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Consequence-based robustness assessment of a steel truss bridge

Pierluigi Olmati[1], Konstantinos Gkoumas*[1], Francesca Brando[2] and Liling Cao[2]

[1]*Faculty of Engineering, Sapienza University of Rome, Via Eudossiana 18, 00184, Rome, Italy*
[2]*Thornton Tomasetti, 51 Madison Ave., New York, N.Y. 10010-1603, United States*

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Abstract. Aim of this paper is to apply to a steel truss bridge a methodology that takes into account the consequences of extreme loads on structures, focusing on the influence that the loss of primary elements has on the structural load bearing capacity. In this context, the topic of structural robustness, intended as the capacity of a structure to withstand damages without suffering disproportionate response to the triggering causes while maintaining an assigned level of performance, becomes relevant. In the first part of this study, a brief literature review of the topics of structural robustness, collapse resistance and progressive collapse takes place, focusing on steel structures. In the second part, a procedure for the evaluation of the structural response and robustness of skeletal structures under impact loads is presented and tested in simple structures. Following that, an application focuses on a case study bridge, the extensively studied I-35W Minneapolis steel truss bridge. The bridge, which had a structural design particularly sensitive to extreme loads, recently collapsed for a series of other reasons, in part still under investigation. The applied method aims, in addition to the robustness assessment, at increasing the collapse resistance of the structure by testing alternative designs.

Keywords: consequence-based design; structural robustness; collapse resistance; progressive collapse; skeletal structures; steel truss bridge; alternative design

1. Introduction

Structural robustness and collapse resistance are research topics particularly relevant both in the design of new structures, and also for the safety assessment of existing structures. The latter are prone not only to local failure due to accidental or malevolent attacks, but also due to long term material degradation (e.g., corrosion), bad design or construction. Behind this attention, there is the interest from a society that cannot tolerate death and losses as in the past. This is evident after:

- Recent terrorist attacks (a series of terror attacks in America and beyond, the deadliest being the September 11, 2001 attacks in New York at the World Trade Centre);
- Recent bridge collapses due to deterioration, bad design or construction (for example, the De la Concorde overpass in Montreal, 2006 and the I-35 West Bridge in Minneapolis in 2007);
- Recent difficult to foresee multiple hazard events from natural sources (wind, earthquake,

*Corresponding author, Associate Researcher, E-mail: konstantinos.gkoumas@uniroma1.it

flooding, wildfire, etc.) and from human sources (terrorism, fire, etc.) that lead to dramatic consequences, the most significant of which is the 2011 earthquake, off the Pacific coast of Tōhoku, that triggered powerful tsunami waves.

Among all other steel structures, many steel truss bridges in their various forms, very common worldwide, are now aged, not often optimally maintained, and need to be checked equally for safety and for serviceability. In this sense, also the optimal cost effective allocation of resources and the prioritization in the retrofitting phase is a very important issue.

Considering what said above, aim of this paper is to apply to a steel truss bridge, a methodology that, among else, takes into account the consequences of unexpected actions on structures with a special focus on the effect of the loss of primary elements on the structural load bearing capacity (Biondini *et al*. 2008).

Conceptually the paper is organized in this manner: Section 2 provides a brief introduction and a recent literature review on topics related to structural robustness, collapse resistance and progressive collapse, focusing on steel structures. Section 3 introduces the member consequence factor (C_f) and the robustness assessment based on the C_f. Section 4 focuses on applications on simple structures, aiming at testing the method. Section 5 introduces the case-study steel truss bridge and the performed FE (Finite Element) numerical analyses. Finally, Section 6 provides some considerations and indications for future research.

2. Structural robustness, collapse resistance and progressive collapse

Even though a variety of terms have been used in literature, robustness in structural engineering is commonly defined as the “insensitivity of a structure to initial damage” and collapse resistance as the “insensitivity of a structure to abnormal events” (Starossek and Haberland 2010).

Similarly, in ASCE 7-05 (2005), progressive collapse is defined as the spread of an initial local failure from element to element, eventually resulting in collapse of an entire structure or a disproportionately large part of it. Starossek and Haberland (2010) focus on the differences of progressive and disproportionate collapse, concluding that the terms of disproportionate collapse and progressive collapse are often used interchangeably because disproportionate collapse often occurs in a progressive manner and progressive collapse can be disproportionate.

From a historical perspective, progressive collapse came up first as a structural engineering concern just after the collapse of the Ronan Point Tower, a residential apartment building in Canning Town, London, UK, in May 1968, two months following initial occupancy of the building. Ronan Point was a 22-story building, with precast concrete panel bearing wall construction. An explosion of natural gas from the kitchen of a flat on the 18th floor failed an exterior bearing wall panel, which led to loss of support of floors above and subsequent collapse of floors below due to impact of debris (Ellingwood 2002).

Concerning the above mentioned topics, there has been a lot of research in the recent years. Starossek and Haberland (2010), provide a terminology. A review of international research on structural robustness and disproportionate collapse is provided in Arup (2011). Regarding the quantification of robustness related issues, Canisius *et al*. (2007) provide an overview of methods. Starossek (2009) covers issues related to progressive collapse. Bontempi *et al*. (2007), Arangio *et al*. (2011) and Sgambi *et al*. (2012) provide a dependability framework, adapted from the electronic engineering field, where dependability attributes are either related to structural safety or

Table 1 Abnormal events that could threaten a structure (from Starossek and Haberland 2012)

Faults			Errors
External		Intrinsic	
Man-made (accidental or intentional)	Environmental (natural)		
Impact (car, train, ship, aircraft and missile) Explosion (gas, explosives) Fire Excessive loading (live load)	Earthquake Extreme wind Heavy snowfall (excessive roof loads) Scour Impact (avalanche, landslide, rock fall, floating debris) Volcano eruption	Lack of strength Cracks Deterioration	Design errors Construction errors Usage errors Lack of maintenance

serviceability. Focusing on structural safety, the attributes of structural integrity, collapse resistance, damage tolerance and structural robustness are investigated. Strategies and methods for the robustness achievement are discussed in Bontempi and Giuliani (2008), together with the robustness assessment of a very long span suspension bridge.

That said, and even though many robustness research topics focus on explosions and terrorist attacks, as Table 1 suggests, there is a variety of reasons or events that could endanger a structure, eventually leading to a progressive collapse (Starossek and Haberland 2012). Potential failure scenarios specific for bridges are also provided in FHWA (2011), within a framework aiming at the resilience improvement.

The collapse likelihood of a structure is typically characterized in probabilistic terms. When an unexpected or critical event occurs, Ellingwood and Dusenberry (2005) describe, in probabilistic terms, the probability of a collapse in a structure as the product of the probabilities of three sub events:

1. The extreme action associated with the event hits the structure;
2. The structure is damaged in the area directly affected by the action;
3. The local damage causes failures of other structural elements and leads to the collapse of a significant part of the structure.

The assessment of the risk associated with the event (commonly defined as the product of a probability of occurrence and of the corresponding consequence) can be performed using standard risk techniques. Several authors have focused on aspects of risk analysis and assessment in the civil engineering field - see for example Faber and Stewart (2003), and, more recently, Gkoumas (2008). Risk related special issues include the risk aversion for low-probability, high-consequence events (Cha and Ellingwood 2012) and the risk consistency in multihazard design for frame structures (Crosti *et al*. 2011).

Focusing on disproportionate collapse in probabilistic terms, Ellingwood *et al*. (2007) decompose the probability of disproportionate collapse $P[C]$ as a result of an abnormal event, into three constituents: abnormal event, initial damage, disproportionate failure spreading. This is represented as the product of partial probabilities

$$P[C] = P\left[C|D\right] P\left[D|E\right] P\left[E\right] \quad (1)$$

where, $P[E]$ is the probability of occurrence of the abnormal event E that affects the structure; $P[D|E]$ is the conditional probability of the initial damage D, as a consequence of the abnormal event, and $P[C|D]$ is the conditional probability of the disproportionate spreading of structural failure, C, due to the initial damage D. The safety of structures with regards to the single elements contained in the equation, each characterizing the single sub-event mentioned above, is pursued in modern structural codes by the introduction of partial safety factors.

According to this approach, Giuliani (2012) identifies these three design strategies for obtaining robustness:

1. Prevention or mitigation of the effects of the event (increase collapse safety);
2. Prevention or mitigation of the effects of the action (increase structural integrity);
3. Prevention or mitigation of the effects of the damage (increase structural robustness).

These strategies are schematically depicted in Fig. 1.

The assessment of structural robustness is also strongly related to the degradation state of the structures, caused by environmental agents: concrete carbonation, steel reinforcement corrosion, alkali aggregate reaction, freeze-thaw cycles can lead, over time, to an assessment of structural strength that is very different from that provided in the design phase (Biondini and Frangopol 2009). The effect of the above factors could compromise the structural response under a localized event.

Furthermore, different structural systems exhibit different degrees of robustness (Wolff and Starossek 2010), something neglected even in modern design procedures that use partial safety factors. Another issue very important in determining structural robustness for bridges is redundancy. Bridge redundancy, is defined in the Ghosn and Moses (1998) as the capability of a bridge to continue to carry loads after incurring damage or the failure of one or more of its members. This capability is due to redistribution of the applied loads in transverse and/or longitudinal directions.

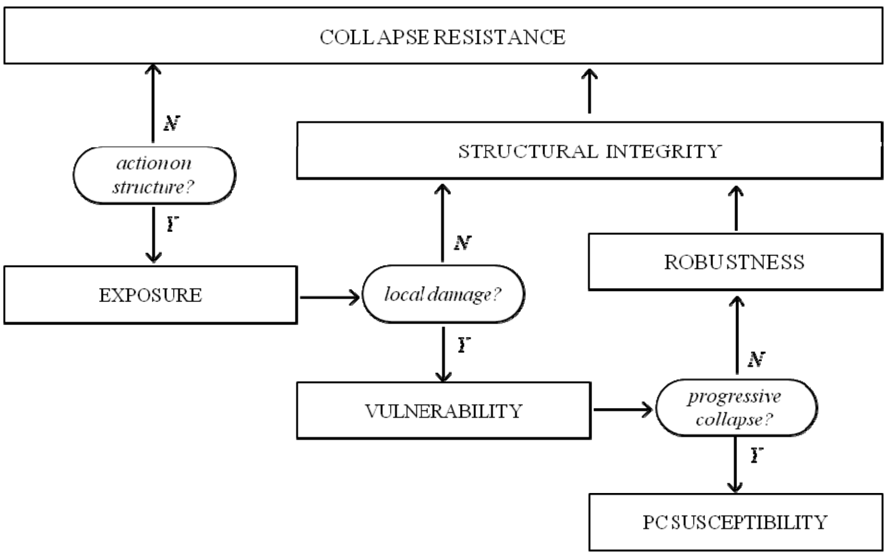

Fig. 1 Strategies for safety against extreme events and corresponding requirements (Giuliani 2012)

Moreover, the inherent uncertainty associated with actions and mechanical, geometric and environmental parameters cannot be ignored since they affect the structural response (Biondini *et al*. 2004, Ciampoli *et al*. 2011, Garavaglia *et al*. 2012, Petrini and Ciampoli 2012).

Steel truss structures and bridges have been the subject of recent research on what concerns their ultimate strength and progressive collapse susceptibility. Choi and Chang (2009), focus on the vertical load bearing capacity of truss structures, using a sensitivity index that accounts for the influence of a lost element to the load bearing capacity. Miyachi *et al*. (2012) focus on how the live load intensity and distribution affect the ultimate strength and ductility of different steel truss bridges, similar to the one considered in this study. Malla *et al*. (2011) conduct nonlinear dynamic analysis for the progressive failure assessment of bridge truss members, considering their inelastic post-buckling cyclic behavior. Saydam and Frangopol (2011) use FE skills to investigate the vulnerability, redundancy and robustness of truss bridges, taking into account the stochastic time-dependent deterioration of the structure.

Progressive collapse literature indicates that the extensive research has been performed in the last few years on steel moment frames possibly owed to the fact that different design guidelines are issued in the US by the General Service Administration (GSA 2003) and the Department of Defense (DoD 2009). Kim and Kim (2009) conduct nonlinear dynamic analysis on benchmark buildings (3, 6 and 15-story) and compare the results with more straightforward linear static step-by-step analysis. Using nonlinear dynamic finite element simulations, Kwasniewski (2010) investigates the collapse resistance of an 8-story steel framed structure, and inquire on the uncertainties affecting the problem. Izzuddin *et al*. (2008a), provide a framework for progressive collapse assessment of multi-story buildings, considering as a design scenario the sudden loss of a column. Using this framework, the same authors (Izzuddin *et al*. 2008b) investigate possible scenarios, in the form of the removal of either a peripheral or a corner column, in a typical steel-framed composite building. Yuan and Tan (2011) investigate the progressive collapse of a 9-story building, at a global level, using a numerical spring-mass-damper model. Hoffman and Fahnestock (2011) investigate different column loss scenarios on 3 and 4-story steel buildings, focusing on different aspects of the problem, among else, the load redistribution and the column lost location. Galal and El-Sawy (2010) compare retrofitting strategies for 18-story buildings with different spans using 3D nonlinear dynamic analyses.

An important issue is the model complexity in the progressive collapse assessment. Alashker *et al*. (2011) deal with approximations in the numerical modeling, using a 10-story steel building as a case study, and compares four models of different levels of complexities (planar and 3D). Their conclusion is that, under restricted conditions, planar models can lead to reasonable results regarding the progressive collapse characterization, however, a full 3D analysis, in spite of its computational cost, may be the only sure way to rigorously investigate this aspect. Rezvani and Asgarian (2012), conduct different non-linear static and dynamic analyses, among else, on an 8-story building, aiming at the progressive collapse assessment, and compare the results from the different analysis methods.

A relevant issue related to the structural robustness evaluation, is the choice of proper synthetic parameters describing the sensitivity of a damaged structure in suffering a disproportionate collapse. Recently Nafday (2011) discusses the usefulness of consequence event design (as opposed to using a probabilistic approach), for extremely rare, unforeseen, and difficult to characterize statistically events (black swans). In this view, the author, with reference to truss structures, proposes an additional design phase that focuses on the robustness, the damage tolerance and the redundancy of the structure. This proposed metric is based on the evaluation of the determinants of the normalized stiffness matrixes for the undamaged and damaged structure.

Concerning extreme loads on structures, a scientific debate takes place nowadays on the appropriate design methodology to adopt (see for example COST 2011). To this point, the member-based design is not efficient for contrasting extreme loads on structures that in general are unpredictable and not probabilistically characterized (Nafday 2011). Following the approach of HSE (2001) in the case of high uncertainties regarding the extreme loading likelihood, it is necessary to put emphasis on the consequences of the event.

Considering the above, the method applied in this study aims at increasing the collapse resistance of a structure, by focusing on the resistance of the single structural members, and by accounting for their importance to the global structural behavior consequently to a generic extreme event that can cause a local damage. Moreover, the method is particularly helpful for unpredictable events that by definition are not easy to take into account in the design phase. This does not mean that the collapse resistance (Starossek 2009) is accounted only for the single member resistance, because the authors intend, as a design philosophy, to increase the resistance of the single members in addition to the structural stability analysis that provide the assessment of the global structural behavior. In other words, if the collapse resistance of a structure is identified by: the "load characterization", the "local resistance", and the "insensitivity to a local damage" (Starossek 2009), this method focuses on the issue of "local resistance". Thus, it neglects the "load characterization" of the extreme load since it is considered unpredictable, and it is complementary to the so-called threat independent stability analyses (DoD 2009, GSA 2003).

3. Member consequence factor and robustness assessment

Focusing on skeletal structures (e.g., trusses), current member-based design in structural codes does not explicitly consider system safety performance during the structural design, while the level of safety in new designs is usually provided on the basis of intuition and past experience (Nafday 2008). On the other hand, the Ultimate Limit State (ULS) of the Performance-Based Design (PBD) requires (see for example EN 1990 2002) that individual structural members are designed to have a resistance (R) greater than the load action (E), where both R and E are probabilistically characterized (Stewart and Melchers 1997).

The member-based design is summarized in the following design expression, valid for a single structural member:

$$R_d^{\text{undamaged}} - E_d^{\text{undamaged}} \geq 0 \tag{2}$$

where $R_d^{\text{undamaged}}$ and $E_d^{\text{undamaged}}$ are the design values respectively of the resistance and of the solicitation (EN 1990 2002) in the undamaged configuration of the structure. Concerning the commonly implemented standards this equation is not respected with a probability of $10^{-(6\div7)}$. The method applied here aims to introduce an additional multiplicative coefficient in the first term of Eq. (2): this is identified as the member consequence factor (C_f), takes values within a range from 0 to 1, and quantifies the influence that a loss of a structural element has on the load carrying capacity.

Essentially, if C_f tends to 1, the member is more likely to be important to the structural system; instead if C_f tends to 0, the member is more likely to be unimportant to the structural system. C_f provides to the single structural member an additional load carrying capacity, in function of the nominal design (not extreme) loads. This additional capacity can be used for contrasting unexpected and extreme loads.

$$(1-C_f^{\text{scenario}})*R_d^{\text{undamaged}}-E_d^{\text{undamaged}}\geq 0 \tag{3}$$

Nafday (2011) provides Eq. (3) in a similar manner, with the only difference being C_f that is the complementary to the proposed one, so the first term of Eq. (3) is multiplied directly by C_f. Thus, in this study the equation proposed by Nafday (2011) has been slightly revised in order to fit with the here proposed expression of the C_f - see both Eq. (3) and Eq. (4).

The structure is subjected to a set of damage scenarios and the consequence of the damages is evaluated by the consequence factor (C_f^{scenario}) that for convenience can be easily expressed in percentage. For damage scenario is intended the failure of one or more structural elements. Moreover, the robustness can be expressed as the complement to 100 of C_f^{scenario}, intended as the effective coefficient that affects directly the resistance - see Eq. (3).

C_f^{scenario} is evaluated by the maximum percentage difference of the structural stiffness matrix eigenvalues of the damaged and undamaged configurations of the structure.

$$C_f^{\text{scenario}}=\max\left(\frac{(\lambda_i^{un}-\lambda_i^{dam})}{\lambda_i^{un}}100\right)_{i=1-N} \tag{4}$$

where, λ_i^{un} and λ_i^{dam} are respectively the *i*-th eigenvalue of the structural stiffness matrix in the undamaged and damaged configuration, and N is the total number of the eigenvalues.

The corresponding robustness index (R^{scenario}) related to the damage scenario is therefore defined as:

$$R^{\text{scenario}}=100-C_f^{\text{scenario}} \tag{5}$$

Values of C_f close to 100% mean that the failure of the structural member most likely causes a global structural collapse. Low values of C_f do not necessarily mean that the structure survives after the failure of the structural member: this is something that must be established by a non-linear dynamic analysis that considers the loss of the specific structural member. A value of C_f close to 0% means that the structure has a good structural robustness.

Some further considerations are necessary. The proposed method for computing the consequence factors, for different reasons, should not be used 1) for structures that have high concentrated masses (especially non-structural masses) in a particular zone; and 2) for structures that have cable structural system (e.g., tensile structures, suspension bridges).

For the first kind of structures, the reason is due to the dynamic nature of a structural collapse, since Eq. (4) does not take into account the mass matrix of the system that is directly related to the inertial forces. It is possible to accept this limitation only if the masses are those of the structural members, thus distributed uniformly. Moreover it is impossible to account for any dynamic magnification phenomena with Eq. (4).

For the second kind of structures, the reason is related to the geometrical non-linearity of cable structures. For such structures the stiffness matrix is a function of the loads, something not accounted for in the elastic stiffness matrix. Moreover for the nature of the elastic stiffness matrix, eventual structural dissipative behaviors and non-linear resistive mechanisms (e.g., catenary action) are not taken into account.

In the authors' opinion the above limitations can be accepted if the desired outcome is a non-computational expensive method, since the C_f value provides an indication of the structural robustness in a quick and smart manner. Additional research could focus on the development of criteria that a Robustness index should have to take into account the previous issues that Eq. (4) does not account for.

With this in mind the C_f as expressed in Eq. (4) can be used primarily as an index to establish the critical structural members for the global structural stability, or to compare different structural design solutions from a robustness point of view. The latter implementation of C_f can be very helpful for the robustness assessment of complex structures, for example wind turbine jacket support structures (Petrini *et al*. 2011), since it provides an indication on the key structural elements that in a complex structure are of difficult evaluation.

In this study, the member consequence factor is computed for the structural elements of a steel truss bridge. Before that, the method is applied to simple structural systems.

4. Tests on simple structures

In this section, first, a simple example is shown, in order to provide insight on the method proposed for computing the consequence factor and the structural robustness index. Fig. 2 shows a linear spring system.

In a two-dimensional space there are two single degree of freedom translational springs. Spring "*a*" has stiffness k_a and spring "*b*" has a stiffness k_b. The stiffness matrix of the system is given by Eq. (6).

$$\underline{K} = \begin{bmatrix} k_a & 0 \\ 0 & k_b \end{bmatrix} \tag{6}$$

To make a numerical example, assuming $k_a = k_b = 10$ kN/m, the obtained undamaged stiffness matrix is

$$\underline{K}^{\text{undamaged}} = \begin{bmatrix} 10 & 0 \\ 0 & 10 \end{bmatrix} \tag{7}$$

A hypothesis is made that a damage scenario (called scenario 1) reduces the stiffness of the spring "b": $k_b^{\text{damaged}} = 7$ kN/m < $k_b^{\text{undamaged}} = 10$ kN/m. Consequently, the damaged stiffness matrix takes the form of

$$\underline{K}^{\text{damaged}} = \begin{bmatrix} 10 & 0 \\ 0 & 7 \end{bmatrix} \tag{8}$$

At this point, applying Eq. (4), the following values for the consequence factors are obtained

$$C_{f1}^{1} = 0\% \qquad C_{f2}^{1} = 30\% \tag{9}$$

The maximum consequence factor of the two, for the scenario 1, is C_{f2}. Consequently for this scenario the consequence factor is the C_{f2} equal to 0.3. Finally applying Eq. (5) the robustness index obtained is 70%.

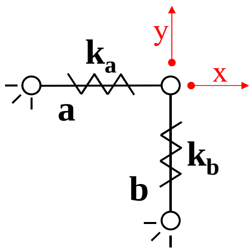

Fig. 2 Example spring structure

This method, previously applied analytically, is now applied numerically to two additional examples (two simple structures). First, a single bay frame structure with a diagonal beam brace, composed in total of 5 members, is considered (Fig. 3, left). All of the cross sections of the structural members are the European IPE 300 (similar to a W 12 × 30) in European S235 steel (comparable to the ASTM A36), while both the frame span and the frame height are one meter. The structure is plane and the boundary conditions are of the pinned type. The evaluated scenarios consist in the removal of elements 1, 2 and 3 sequentially, so the damage is cumulative: this means that the each scenario includes the damage from the previous one. C_f is computed by Eq. (3) and the results in terms of C_f and robustness are indicated on the right side of Fig. 3. After the failure of members 1 and 2 the structure is still redundant so the R^{scenario} has a non-zero value; instead after the failure of members 1, 2, and 3 the structure is a mechanism and consequently the R^{scenario} is zero (C_f is equal to 100%).

The second structure considered is a star-shaped structure (Fig. 4, left). In totally there are 8 members with a pipe cross section: the outside diameter is of 20 centimeters, and the thickness is of 20 millimeters. The steel is the European S235 one. With respect to the left side of Fig. 4, members 1, 3, 5, and 7 are 0.5 meters long and members 2, 4, 6, and 8 are 0.7 meters long. All the members are connected to each other by a fixed type connection. Also the boundary conditions are of the fixed type and the structure is plane.

The evaluation consists in removing elements 1 through 8, and the damage is intended as cumulative, just like in the previous example. The results in terms of C_f and robustness are indicated on the right side of Fig. 4. Until reaching damage scenario 6 the $\mathrm{R}^{\text{scenario}}$ has a non-zero value. After that for damage scenario 7 the structure is reduced to a cantilever and the R^{scenario} is 0.4%. Finally, R^{scenario} is equal to zero when the final structural member is eliminated (C_f in this case is equal to 100%).

It is possible to observe from Figs. 3 and 4 that the proposed method captures the structural robustness reduction with the increase of the damage level. On the other hand, C_f increases with the damage level.

5. Application on a steel truss bridge

This section focuses on the robustness assessment of a steel truss bridge using the member consequence factor method.

5.1 Description and issues of the case-study bridge

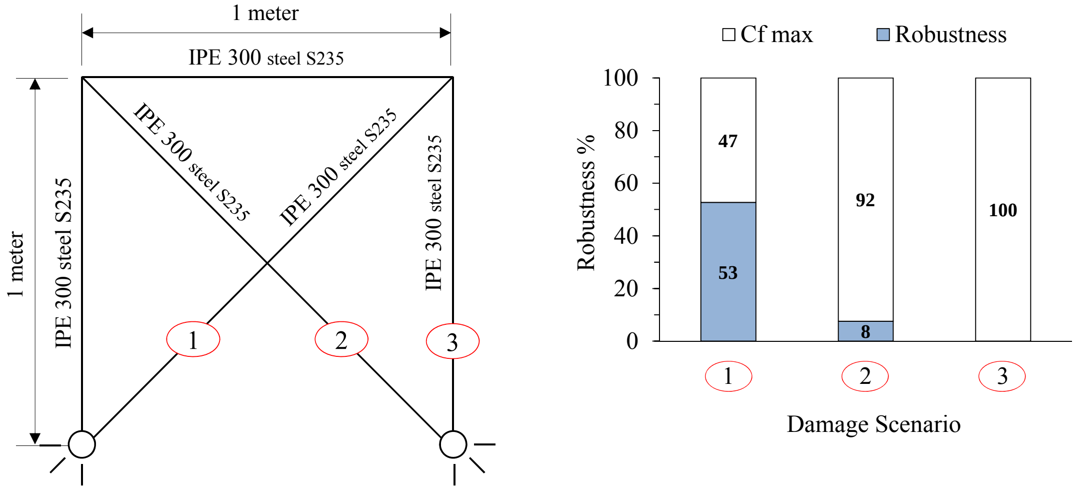

Fig. 3 Example truss structure (left) and damage scenario evaluation (right)

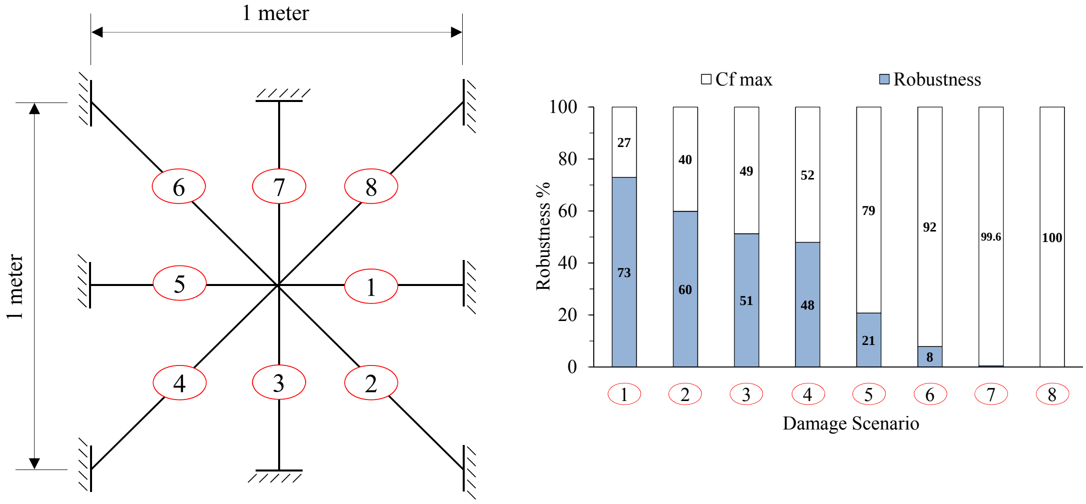

Fig. 4 Example star structure (left) and damage scenario evaluation (right)

The bridge used as a case study is the I-35 West Bridge in Minneapolis. The I-35 West Bridge was built in the early 1960s and opened to traffic in 1967. The bridge spanned across the Mississippi River, Minneapolis. The bridge was supported on thirteen reinforced concrete piers and consisted of fourteen spans. Eleven of the fourteen spans were approach spans to the main deck truss portion. The total length of the bridge including the approach and deck truss was approximately 580 meter (1,907 feet). The length of the continuous deck truss portion which spanned over four piers was approximately 324 meter (1,064 feet). The elevation of the entire bridge is shown in Fig. 5 (data and figure from NTSB 2007).

The deck truss portion of the bridge was supported on a pinned bearing at Pier 7 and roller bearings at the other three supports. The main bridge trusses were comprised of built-up welded

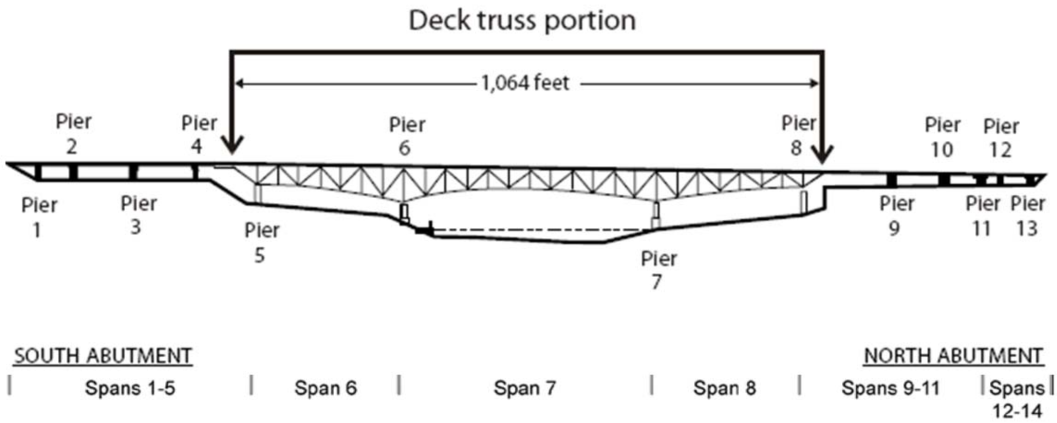

Fig. 5 East elevation of the I-35W Bridge (source: NTSB 2007)

box and I-sections sandwiched by gusset plates at each panel point.

Steel truss bridges, like the I-35 West Bridge, had longer and lighter spans than their contemporaries. The innovations, which facilitated the reduction in weight, include the efficiencies inherent in statically determinant trusses, new stronger steels, thin gusset plate connections, and welded box sections.

The catastrophic collapse which occurred on August 1st 2007 was probably due to a combination of the temperature effect, roller bearings condition, and increased gravity loads on the bridge prior to collapse. For this functionally non-redundant bridge the initial buckle at the lower chord member close to the pier and local plastic hinges in the member resulted in global instability and collapse (Malsch *et al*. 2011).

The bridge has been thoroughly studied by Brando *et al*. (2010) focusing on the issues of redundancy, progressive collapse and robustness, studies have been conducted in order to assess the effect of the collapse of specific structural components (Crosti and Duthinh 2012), while Crosti *et al*. (2012) performed non-linear dynamic analysis on specific damage scenarios.

5.2 FE model and numerical analyses

For computing the consequence factors and the robustness index of the structure for the selected damage scenarios a FE model of the structure is necessary. Fig. 6 shows the three-dimensional FE model of the I-35 West Bridge built using the commercial FE solver Sap2000® (Brando *et al*. 2010).

Both shell and beam finite elements are used in the FE model. The bridge superstructure and both the deck girders and beams are built using beam elements, while, the concrete deck is modeled using shell elements. Moreover, contact links connect the deck with both the deck girders and beams. In accordance to the original blueprints of the I-35 West Bridge (MDT 2012), standard and non-conventional beam cross sections are implemented in the model.

From this model a simplified (plane) FE model is extracted and is adopted for computing the structural stiffness matrix in both the damaged and undamaged configurations. This choice has mostly to do with computational challenges in computing the stiffness matrix for the full model. Regarding the structural decomposition of complex structures it is possible to refer to Bontempi *et*

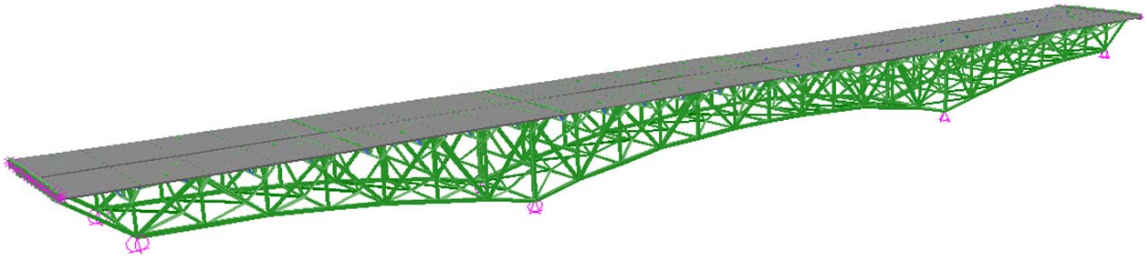

Fig. 6 3D FE model of the I-35 West Bridge

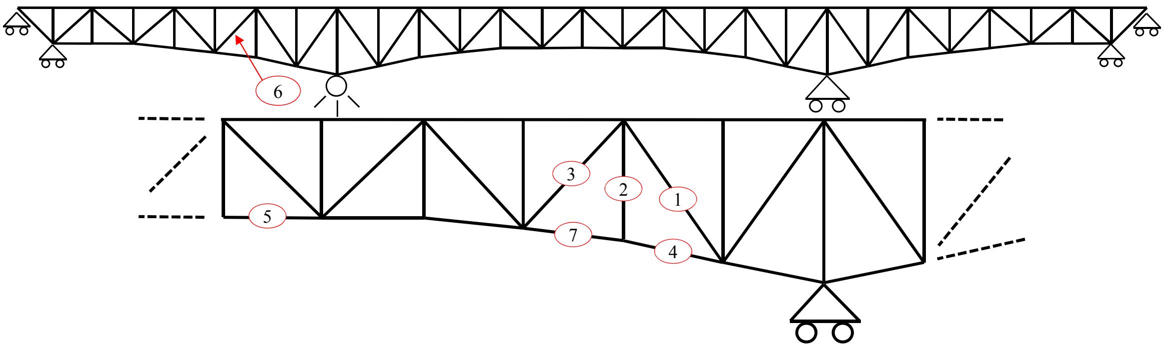

Fig. 7 Lateral truss of the bridge and selection of damage scenarios

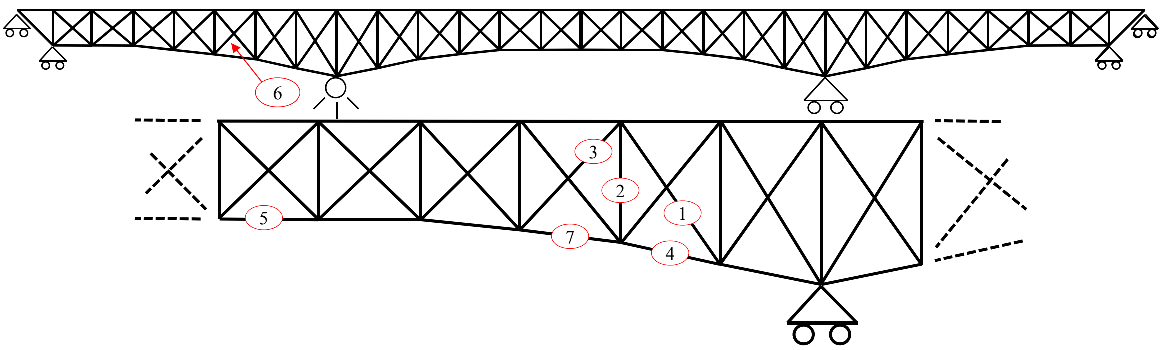

Fig. 8 Updated lateral truss of the bridge and selection of damage scenarios

al. (2008) and Petrini and Bontempi (2011).

The expression of the consequence factor provided by Eq. (4) refers to the eigenvalues of the elastic stiffness matrix. The choice to use a simplified model is also justified and feasible since Eq. (4) is independent from the mass of the structure. Eq. (4) is also independent from the loads, so the loads in the FE model are not considered. The concrete deck is only simply-supported by the bridge superstructure, so the concrete deck is not considered in the analyses and it is omitted in the model, consequently, the contact links are deleted as well. The deck girders and beams are also omitted since they do not have a strong influence to the load bearing capacity of the bridge.

The two trusses of the bridge superstructure are similar and connected by a transverse truss structure, so the analyses focus on a single truss; at this point one plane truss is obtained from the three-dimensional model, in order to have a two-dimensional FE model, implemented for computing the stiffness matrix in both the damaged and undamaged configurations.

Concluding, only a single lateral truss of the bridge is considered, and a set of damage scenario is selected (Fig. 7). The damage scenarios for this application are not cumulative, so only a single member is removed from the model for each damage scenario (Brando *et al*. 2012). In this application the scenarios chosen focus on the area recognized as initiating the collapse according to forensic investigations in different researches (NTSB 2007, Malsch *et al*. 2012, Brando *et al*. 2013).

With the aim of increasing the structural robustness of the bridge, and in order to test the sensitivity of the method proposed, an improved variation of the structural system is considered. In this case (Fig. 8) the updated bridge truss is a hyper-static steel truss structure.

The results of both the original and the enhanced structural schemes, under the same damage scenarios, are shown in Figs. 9 and 10.

The proposed robustness index (based on the member consequence factor C_f) captures both the lack of robustness of the I-35 W Bridge, and its robustness enhancement as a consequence of increasing the redundancy of the structure.

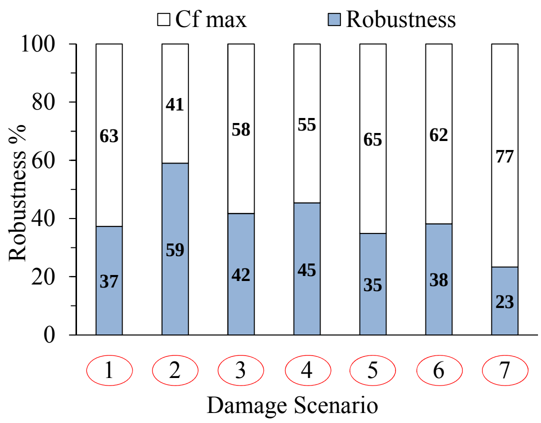

Fig. 9 Damage scenario evaluation in terms of C_f for the original configuration of the bridge

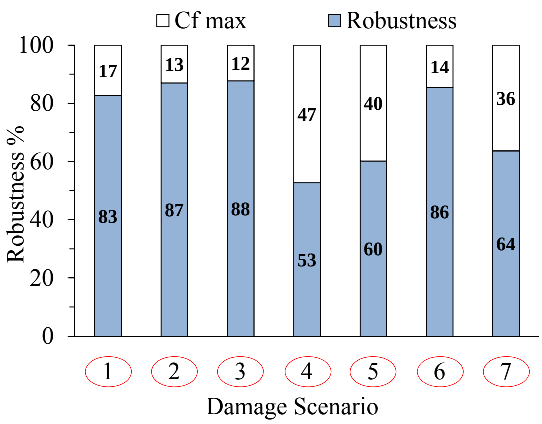

Fig. 10 Damage scenario evaluation in terms of C_f for the improved configuration of the bridge

Generally speaking, it can be observed that the case-study bridge shows a low robustness index. This is due to the fact that it is (internally) statically determined. In order to better understand the use of the proposed consequence factor, it is useful to focus the attention on the Damage Scenario number 7 (DS7), since it is particularly critical for the robustness requirement of this structure. It has to be noted moreover that the proposed method highlights the sensitivity of the bottom chord member, which was pinpointed from the investigation on the causes of the collapse performed by Malsch *et al*. (2012). From the analysis of the bridge in its original configuration and for the chosen damages configurations, a consequence factor of 0.77 has been computed for the DS7 and, consequently, a robustness index of 0.23 is obtained. This result can be used to design or improve the bridge structure by means of different strategies:

- The consequence factors obtained by the analysis of the various damage scenarios can be used, as shown in Eq. (3), for the re-sizing of the structural elements (each element is linked to the specific C_f obtained from the analysis considering its failure). In this case the structural scheme of the bridge does not change with respect to the original one. This option can be considered as a local (element-based) improvement of the structural system.
- The consequence factor can be used only as a robustness performance index, without making use of Eq. (3). More than one structural configuration can be examined in order to assess which is the best solution in terms of C_f. An example of this strategy is given in the previous application of Fig. 8. In this case the scheme of the bridge has been modified by inserting additional structural elements in order to obtain a redundant truss bridge. In the examined case the consequence factor obtained by the DS7 decreases from 0.77 to 0.36; this appreciable result is probably due to the position of the failed element in the DS7, which, being a lower element of the truss, plays an important role in the load carrying capacity of the original system. Generally speaking, the redundant bridge configuration (Fig. 8) shows certain insensitivity to the internal damage scenarios (number 1, 2 and 3). This option can be considered as a global improvement of the structural system.
- The previous strategies can be adopted simultaneously: i) the designer-sizing of the elements can be affected by the robustness index by using Eq. 3; and ii) the structural scheme can be changed (also on the basis of the C_f values) in order to increase the robustness. In this case, both local and global solutions provide improvements to the system.

6. Conclusions

In this study, the robustness of structures is inquired using a metric based on the member consequence factor. The application of this metric seems to be promising for the robustness assessment of a complex structural system, such as the I-35 Bridge used as a case study, by identifying critical damage scenarios (scenarios involving the loss of elements) associated with low values of this metric. This method could be used as tool in the design, analysis and investigation processes, for localizing critical areas. Furthermore, comprehensive assessments that consider a larger set of damage scenarios can be performed by implementing this method using appropriate search heuristics.

Limitations of the implemented method arise from the fact that in the analyses a reduced structural system is used. In this sense, findings can be considered preliminary, and have to be verified using complete models and advanced numerical analyses.

Some indications for further research can be identified. A better expression for the C_f could be obtained by considering both the stiffness and mass matrix of the structure. Moreover, the plastic resources of the structure could be take account in the C_f expression.

Future studies could also focus on the establishment of a threshold value for the member consequence factor that does not lead to a structural collapse and nonlinear dynamic analyses can be performed on the complete 3D model for the critical cases identified with this method.

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