests, such as proof load tests, and refined load models into a performance-based framework using Bayesian updating in order to ensure long-term reliability for the historical Széchenyi Chain Bridge.

CRedit authorship contribution statement

B. Kövesdi: Writing – review & editing, Supervision, Resources, Project administration, Methodology, Funding acquisition, Conceptualization. **D. Kollár:** Writing – original draft, Visualization, Software, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **B. Szabó:** Writing – original draft, Visualization, Data curation. **L. Dunai:** Writing – review & editing, Supervision, Project administration, Conceptualization.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

Data will be made available on request.

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