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DIAGNOSTICS OF HISTORICAL STEEL BRIDGES: THE CASE OF THE VALENCIAN RAILROAD NETWORK

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Abstract

Exposure to aggressive environmental agents, aging *and extreme weather events can seriously affect the performance of historical steel bridges. In the last decades, the need for diagnostic and monitoring of existing steel bridges became a huge widespread issue, as some recent catastrophic events pointed out (Mississippi River bridge, Minneapolis, Minnesota 2017; Kinzua Bridge State Park, Pennsylvania 2003). In this context, the need for monitoring and risk analysis of existing structures became an important concern for holders and administrators that are responsible for the construction operability and user safety. In particular, the issue of bridge inspections recently arose in the Valencian region (Spain) since a large number of railway riveted steel bridges were built between the end of the19th and the beginning of the 20th Century. Today, after more than 100 years of existence, many bridges need repairing or strengthening interventions due to changes in service requirements or pathologies. This paper proposes a methodology for the identification of an analytical failure tree related to the structural response of the historical steel bridges located in the Valencian railroad network considering different critical phenomena and extreme events. Firstly, a suitable investigation plan has been developed to identify the typological, geometric, material and conservation characteristics of a case study. Subsequently, specific diagnostics (i.e. identification of the critical elements under traffic loads) is performed through a numerical model of the bridge calibrated by using the results of on-site static and dynamic tests. Once the critical elements are identified, some failure scenarios considering corrosion, fatigue and extreme events (e.g. earthquakes) are hypothesized in order to forecast consequence scenarios and achieve a failure tree of the bridge.*

Keywords: Historical Steel Bridges, Diagnostics, Failure Tree, On-Site Test, Calibration.

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1 INTRODUCTION

Exposure to extreme natural events, aggressive environment and aging can compromise the structural performances of existing historical steel bridges jeopardizing the safety of users and of the bridge itself [1,2]. The dissemination and knowledge gained from recent failures have contributed to show the need for safe constructions, especially for critical infrastructure systems, with a high patrimonial value. Indeed, in the last decades, some recent catastrophic events pointed out the necessity of diagnosing and preventing serious damages and consequent collapse. Some of the recent catastrophic collapses of bridges occurred in Minnesota 2017 (Mississippi River bridge) and Pennsylvania 2003 (Kinzua Bridge State Park). Consequently, the perception of the society about historical steel bridges safety has changed radically, leading to the need of defining reliable tools for investigation, diagnostics mapping particular natural disasters [3,4].

The problem is highly widespread also in Europe, where *steel railway bridges* characterize the constructive heritage of the past century and are spread throughout the Countries even in small towns [5]. Indeed at European level, one of the main priorities of Horizon Europe (regarding secure societies) is to face the challenge of "*enhance the resilience of our society against natural and man-made disasters, ranging from the development of new crisis management tools to communication interoperability, and to develop novel solutions for the protection of critical infrastructure*". Beyond this, *bridges collapse* is also dealt at a global level. United Nations's Agenda 2030 identifies in the section "*sustainable cities and communities*" the following important topics: a) "*provide access to safe, affordable, accessible and sustainable transport systems*", and b) "*reduce the number of deaths and the number of people affected and substantially decrease the direct economic losses relative to global gross domestic product caused by disasters*."

In the related literature, there are numerous studies concerning the investigation and diagnostics of existing old structures [6,7,8]. These studies confirm that the design and construction criteria of the past do not consider a number of key principles to ensure the structures durability. In addition, the current service loads on the bridges are usually greater than those expected in the design phase of the past, due to the modern increase of people and freight transportation. As a result, there are many inadequacies and weaknesses in the existing steel bridges, particularly in their conceptual designs and structural details to resist cyclic loads.

A study of the ASCE Committee on Fatigue and Fracture Reliability [9] showed that eighty to ninety percent of failures in steel structures were related to *fatigue* and fractures. Vibrations, transverse horizontal forces (e.g. wind), distortions of member cross-sections, localized and diffused defects (such as corrosion damages) represent concurring causes of fatigue damage [10]. In addition, old bridges are usually characterized by their riveted connections and this is actually one of the weakest points of this type of steel structures. In this field, some research studies developed extensive tests concerning fatigue strength of rivets under shear loads demonstrating the insufficient performance level of this type of connection [11].

In this context, it is fundamental to investigate the contemporary presence of different typologies of damages in the *steel railway bridges* in order to understand the possible arising of concatenated failures. Some authors investigated how to include different typologies of damages in Finite Element Method (FEM) models [12]. In addition, fault trees are useful tools to get a complete overview of the possible arising of concatenated failures of different typologies (*fatigue*, *corrosion* damages, *seismic* damages) [13]. On the other hand, there are few attempts to connect a fault tree with a numerical FEM model. Typically the fault trees are heuristic approaches that only provide a decision support linked to the monitoring of the bridge. Research

and approaches to achieve the fault tree from the results of a structural model are missing in related literature.
This paper proposes a methodology to derive a fault tree from a FEM model (in this case calibrated by static and dynamic tests performed on-site). Hereafter, this novel graphical tool is named Analytical Failure Tree (AFT). Compared with the classical fault tree, the proposed AFT is not based on a heuristic approach, whereas it is achieved by numerical analysis and forecasting scenarios. In addition, this novel tool offers the advantage not only to numerically identify the component level failures (basic event) that cause the system level failure (top event) but also provides the possibility of displaying the failure events on a timeline. This is a useful tool also for stakeholders which can easily predict the remaining service life of the considered structure.
The proposed procedure is applied to a riveted steel bridges located in the Valencian railroad networks (Valencia region, Spain).
Particularly, the approach is articulated in three phases:

Firstly, a suitable investigation plan provides useful information (typological, geometric, material and conservation data) to characterize the bridge and its elements. The first step of the investigation plan is devoted to acquire geometrical information of the bridge. The second step provides useful historical data clarifying the history of the past structural interventions. The third and last steps of the investigation plan regard static and dynamic load tests performed directly on the bridge where a tailored monitoring system is installed to collect displacements in critical locations and stress/strains in significant structural components.

Secondly, starting from the information retrieved from the investigation plan, a FEM structural model of the bridge can be realized. The FEM model is calibrated exploiting the results in terms of strains on the monitored members by static and dynamic tests performed on the bridge and modal properties achieved by on-site measured vibrations.

Thirdly, phenomena such as corrosion, fatigue and other possible extreme events can be included and simulated with the FEM analysis. In this third phase, the procedure to derive the analytical failure tree of the bridge is explained. Moreover, by means of the FEM structural model simulating a different typology of damages and initial condition, it is possible to hypothesize different scenarios and consequently different analytical failure trees can be obtained.

2 THE HISTORICAL BRIDGES OF VALENCIAN RAILWAY NETWORK

In *Valencian region* a large number of *riveted steel bridges* were built to create the railroad network from the middle of the 19th century to the first decades of the 20th century when the use ofsteel in bridge construction was widespread.. After more than 100 years of existence of this railroad network, many bridges need repairing or strengthening interventions due to changes in service requirements or pathologies. In addition, since the steel bridges have been subjected over the years to an increasing number of train passages (which means a large number of load cycles from the beginning to nowadays), the *fatigue* phenomena could affect their remaining life. In particular, fatigue often leads to a brittle fracture that, joined with a typical low level of redundancy of these types of structures, produces a high risk of collapse. Figure 1 shows two examples of old steel bridges in the Valencian railroad network.
In the last decades, the need for diagnostic and intervention of existing structures became a huge widespread issue for holders and administrators that are responsible for the construction

operability and user safety [14, 15]. In this context, an analytical failure tree of the bridge can be of great help in evaluating the diagnostic and monitoring of the structure.

Figure 1. Two examples of old steel bridges in the Valencian railroad network: “Quisi” (left) and “Ferrandet” (right) bridges.

2.1 The case-study bridge

The proposed methodology to derive an analytical failure tree from a FEM model is applied to the “Quisi” riveted steel bridge located in the Valencian railroad networks (Valencia region, Spain). The “Quisi” bridge is a multi-span truss deck bridge connecting the towns of Alicante and Denia, built between 1913 and 1915, which currently is still in-service within the Spanish railway network.

3 THE INVESTIGATION PLAN TO THE CHARACTERIZATION OF THE BRIDGE AND ITS ELEMENTS

The investigation plan procedure is devoted to identifying the geometrical, typological, dimensional and material information to characterize the bridge and its elements. In addition, this investigation plan provides useful information to realize and calibrate an effective FEM structural model able to simulate the performance of the bridge under a given load condition.

A preliminary historical analysis of the bridge is performed in order to retrieve the historical documentation useful to correctly identify the construction technique and be aware of possible modifications after construction. After achieving the documentation, the geometrical and visual survey can complete the information to obtain the geometric inspection of the complete system. Finally, static and dynamic load tests are performed directly on the bridges. Consequently, the FEM structural model can be realized, specific modelling assumption can be carried out thanks to the investigation and a calibration can be achieved exploiting the result of the static and dynamic load tests.

3.1 The investigation plan step 1: geometrical information of the bridge

The first step of the investigation plan is fundamental to identify the exact geometrical dimension of all the elements of the bridge. Useful geometrical and technical design documents of the bridge are retrieved and combined with no site investigations. In addition, in this step, photogrammetric processes of images acquired by drones or LiDAR techniques [16,17], can be used to acquire geometrical data which can be directly used to generate FEM models. A brief description of the resulting geometric and constructive features of the bridge is reported as follows. The bridge exhibits six spans for a total length of approximately 170 m. The first two, and the last two spans exhibit an isostatic structural scheme with simply supported deck trusses (approximate length 21.00 m). Conversely, the central spans (the third and the fourth) are characterised by a continuous truss deck (hyperstatic) having a total length of 84 m. The truss deck is composed of two main truss beams, connected by transverse (i.e. floor) beams (which, in turn, sustain the stringers) and secondary lateral bracing systems at the level of the upper and lower chords.The two central spans present a continuous structural scheme, while the four lateral ones the scheme is isostatic.

The *sub-structure* system is constituted by 5 steel braced towers. The bracing system is arranged in panels, composed of X-bracing systems, considering the longitudinal and transverse direction of the bridge, respectively. At the top of the steel towers, 0.75 m-high beams are placed to absorb the gravity loads from the bearings. The legs are battened steel members: two built-up C-shaped parallel steel profiles are connected by steel plates, one per 0.85 m, creating an open-box cross-section. Rivets are used to connect the different steel elements together in each member. At the bottom of the steel towers, steel anchor bolts attach the legs to the masonry foundations. Figure 2 shows the geometrical drawing of the investigated bridge.

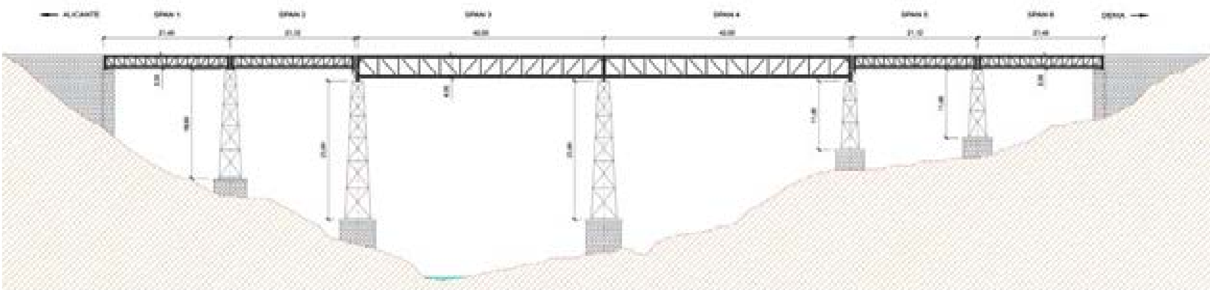


Figure 2: The geometrical drawing of the investigated bridge.

3.2 The investigation plan step 2: the historical analysis

After 100 years of useful life, the investigated bridge presents some structural criticalities related to corrosion and fatigue damages due to the operation time. In addition, the structure was designed and built according to the type of trains and railway traffic of the 20th century. In order to improve the structural safety of the bridge under the current conditions of use, in 2018 it underwent a substantial retrofit intervention. In this intervention, some reinforcing-plates were applied between the upper chords of the deck and the longitudinal members, to allow these elements to work together against the horizontal braking forces, in order to reduce the stresses in the cross beams. In addition, the dimensions of the upper and lower chord of the side members have been increased through the application of two additional bolted plate (8 mm-thick). Figure 3 emphasizes the intervention in the lower chord showed red in the photo (left part of the figure) and in a section of the structural detail (in the right).

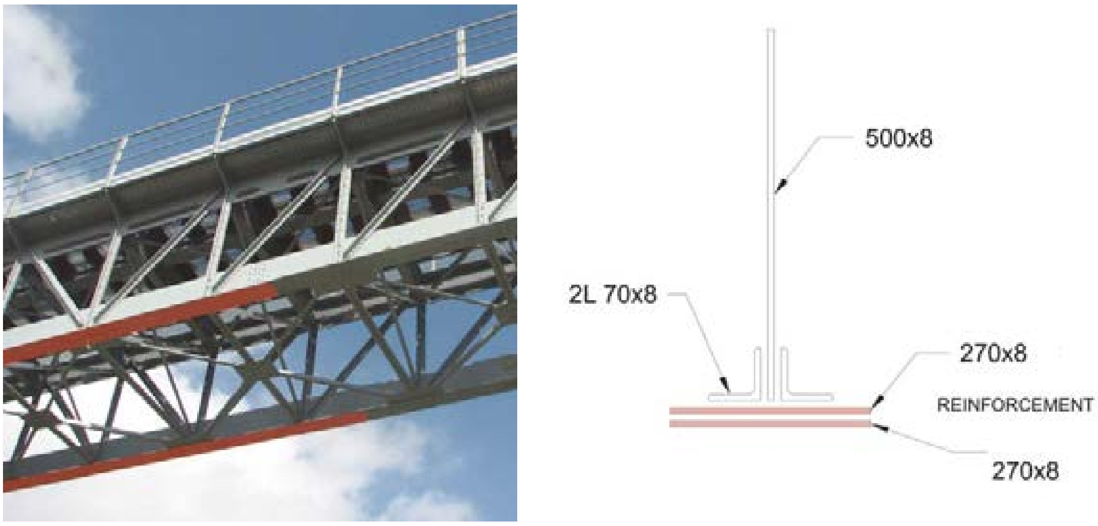


Figure 3: Example of the intervention in the lower chord.

The last important retrofit intervention regards the replacement of bearing devices. Pot bearings (i.e. confined elastomeric bearings) are placed on the top of the steel towers and on the masonry abutments, preventing relative displacement between the deck and the substructure members in the transverse direction, while allowing longitudinal thermal deformations. Shock transmitters are also present to ensure the transmission of longitudinal seismic forces from the truss deck to the supporting steel towers.

3.3 The investigation plan step 3: static and dynamic load test

Beyond the geometrical dimension, the second important result of the investigation plan regards the mechanical and structural characteristics of the bridge.

Some mechanical tests (according to EN ISO 6892-1) [18] are performed in order to identify the characteristics of the materials. All the tested samples showed consistent results in terms of both yielding and ultimate strengths (Table 1.)

Sample	Yielding strength [MPa]	Ultimate strength [MPa]	Elongation [%]
Quisi	271	399	32

Table 1: Quisi tensile test results.

A *static load* test is performed: a train composed of two wagons, representative of the real service loads, is placed at the middle of each span and the stabilized *maximum deflection* of the truss is measured. Furthermore, the deformations in some significant truss components are registered thanks to the numerous strain and displacement sensors (Figure 4). In addition, a dynamic test is performed, in which a moving train travels on the bridge (speed of 30 km/h) and the strains and stresses of significant members are evaluated. This latter test allows for measuring bridge dynamic parameters such as the *modes of vibration* and *natural frequencies* by using a set of accelerometers during the same test.

Figure 4: Static and load test on the bridge.

4 THE CALIBRATION OF THE STRUCTURAL MODEL

The FEM model is realized with the software SAP2000 (Computer and Structures INC (CSI). SAP2000 - Structural Analysis Program 2018) by exploiting the results of the described investigation plan. Figure 5 shows the structural model of the investigated Bridge. All the built-up members are modelled by single frame elements. The bearing devices connecting the trusses to the top beams of the towers are represented by linear *two-nodes links*. The elastic modulus of the steel is fixed at 210000 MPa and fully fixed foundations are considered. Rigid end zones are placed to model connection zones between the frame elements.

Figure 5: Structural Model of the investigated Bridge realised with SAP2000.

Some geometrical modelling assumption are necessary in order to reflect correctly the acquired information in the numerical model. In particular, the riveted connection beam-diagonal is modelled by using a fixed connection with rigid end offsets. Moreover, the isostatic span is assumed with the following external restraints: i) the fixed bearing modeled through pinned (i.e. fixed displacement) connection, ii) the movable bearing allowing a free longitudinal translation.

Figure 6 shows the connection between the isostatic spawn 2 (left), the hyperstatic span 3 (right) and the steel tower.

Figure 6: Connection between the isostatic spawn 2 (left), the hyperstatic span 3 (right) and the steel tower.

The structural model is calibrated in three steps: 1) identification of the fixity degree of connection between the diagonals and the chords, 2) calibration of the value of the gravity loads, 3) calibration of the model to simulate dynamic loads (the passage of the train).

4.1 Calibration with static and dynamic loads

The calibration is performed via sensitivity analyses considering the variability in some modelling parameters. *Firstly***,** the isostatic spans and hyperstatic spans are isolated from the model and are subjected to a load pattern representing the axle loads used for the static tests. A first calibration and sensitivity analysis concern the fixity degree of connection between the diagonals and the chords (rigid end offsets) provided by riveted connections; the frame mesh of the elements (element discretization) and Young modulus of the steel. This sensitivity analysis is carried out by hand, updating step-by-step the model and identifying the value of each parameter (or their combination) that best reflects the outcomes of the on-site tests. In this case, both the maximum deflection of the truss and the strain values are used for benchmarking the outcomes of the model.

Secondly, another sensitivity analysis is performed to calibrate the value of the gravity loads (considering also non-structural loads such as sleepers or rail profiles). For this aim, the results of a modal analysis performed using several mass values consistently with some assumptions about the non-structural loads are compared to the period of vibrations registered on-site. This process is repeated for one of the isostatic truss decks and the hyperstatic one.

Thirdly, a Moving Load Analysis is carried out in the SAP2000 environment simulating the dynamic load test performed on-site. This analysis leads to further refining of the previously-fixed parameters to best reflect the local strains value of some selected elements.

4.2 Calibration results

The real on-site load test and the simulation performed via SAP2000 provides very similar results in terms of max deflection and first mode frequency. To provide an example Figure 7 shows that in the isostatic span the max deflection measured on-site is 5.5 mm and in the sensitivity analysis of the model, it ranges from 5.39- 6.06mm. Moreover, the first mode frequency is 8.79 Hz in the real test and ranges from 8.84 – 8.78 Hz in the sensitivity analysis of the model. These results of the sensitivity analysis demonstrate that the calibration is robust.

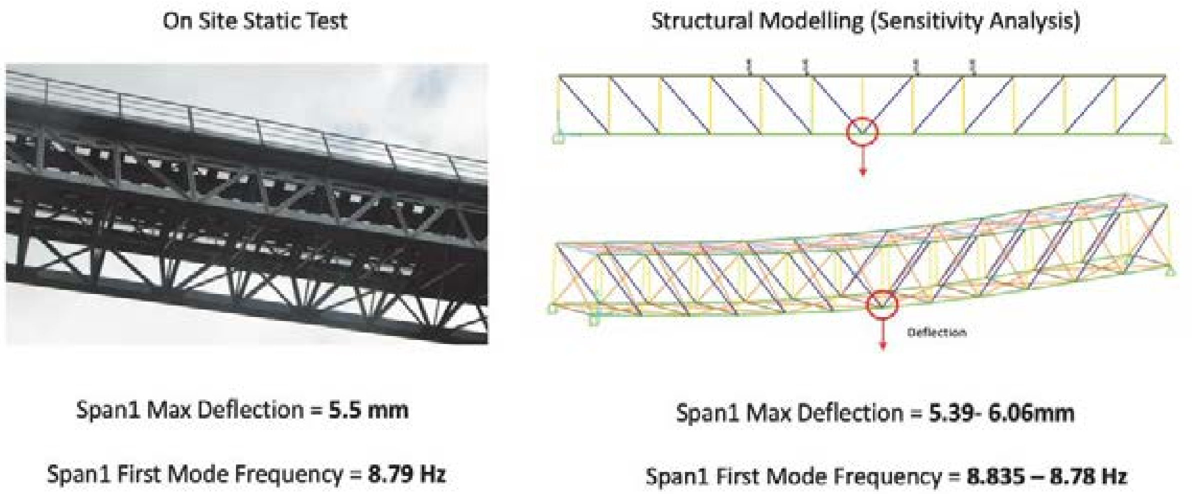
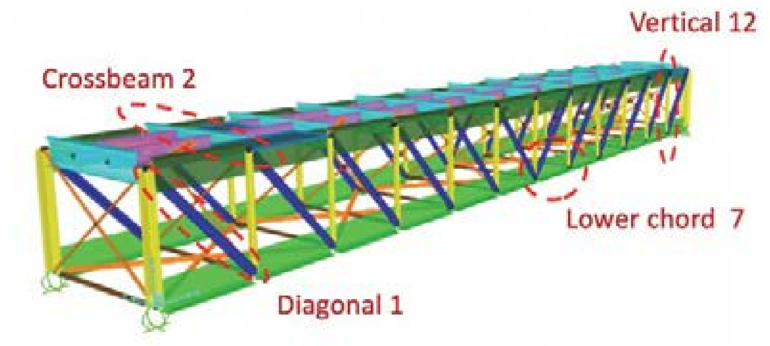


Figure 7: Static and modal test-based calibration of the isostatic Span 1

Beyond the max deflection and first mode frequency, all the strains sensors applied on the structural elements of the bridge are exploited for validating the calibration.
In particular, it is verified that the calibrated structural model (simulating the load tests) provides a deformation of the elements of the bridge compatible with that actually measured during the load tests (on which strain sensors are applied). In total, more than two hundred strain sensors are applied to the bridge, and consequently, numerous elements are available for validation. The comparison confirms the effectiveness of both the static and dynamic load simulation obtained with the calibrated structural model. Indeed, the ratio between the deformation obtained with the structural model and the deformation measured during the tests is on average equal to 0.923 (where 1 represents the perfect coherence among the two deformations). Figure 8 shows the comparison between the deformation of the elements of the Span 1 during the static and dynamic load tests, and the theoretical deformation achieved by using the SAP 2000 software.

In both in the static and load test, the only values that deviate from consistent behavior regards the cross beams. This situation occurs because cross beams take the load directly from the train. Consequently, the small differences of the model compared with the real bridge can cause overestimation or underestimation depending on the position of the train (load) on the bridge.



STATIC LOAD					DINAMIC LOAD				
Span	Element	SAP Deform	Test Deform	Ratio SAP/Test	Span	Element	SAP Deform	Test Deform	Ratio SAP/Test
Span 1	Lower chord 7	102.1	118.2	0.9	Span 1	Lower chord 7	89.7	107.2	0.8
	Vertical 12	-61.4	-65.5	0.9		Vertical 12	-89.6	-79.6	1.1
	Diagonal 1	63.7	53.1	1.2		Diagonal 1	83.1	69.9	1.2
	Crossbeam 2	2.0	10.0	0.2		Crossbeam 2	2.3	23.3	0.1

Figure 8: Comparison between the simulation (SAP 2000 software) and the real load test.

5 HOW TO GET AN ANALYTICAL FAILURE TREE OF THE BRIDGE

The calibrated model is the starting point to include different types of damages (e.g. corrosion, fatigue, natural hazards) in the analyses and achieve an analytical failure tree of the bridge. In this section, firstly it is explained how to include different damages in the structural model and secondly the procedure to achieve an analytical failure tree of the bridge is described. In order to have a complete overview of the possible damages of the bridge, different scenarios considering different combinations of corrosion, fatigue, earthquakes damages and different traffic volumes can be evaluated.

5.1 Evaluation of Corrosion Damage

The evaluation of the atmospheric corrosion is assumed on all the elements of the bridge according to the ISO 9224-2012 [19]. In particular, this approach assumes a uniform section reduction on the bridge elements. The speed of the steel corrosion depends on the type of steel and the aggressive environmental condition to which the element is exposed. In particular, the thickness reduction d is evaluated in function of the time of exposure according to the following equations:

$$\begin{cases} d_1(t) = r_{av} * t & for\ t < 10\ Years \\ d(t) = 10 * r_{av} + (t-10) * r_{lin} & for\ t > 10\ Years \end{cases} \tag{1}$$

Where $d_1(t)$ is the average depth of corrosion in the first 10 years of exposure, $d(t)$ is the average depth of corrosion when the exposure time is more than 10 years; r_{av} is the average corrosion speed (during the first 10 years of exposure), r_{lin} is the average speed of stabilized corrosion (after an exposure time of 10 years), and t is the Exposure time. Moreover, the r_{av} and r_{lin} are tabulated values indicated in the ISO 9224-2012.

The reduction of the cross-section area of the elements calculated with the Equation (1) can be easily implemented in the structural model by property modifiers to the cross-section elements defined in SAP2000.

5.2 Evaluation of Fatigue Damage

The evaluation of the fatigue phenomenon can be evaluated according to the Miner's Rule [20]. The linear Miner's damage equation is widely used in engineering for its effectiveness and ease of application. This mathematical formulation allows for achieving the damage D of a specific element subjected to cyclic loads as follows:

$$D = \sum_{i=1}^{k} \frac{n_i}{N_i} \tag{2}$$

where N_i is the fatigue life of the element under some stress levels typically determined by laboratory tests and n_i is the number of load cycles of a given amplitude applied on the element. Moreover, when D is equal to 1, the component fails.

The value N can be achieved by using the curve Stress – Number of cycle (S-N) proposed by proposed in Kühn et al. [21] also applied in the Eurocode 3. In particular, some laboratory tests are performed to identify the S-N curve of the investigated bridge elements.

A previous investigation to achieve the curve S-N was performed in the laboratory of the ICITECH [22]. In particular, an upper cross beam and a full-scale bridge span of the Ferrandet Bridge was tested. The span and the cross beam have the same geometry and similar characteristics of spans 2 and 5 of the Quisi Bridge. To this aim, the results are valid for both the bridges as discussed in [22]. During the tests, Linear Variable Displacement Transducers (LVDTs) and Strain Gauge (SG) sensors were used to capture the possible nucleation and propagation of fatigue cracks.

Figure 9 shows the curve S-N of the investigated components of the bridge. In addition, an example of the application of the curve is showed in the following.

Let us assume that the structural model shows that the diagonal 12 of the Span1 suffer a load cycle of an amplitude of $\Delta\sigma_c = 41$ N/mm^2. Consequently, thanks to the curve S-N it is possible to identify the amount of residual cycles before the component fails due to fatigue stress (that are approximately 10^7). It is worth noting that knowing the traffic on the bridge (number and typology of trains per day) it is possible to understand the remaining life of a component. The instant in time at which the failure occurs (considering a predetermined traffic condition) can be precisely identified.

The effects of fatigue phenomena can be accounted for in the numerical model, by appropriate reductions of the mechanical properties of the elements or by neglecting their contribution in case of fatigue failure. This operation should be performed by updating by-hand the model (e.g. no automatic procedure is available).

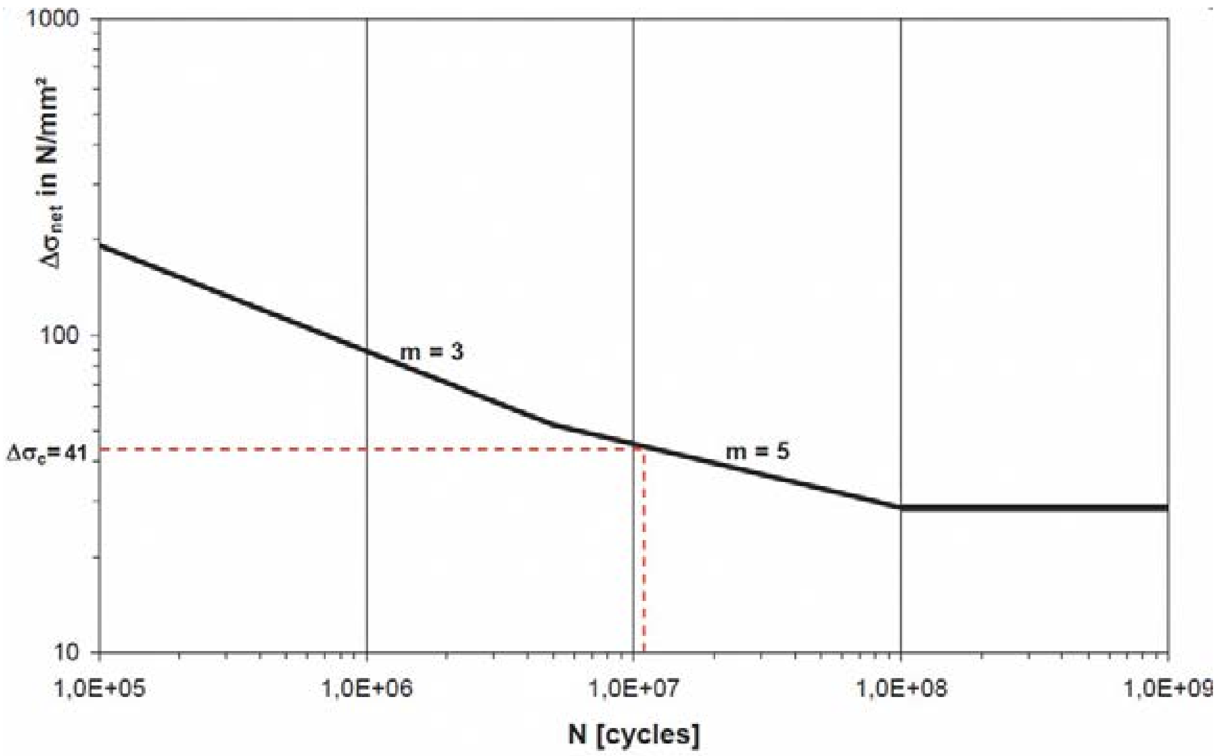


Figure 9: Curve Stress – Number of cicle for the components of the investigated Bridge: example of the application for the diagonal 12 of the Span1.

5.3 The evaluation of earthquake damage

Also the structural performance of the model under actions induced by natural hazards can be included in the analytical failure tree framework. As an example, in the case of seismic actions, state-of-the-art numerical analysis approaches (i.e. nonlinear static or nonlinear dynamic) can be performed to analyse the performance of the case-study structure under a given seismic action (i.e. code-based design action) or for a given earthquake scenario.

To this aim, a nonlinear numerical model should be adopted to predict the inelastic demand for each member during the seismic excitation. An appropriate modelling strategy should particularly consider the inelastic response of the substructure members which exhibit the highest probability to reach selected damage states, as proved in the large literature about the seismic response of multi-span bridges types [23,24]. To this aim, distributed- or lumped-plasticity strategies can be adopted. In the latter, plastic hinges should be used to predict the nonlinear axial response of bracing members (subjected to buckling or tensile yielding) and nonlinear axial-flexure response in the legs of the steel towers (Figure 10). Also, failures of the riveted connections or for shear should be considered. Appropriate recommendations about modelling and seismic analysis of truss bridges are reported in [25].

Nonlinear dynamic analyses are widely recognised as the most accurate strategies to predict the response of the case-study bridge under a given ground-motion shaking. However, such analysis approaches are very demanding in terms of computational efforts, particularly for complex structures such as steel truss bridges. Moreover, these analyses require models about the cyclic response of built-up steel members which are rarely available in the literature [26,27]. Conversely, nonlinear static procedures are less demanding with respect to nonlinear dynamic approaches. The accuracy of these simplified methods strongly depends on various assumptions (e.g. regular dynamic response) which can be not valid for typical steel truss bridges. However, according to [25], this simplified methodologies are effective for local analysis of substructure members, such as steel towers, to identify the corresponding ductility capacity and failure mechanisms.

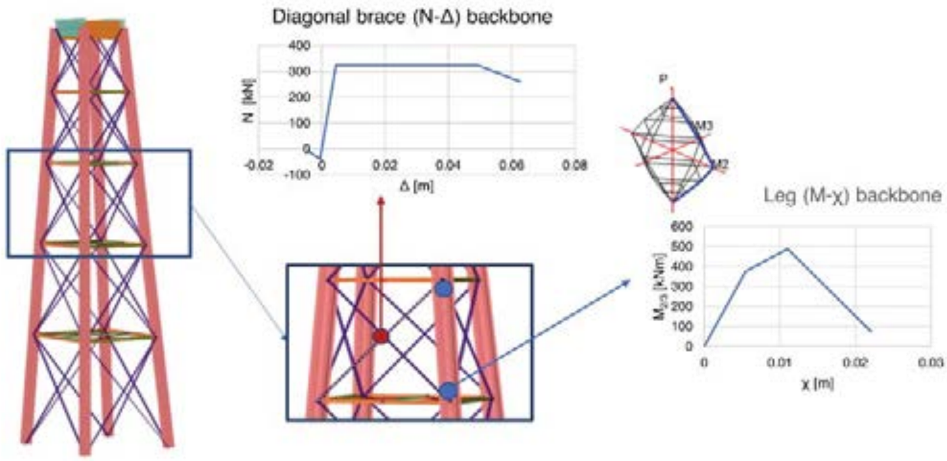


Figure 10: Modelling strategy (lumped-plasticity) for seismic response of supporting steel towers.

5.4 The proposal of the analytical failure tree

In this subsection, the methodological process to achieve an analytical failure tree of a bridge is discussed. More precisely, the analytical failure tree is intended as a graphical representation developed on a timeline to show when faults can occur according to the structural model. In addition, the analytical failure tree also shows the connections among failures. A failure can be isolated without causing other consequences. In other cases, the failure can create a cascade effect. The analytical failure tree is also useful to understand the possible occurrence of serious cascade effects causing rapidly a global bridge collapse.

The analytical failure tree is obtained with an iterative process that concerns the evaluation of the amplitude of the load cycles from every element of the bridge (from the FEM model) and the related evaluation of the fatigue damages (and concurring causes) over time. In particular, the process is explained in the following bulleted list:

1) Initially there are no damages and the dynamic analysis is performed. Consequently, the stresses of all the elements (or components) are obtained from the FEM model;
2) The elements that undergo significant load cycles (named critical elements) according to the curve "Stress – Number of cycles" are considered. Fatigue and other concurring causes (e.g. corrosion) are evaluated over time by using procedures described in subsections 5.1 and 5.2;
3) The first element that fails in the instant t_1 is identified;
4) The failure is shown in the analytical failure tree in the instant t_1 (indicating component and cause);
5) The contribution of the damaged element is implemented, updating the FEM model (updated in the instant t_1),
6) The new stresses of all the components (after t_1) are obtained from the FEM model subjected to a new analysis,

7) If no other elements have reached a limit state it is possible to continue with the next step. If other elements have reached a limit state, the analytical failure tree and the structural model are updated by considering all the elements failed in the instant t_1 and new stresses are evaluated (this step is repeated until the updating of the structural model does not cause other element reaching a limit state or a global failure is reached);
8) This phase is analogous to point 2. New fatigue damages and other concurring causes are evaluated over time;
9) The second element (or group of elements) fails in the instant t_2. If this second failure is related to the first one (instant t_1), then it is placed on the same branch of the fault tree. On the contrary, if the second element fails is not related to the first one a new independent branch is drawn.
10) The process is repeated until the instant t_n is greater than a preset year or a global failure is achieved.

More in details, the rules to understand if two consecutive failures belong to the same branch are specified in the following. Let us assume the presence of two failures (a first failure in the instant *a*, and a second failure in successive instant *b*). The conditions so that these two consecutive failures are positioned on the same branch of the failure tree are the following:

1) the two failures belong to the same span, or to two adjacent hyperstatic spans;
2) the element that undergoes the second failure (instant *b*) suffers an increase in load due to the firstfailure (this condition can be identified when the numerical model is updated).

If these two conditions do not both occur, it means that the two damages are independent, and it is right to represent them differently on two different branches in the fault tree.

It is worth noting that in the proposed application, the failure can occur for the *fatigue* and all the other phenomena can only speed up the damage. Indeed, the *corrosion damage* causes a section reduction, that consequently can produce a premature fatigue failure. The *seismic damage* creates small structural damages (due to the low seismicity of the region in question) which in some cases can aggravate the effects to the fatigue damage. Other possible damages can occur as a consequence of the redistribution of loads after a fatigue failure. Consequently, the proposed approach is in line with the assumption demonstrated in the study of the ASCE Committee on Fatigue and Fracture Reliability [9] assessing that the failures in steel structures were related principally to *fatigue* and all the other phenomena are concurring causes that can involve the reaching of a limit state.

5.5 The analytical failure tree and different scenarios of the investigated bridge

The proposed analytical failure tree can consider different aggressive phenomena including corrosion, fatigue and it can also be used to understand the consequences of earthquake damages. In addition, different scenarios can be defined on the basis of the presence (with different intensity level) or absence of every aggressive phenomenon.
In particular, the *atmospheric corrosion* can produce a section reduction of the elements of the bridge. In an alternative scenario, corrosion can be avoided thanks to an effective maintenance and the use of adequate corrosion inhibitors.
The intensity of the *fatigue phenomenon* is strictly correlated to the traffic volume present on the bridge. The current traffic is about 32 trains per day, but it could be increased due to specific

needs of the region. Both the current and possible “traffic increased” scenario must be considered in order to have an exhaustive overview of the possible damage of the bridge.
The probability of occurrence of a *seismic event* of a given intensity is extremely connected to the hazard characteristics of the site. Even if the Valencian region has low seismicity, it is important to understand how the eventual presence of seismic damage would interact with other aggressive phenomena.
Consequently, it is possible to hypothesize eight damage scenarios corresponding with all the possible combination of *atmospheric corrosion*, *fatigue phenomenon* and presence or not of a *seismic event* (Table 2).

SCENARIOS	*CORROSION*	*FATIGUE*	*SEISMIC EVENT*
Scenario 1	No corrosion (Constant maintenance)	Current traffic volume	No seismic damage
Scenario 2	No corrosion (Constant maintenance)	Current traffic volume	Seismic event
Scenario 3	No corrosion (Constant maintenance)	Increased traffic volume	No seismic damage
Scenario 4	No corrosion (Constant maintenance)	Increased traffic volume	Seismic event
Scenario 5	Atmospheric corrosion	Current traffic volume	No seismic damage
Scenario 6	Atmospheric corrosion	Current traffic volume	Seismic event
Scenario 7	Atmospheric corrosion	Increased traffic volume	No seismic damage
Scenario 8	Atmospheric corrosion	Increased traffic volume	Seismic event

Table 2: Different damage scenarios.

For every different damage scenario, an analytical failure tree can be achieved in order to understand what types of damage could occur.

To provide an example, Figure 11 show an analytical failure tree for the Scenario 1 regarding the presence of an effective maintenance (to avoid corrosion), current traffic volume and no presence of seismic damage.

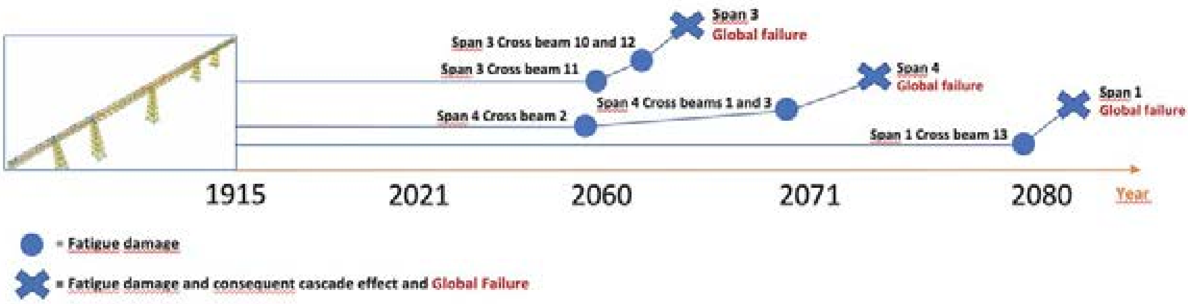


Figure 11: Analytical failure tree for the Scenario 1.

6 CONCLUSION

The work proposes a methodological approach to develop an analytical failure tree of historical steel bridges based on the calibration of a numerical FEM model able to include different damage scenarios in the analysis:

i) The atmospheric corrosion phenomenon is evaluated in accordance with the ISO 9224-2012 assuming a uniform section reduction of the steel elements of the bridge.
ii) The evaluation of the fatigue phenomenon exploits the Miner's Rule and the curve "Stress – Number of cycles" to identify the number of load cycles of a given amplitude that the element can experience before the failure.
iii) The seismic damages can be included with state-of-the-art analysis approaches based on a nonlinear numerical model of the bridge representing its current condition.

Compared with the classical fault tree, the proposed analytical failure tree is achieved by exploiting a FEM model and numerical analysis. On the contrary, the classical fault tree is drawn based on the technician's expertise and by following a heuristic approach. In addition, the proposed failure tree has the advantage to displaying the failure events on a timeline.

The approach is applied to the real case study of a riveted steel bridge located in the Valencian railroad network (Spain) and the example of the analytical failure tree achieved for the scenario involving fatigue damaged related to 32 trains per day, absence of atmospheric corrosion and seismic damage is shown. The analytical failure tree obtained with the proposed approach can be useful for diagnostics, monitoring and forecast intervention by simulating different consequence scenarios.

Future research will perform an exhaustive investigation to achieve the analytical failure trees of several different scenarios in order to have a complete overview of the possible failures and any cascade effects on the investigated bridge.

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