Available online at www.sciencedirect.com

ScienceDirect

Procedia Structural Integrity 25 (2020) 305–315

Structural Integrity Procedia

www.elsevier.com/locate/procedia

1st Virtual Conference on Structural Integrity – VCSI1

An investigation on the structural integrity of network arch bridges subjected to cable loss under the action of moving loads

Domenico Ammendolea[a], Domenico Bruno*[a], Fabrizio Greco[a], Paolo Lonetti[a], Arturo Pascuzzo[a]

[a]*Department of Civil Engineering, University of Calabria, Via P. Bucci, Cubo39B, 87030, Rende, Cosenza, Italy*

Abstract

A numerical study is developed to investigate the structural behavior of network arch bridges subjected to the cable loss of hanger elements of the cable system under the action of moving loads. The main aim of the paper is to analyze the effects produced by potential cable loss scenarios on the main kinematic design variables of the structure. The structural behavior is investigated by means of a refined FE nonlinear geometric formulation, in which the influence of large displacements and local vibrations of cable elements are taken into account. The loss of a cable is properly reproduced by means of a damage law consistently with Continuum Damage Mechanics theory. Moreover, a refined formulation is implemented with the aim to reproduce the inertial characteristics of the moving loads, by accounting for the coupling effects arising from the interaction between bridge deformations and moving system parameters. Numerical analyses are performed by means of both nonlinear dynamic analyses and simplified methodologies proposed by existing codes on cable supported bridges. In this framework, the applicability of such simplified methodologies in the case of the network arch bridges is discussed.

Keywords: Network arch bridges; Loss of a cable; Nonlinear analysis; Finite elements; Moving Loads; Dynamic analysis;

* Corresponding author. Tel.: +39 0984 496914
E-mail address: d.bruno@unical.it

10.1016/j.prostr.2020.04.035

Nomenclature

L	Bridge Span Length
B	Bridge width
f	Arch rise
α	Hanger slope
DL	Dead Load
LL	Live Load
H^R	Height of the arch rib cross-section
B^R	Width of the arch rib cross-section
t_w^R	Web thickness of the arch rib cross-section
t_f^R	Flange thickness of the arch rib cross-section
H^T	Height of the tie girder cross-section
B^T	Width of the tie girder cross-section
t_w^T	Web thickness of the tie girder cross-section
t_f^T	Flange thickness of the tie girder cross-section
m	Number of hangers
p	Hanger step
A^C	Hanger cross-section
S^C	Hanger initial stress
c	Moving load speed
DAF	Dynamic amplification factor
SA	Standard Analysis
NSA	Nonstandard Analysis
D	Damage structure
UD	Un-damage structure

1. Introduction

Network arch bridges have been extensively used for overcoming short, medium, and long spans because they efficiently combine aesthetic, structural, and economic benefits (Lonetti et al. (2019); Greco et al. (2019); Pellegrino et al. (2010)). Typically, network arch bridges consist of the following components: two vertical arch ribs, a plane deck, and a cable system. The arch ribs, which are commonly parallel and circular shaped, sustain the deck by means of the cable system; the deck is composed of two longitudinal tied beams and several transversal beams, which support a concrete slab. In particular, the tie beams are mutually connected to the arch ribs extremities thus eliminating the arch horizontal thrust. This ensures the structure to be simply supported globally. The cable system is formed by two specular planes, each made of parallel and equally spaced hangers inclined at a constant angle with respect to the horizontal (Lonetti and Pascuzzo (2019)).

Network arch bridges are frequently used in the context of railway bridge structures, which are normally crossed by heavy and fast-moving trains (Greco et al. (2018)). Such dynamic loads induce relevant stress and deformation fields in the bridge structure, which becomes potentially exposed to damage mechanisms. In particular, the hangers of the cable system are subjected to relevant tractions and vibrations, which may cause cable loss events due to yield or fatigue failure mechanisms. It is worth noting that, the sudden loss of a single hanger may potentially generate the failure of other structural elements, thereby triggering the collapse of the entire structure. Consequently, the analysis of the structural behavior of network arch bridges subjected to the sudden loss of hangers represents a fundamental issue to be addressed in order to design more robust and safety structures.

Current codes on cable-supported bridges do not provide exhaustive design guidelines to address the accidental situation generated by the sudden loss of hangers. In particular, network arch bridges are not treated explicitly. Eurocode 3 (European Committee for Standardization (2006)) provides a simplified method to analyze the sudden loss of cable elements that can be applied to any kind of cable structure. However, the applicability of the cable loss accidental design situation is postponed to the National Annex and no further prescriptions are provided. The PTI recommendations (Post-Tensioning Institute (2007)) of the Post-Tensioning Institute provide prescriptive guidance on the cable loss extreme event, but exclusively with reference to cable-stayed bridges.

A considerable amount of literature has been published on the effect produced by damage mechanisms on cable elements by means of experimental and numerical studies (Mahmoud (2007); Lonetti (2010); Materazzi and Ubertini (2011); Sun et al. (2018); Funari et al. (2018, 2019)). The results have been employed to develop a number of damage models, which were used to analyze numerically the structural behavior of cable-supported bridges in the presence of sudden loss of cable elements. The majority of these studies have focused on cable-stayed bridges considering several structural configurations, load cases, and modeling approaches. Wolff and Starossek (Wolff and Starossek (2009)) examined cable-stayed bridges subjected to the loss of multiple cables by using nonlinear dynamic analyses and concluded that the loss of more than two cables may lead to the overall collapse of the structures. Zhou and Chen (Zhou and Chen (2015)) evaluated the combined effects produced by the cable loss, the service traffic, and wind loads illustrating how traffic-wind loads coupling effects are important to the bridge response following cable-loss events. Jani and Amin (Harshil and Jignesh (2017)) analyzed the effects of a sudden cable breakage due to increasing corrosion on fan and semi-fan type cable-stayed bridges. Greco et al. (Greco et al. (2013)) evaluated the dynamic amplification effects on cable-stayed bridges produced by the cable loss and the moving load action, which was simulated taking into account for nonstandard inertial forces arising from Coriolis acceleration and centripetal acceleration.

Despite the relevance of the topic, few studies have investigated different types of cable-supported bridges (Wu et al. (2019); Wickramasinghe et al. (2020)), and to the Author's knowledge, investigations on arch bridges are still limited. Recently, Sophianopoulos et al. (Sophianopoulos et al. (2019)) have developed a mathematical model to investigate the behavior of tied arch bridges subjected to the sudden loss of cables with the aim to analyze the effects produced by potential cables failure scenarios on the deformations and stresses of the bridge. An investigation on network arch bridges affected by cable loss has been proposed in (Bruno et al. (2018)), in which the most dangerous situation produces by cable loss has been identified and the distribution of stresses in undamaged elements has been explored. However, the previous study considered the structure subjected to dead load only; much uncertainty still remains about the behavior of the structure affected, at the same time, by cable loss hazard and other dynamic actions, such as moving load, wind, earthquake, and seaquake (Lonetti and Maletta (2018)).

This paper examines the structural behavior of network arch bridges subjected to the sudden loss of one hanger of the cable system under the action of moving loads. The study is developed in a dynamic context, employing an efficient time-dependent damage model to properly reproduce the sudden loss of cable elements. Furthermore, a refined formulation to accounting for the inertia effects induced by moving loads is employed with the aim to accurately quantify amplification effects of the main kinematic and stress design variables. Comparisons between numerical analyses and simplified methodologies prescribed by codes are developed in order to assess their accuracy.

The paper is organized as follows: the first section provides a brief overview of prescriptions reported in the main codes on cables supported bridges for the structural analysis in the presence of cable loss; the second section reports a description of the numerical model used for this study; finally, in the last section, numerical results are presented and discussed.

2. A review of existing guidelines for the analysis of cable loss hazard in cable supported structures

Guidelines for the design of cable-supported bridges against the hazard of cable loss event are reported in PTI recommendations of Post-tensioning Institute (Post-Tensioning Institute (2007)) and part 11 of Eurocode 3 (European Committee for Standardization (2006)).

The PTI recommendations deal with cable-stayed bridges only and prescribe the use of the following load combination for the loss of one cable:

$$1.1DC+1.35DW+0.75(LL+IM)+1.1CLFD \tag{1}$$

where, DC is the contribution of structural and non-structural dead loads, DW is the dead load of wearing surfaces and utilities, LL is the full vehicular live load, IM is the vehicular dynamic load allowance and CLDF is the impact effect due to the cable failure. The structure subjected to cable loss can be analyzed by means of two methods: the

first consists of a nonlinear dynamic analysis of the structure without the broken cable under the action of full dead and live loads. Despite the effectiveness of the approach to properly simulate the structural response, limited details are reported on how performing such dynamic analysis. The second method is based on a simplified static analysis of the structure without the broken cable and subjected to dead loads, live loads, and two additional static forces, which account for the impact dynamic effect produced by the sudden loss. The static forces shall be applied at the anchorage of the broken cable and their entity should be equal to that of the cable prior to the loss, considering the effect of dead and live loads, and amplified by a factor of 2. The sudden loss effect is reproduced by orienting the forces oppositely to supporting action provided by the cable prior to the loss.
Eurocode 3 reports a general method for the analysis of cable structures affected by the sudden loss of cable elements. Contrarily to PTI recommendations, the method proposed by EC3 can be applied to any kind of cable structure and, consequently, to any cable-supported bridges. The effect produced by the sudden loss (E_d) is quantified by means of the following expression:

$$E_d = k(E_{d2} - E_{d1}) \tag{2}$$

where k is equal to 1.5, and E_{d1} and E_{d2} represent the design effects with all cable intact and with the relevant cable removed, respectively. E_{d1} and E_{d2} can be evaluated by using a quasi-static analysis of the structure subjected to dead and possible live loads. Alternatively to the simplified method, EC3 allows to employing an advanced numerical analysis to evaluate the effects produced by the cable loss. However, recommendations on how to perform this analysis are not reported. Although not prescribed explicitly in the code, the extreme event caused by the loss of one cable should be investigated by adopting the load combination of accidental design situations reported in EC0 (European Committee for Standardization (2002)) defined as follows:

$$\sum_{j\geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1} + \sum_{i>1} \psi_{2,1} Q_{k,i} \tag{3}$$

where "+" must intended as "*to be combined with*". In Eq.(3), $G_{k,j}$ is the characteristic value of permanent action, P accounts for the relevant representative value of a pre-stressing action, A_d is the design value of an accidental action, and $\psi_{k,j}\ Q_{k,j}$ are the contribution of variable loads multiplied for the relative combination factor.

3. Theoretical formulations and numerical implementation

The structural behavior of network arch bridges is analyzed by means of a 2D FE numerical model, in which a damage model is implemented for simulating the sudden loss of one or multiple hangers. Furthermore, a refined formulation is employed to accurately quantify dynamic amplification effects induced by moving loads. The study is performed with reference to the network arch bridge scheme depicted in Fig 1-a.

The arch rib (R) and the tie girder (T) are made of steel and have rectangular hollow cross-sections, whose width and height are denoted by $B^{R(T)}$ and $H^{R(T)}$, respectively. The cable system consists of two specular sub-systems, each made of $m/2$ hangers inclined of α with respect to the horizontal. This configuration guarantees intermediate supports for the girder with a step equal to $p = L/(m + 2)$. The hanger cross-sections are dimensioned according to design rules typically adopted in the design of cable supported bridges, as reported in (Lonetti and Pascuzzo (2016, 2014a)). Finally, the bridge presents external boundary conditions, which are clamped or simply supported for each cross-section extremities.

3.1. Numerical model

The network arch bridge described above is modeled by using Timoshenko beam elements for arch ribs and tie girders, whereas truss elements are adopted for the hangers. In particular, every single hanger is subdivided into a series of elements according to the Multi-Element Cable System (MECS) approach, which permits to take into account for local and global vibrations of cables. The hangers are connected to arch rib and tie girder by using explicit constraint equations defined at cable anchorage nodes. The material behavior is assumed to be linear elastic. The governing equations are solved numerically by using a user customized finite element program, that is, COMSOL Multiphysics

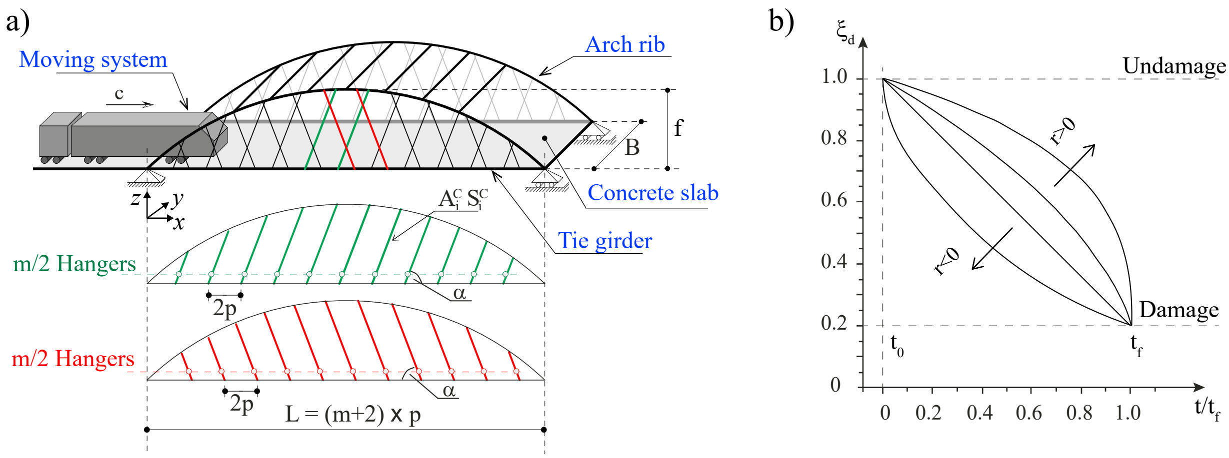

Fig. 1. (a) Structural scheme of the network arch bridge; (b) Damage law for cable element

version 5.4 (COMSOL (2018)). The governing equations of the FE model are not reported here for the sake of brevity. However, detailed descriptions of the governing equations can be found in (Lonetti et al. (2016); Lonetti and Pascuzzo (2014a)). The identification of the initial configuration of the bridge structure under the action of dead and permanents loads represents the preliminary step in the numerical investigation. The main aim consists to evaluate the distribution of the initial stress in hangers, arch rib, and tie girder in order to minimize the initial deformation of the structure. In the present study, the initial configuration of the structure is defined by using a numerical procedure consistently with the "*zero displacement method*" approach, which is usually adopted in the framework of long-span cable supported bridge. Exhaustive details regarding the theory and the numerical implementation of the procedure for the identification of the initial configuration can be found in (Bruno et al. (2016); Lonetti and Pascuzzo (2014b)).

3.2. Damage model for cable loss

The sudden loss of a cable is simulated by using a time dependent damage law, which reduces the mechanical properties and initial stress of a cable element in the failure time domain, defined by the initial (t_0) and the final (t_f) times of the failure. The damage law used in the present study is based on a Kachanov's law, which consists of a damage variable defined by the following expression (Bruno et al. (2018))

$$\xi(t) = \left(1 - \frac{t - t_0}{t_f}\right)^{\frac{1}{r+1}} \tag{4}$$

where r is an asymptotic parameter of the damage, which controls the evolution of the damage function. This is linear for r equal to zero and convex or concave for positive or negative values of the exponential parameter, respectively. From the practical point of view, the value of the parameter r in the damage definition is typically assumed close equal to 0.98. Eq.4 is represented in Fig.1-b. In the numerical model, the time-dependent damage law (Eq. 4) reduces the Young's Modulus and initial stress of the hanger that will break during numerical simulation, leaving the remaining ones integers. The broken hanger represents an input variable of the problem and no progressive collapse are considered in the numerical simulation. This aspect allows to investigate the re-distribution of internal stresses in the undamaged elements due to the sudden loss of the element under investigation, thus identifying the worst unsafe damage scenario.

3.3. Moving loads formulation

The structural behavior of network arch bridges under the action of moving loads is investigated by means of static and dynamic analysis methods. In the case of the simplified approaches described in Section 2, a static analysis is

employed and the structure is investigated considering several load arrangements. The dynamic effect is considered in the calculation by means of dynamic amplification factors, which typically are provided by codes for the most common bridge structures.
In the case of dynamic analysis, the structure is analyzed considering the moving loads proceeded, at constant speed (c), from left to right along the bridge development. The moving load refers to railway vehicle loads, which are reproduced by means equivalent uniformly distributed loads. The model accounts for the interaction due to coupling behavior between the mass of the moving system and bridge deformations (Lonetti et al. (2016)). The interaction produces nonstandard inertia actions arising from Coriolis and centripetal accelerations, which produce relevant amplification effects for high speeds. In particular, with reference to a fixed reference system, velocity and acceleration functions of the moving system were evaluated by means of an Eulerian description, as:

$$\dot{v} = \frac{\partial v}{\partial t} + c\frac{\partial v}{\partial x},\ \ddot{v} = \frac{\partial^2 v}{\partial t^2} + 2c\frac{\partial^2 v}{\partial x \partial t} + \frac{\partial^2 v}{\partial x^2},\ with\ c = \frac{\partial x}{\partial t} \tag{5}$$

4. Results

Numerical results are proposed to investigate the behavior of network arch bridges subjected to the sudden loss of a hanger of the cable system under the action of moving loads. The study is developed with reference to a steel network arch bridge with span length (L), rise (f), and width (B) equal to 180 m, 30 m, and 10 m, respectively. The cable system is composed of 34 hangers arranged in two specular sub-systems. Each sub-system comprises 17 hangers equally spaced of 10 m along the girder, and inclined about 65° with respect to the horizontal. The combination of the two sub-systems guarantees intermediate supports for the tie girder every 5 m. Cross-section properties of the arch rib, tie girder, and hangers are reported in Table 1.

The structure is investigated by means of a nonlinear dynamic analysis, in which the moving load is assumed to proceed on the bridge deck at constant speed (c) from left to right. The intensity of the moving load is equal to 80 kN/m, identical to the LM-71 train model reported in (European Committee for Standardization (2003)), whereas the extension (L_p) is about 60 m (*i.e.* L_p/L=1/3). The sudden loss event is assumed to occur when the moving system is located at the left-extremity of the bridge. Girder deformations, calculated by using the proposed numerical model, are compared with the ones predicted by the simplified static approaches proposed by PIT (Post-Tensioning Institute (2007)) and EC3 (European Committee for Standardization (2006)), (see Section 2).
Since every hanger of the cable system could be affected by sudden failure mechanisms under extreme loading conditions, a preliminary study was conducted with the aim to identify the most dangerous loss scenario for the bridge structure. The study was performed by analyzing the structure subjected to the loss of every single hanger of the cable system with the exceptions of the ones located close to the bridge extremities. The most dangerous scenario is assumed as the one that generates the maximum vertical displacement of the girder. It is worth noting that, the loss events are examined considering the structure subjected to dead and permanent loads only since the evaluation of amplification effects induced by moving loads is not necessary for the purpose of the present analysis. As a matter of fact, moving loads should amplify the effects produced by cable loss under the action of dead loads only. Then, the worst cable loss scenario for the structure under the action on dead loads only should be the same as that originated by moving load actions. For this reason, moving loads are not considered here thus providing considerable computational savings.
Figure 2 depicts the maximum vertical displacements of the girder produced by the sudden loss of every single hanger of the structure. The results show that, side zones of the bridge, *i.e.* close to x=2/5 L and x=3/4 L, are the most vulnerable ones. In this framework, the loss of the hanger 24 located close to 3/4 L caused the most dangerous scenario

Table 1. Section properties of the arch bridge scheme utilized in the study

Structural element	B (m)	H (m)	t_w (mm)	t_f (mm)	A (m^2)	I (m^4)
Arch rib	0.675	1.85	60	60	0.2886	0.1166
Tie girder	1.56	2.00	130	130	0.0918	0.0567
Hangers					0.00238	

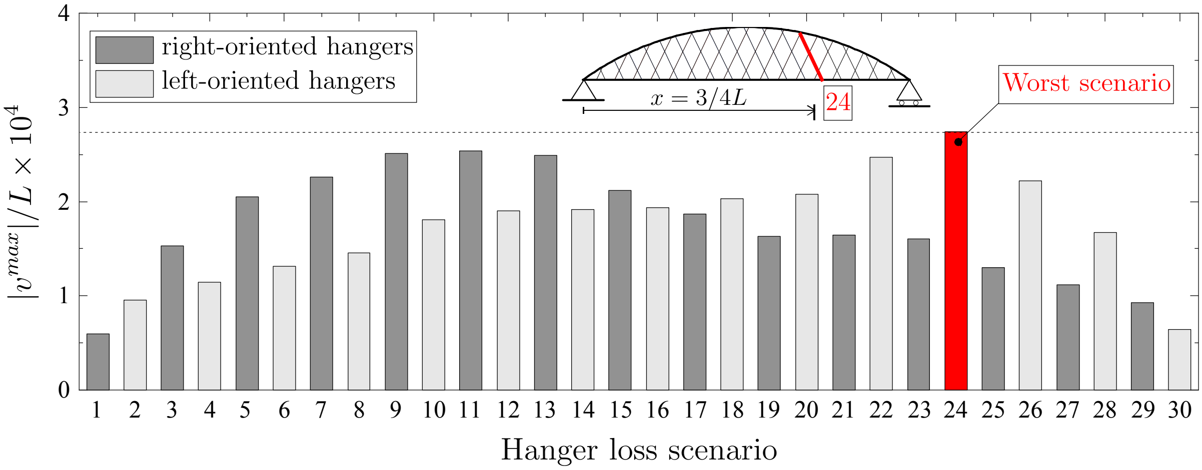

Fig. 2. Identification of the most dangerous cable loss event for the bridge structure in terms of girder vertical displacements

for the structure.
This scenario is further analyzed considering the effect of moving loads. In particular, the structure is investigated by using a standard analysis (SA), i.e. considering only the effects produced by the moving system weight, and a non-standard analysis (NSA), which accounts for inertia effects arising from nonstandard accelerations (see Section 3.3). Furthermore, comparisons results are developed between damage (D) and un-damage (UD) bridge configurations in order to compare the amplification effects induced by the moving loads only (i.e. in the un-damage structure) and the ones produced by the combination of moving loads and the sudden loss event.

The results reported in Fig. 3 a and b show the time histories of the vertical displacements of the girder at x=3/4L for moving load speeds of c=50 m/s and c=100 m/s, respectively. In both figures, the curves refer to the following analysis cases:

- UD-SA: Un-damage structure (UD) analyzed by Standard Analysis (SA);
- UD-NSA: Un-damage structure (UD) analyzed by Nonstandard Analysis (NSA);
- D-SA: Damage structure (D) analyzed by Standard Analysis (SA);
- D-NSA: Damage structure (D) analyzed by Nonstandard Analysis (NSA);

For c=50 m/s (Fig.3-a), the results show that amplification effects induced by nonstandard inertia actions of the moving load are quite limited since vertical displacements obtained from SA and NSA are comparable. The sudden loss of the hanger n.24 increases considerably the maximum vertical displacement of the girder, which becomes quite higher than the threshold value of L/800 usually prescribed by design codes under service conditions. On the other hand, for c=100 m/s (Fig.3-b), inertia effects of the moving system affect considerably the bridge structure, which undergoes relevant vibrations and deflections. In this context, the maximum vertical displacement obtained for NSA-D is considerable higher than the limiting values of L/800, thus representing a notable unsafe condition.
This is also observed from the results relative to UD-NSA, which denote how excluding interaction phenomena between moving system and bridge kinematic may lead to larger underestimations of the structural response. With the aim to quantify amplification effects induced by nonstandard inertia contributions, dynamic amplification factors (DAFs) for the girder vertical displacement at x=3/4L is evaluated by means of SA and NSA analyses. In particular, the following expression for the DAF is adopted:

$$DAF(X) = \frac{X_{UD}^{dyn}}{X_{UD}^{st}} \tag{6}$$

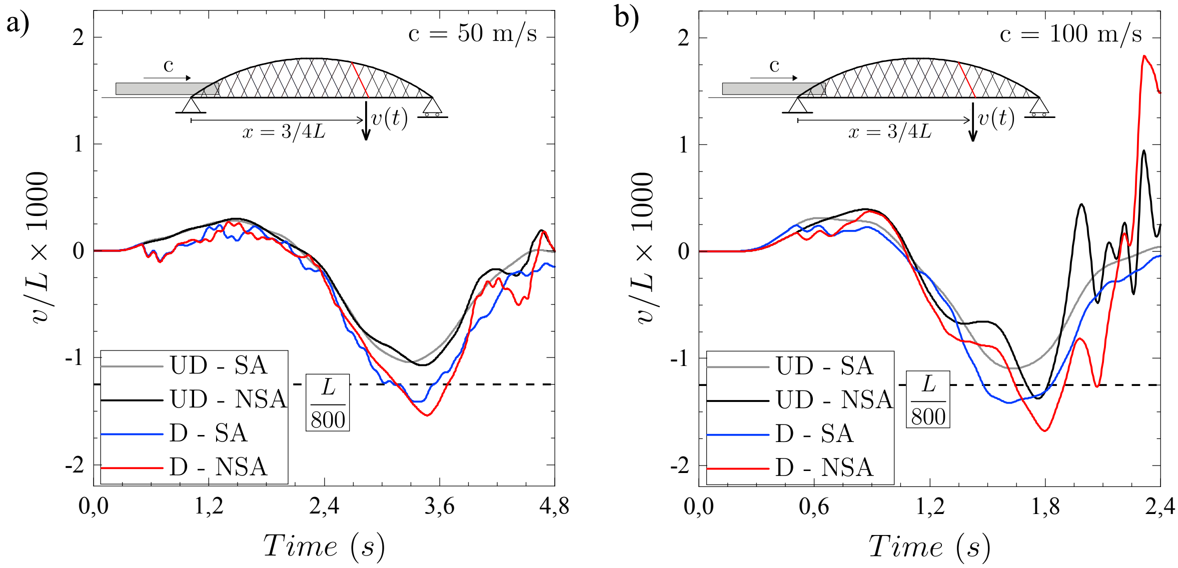

Fig. 3. Comparisons in terms of vertical displacements of the girder at x=3/4L between predictions of Standard (SA) and nonstandard (NSA) analyses in the cases of Damage (D) and Undamage (UD) events of hanger 24: moving load speeds of (a) 50 m/s, (b) 100 m/s

where X_{UD}^{dyn} and X_{UD}^{st} represent the values of the variable X evaluated with reference to the un-damage structure (UD) by means of dynamic (dyn) and static (st) analyses, respectively.
The results reported in Fig.4 show that nonstandard inertia effects increases the vertical displacements for c=120 m/s with a factor approximately of 2.34, whereas only 1.15 is predicted by means of traditional SA analysis.

This aspect may significantly affect the predictions that can be obtained by using the simplified approaches proposed by codes (see section 2), because the nominal value of LL amplified by DAFs is usually adopted to account for

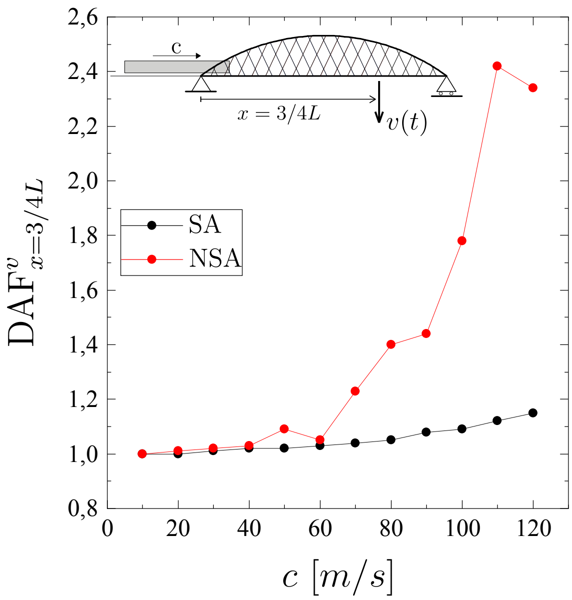

Fig. 4. Dynamic amplification factor (DAF) of the girder vertical displacement at x=3/4 L

the moving load effects. The simplified approaches estimate the impact action produced by the sudden loss by means of empirical methods and combine corresponding effect with the ones produced by dead and live loads (evaluated on the integer structure). In order to assess the accuracy of the simplified approach, the sudden loss of hanger n.24 is investigated by using methodologies proposed by PTI and EC3 and the results are compared with the ones obtained by means of SA and NSA analyses. In particular, the comparisons were performed considering un-factored load combinations, thus giving more emphasis to the consistency of the methodologies. The operative steps of the PTI and EC3 approaches are implemented in a user MatLab script, which interacts with Comsol Multiphysics by means of the LiveLink for Matlab platform. Description of the steps involved are described in Table 2.

Note that, the steps used for both PTI and EC3 are denoted by "PTI & EC3", whereas the ones relatives exclusively to PTI and EC3 are marked by "only PTI" and "only EC3", respectively. It is worth nothing that PTI and EC3 evaluate the dynamic amplification factor for moving load (point 1.2) differentially.

PTI guidelines prescribe to quantify the vehicular dynamic load allowance (IM in Eq.1) consistently with AREA specifications (American Railway Engineering Association, (AREA) (1996)), whereas EC1 (European Committee for Standardization (2003)) defines an amplification factor. For the structure under investigation, the amplification factors for AREA and EC3 are evaluated to be 1.17 and 1.27, respectively.

Comparison results between the simplified approaches and nonlinear dynamic analyses (i.e. SA and NSA) are developed in terms of the maximum deflection of the girder at x=3/4 L obtained for increasing values of moving loads speed (c) (Fig. 5-a). The results show that both approaches proposed by PTI and EC3 underestimate the maximum deflection of the girder for moving load speeds larger than 60 m/s and 90 m/s, respectively, thus being unreliable to predict the structural behavior of the bridge under the actions of high-speed trains. It may be reasonable to suppose that, the DAFs proposed by both the codes do not account for dynamic amplifications induced by nonstandard inertia forces arising from Coriolis and centripetal accelerations. As a matter of fact, the maximum vertical displacements obtained by the simplified approaches are comparable to the results obtained by SA analysis. The PTI and EC3 approaches were re-calculated by using the DAF reported in Fig.4 and the results are compared with NSA estimations (Fig.5-b). The results denote that the simplified approach provide acceptable prediction of the sudden loss effect, since the deflections are slightly higher than that obtained by NSA. These results revealed that the simplified approaches may guarantee acceptable evaluations of structural behavior of network arch bridges due to hanger loss events exclusively if proper DAFs for the main kinematic and stress design variables are employed. The DAFs should account for contributions arising from nonstandard terms of the moving loads.

Table 2. Description of the steps involved in the simplified analyses proposed by PTI and EC3

1. START
2. Analysis of the undamaged structure
    2.1. Define the initial configuration of the structure
    2.2. Set the Dynamic Amplification Factor for moving loads action (PTI & EC3)
    2.3. Run the analysis (PTI & EC3)
    2.4. Evaluation of internal stresses in the structure: E_{d2} (PTI & EC3)
    2.5. Evaluation of the static forces in the hanger that will break (only PTI)
3. Analysis of the damaged structure
    3.1. Define the initial configuration of the structure
    3.2. Remove the broken cable (PTI & EC3)
    3.3. Apply the static forces evaluated in step 1.4 amplified by a factor of 2. (only PTI)
    3.4. Run the analysis (PTI & EC3)
    3.5. Evaluation of internal stresses in the structure: E_{d1} (PTI & EC3)
4. Quantify the effects of cable loss only
    4.1. Evaluate E_p:=E_{d1}-E_{d2} (EC3 only)
    4.2. Subtract the effects of the initial configuration from E_{d1}: E_{dPTI} (PTI only)
    4.3. Subtract the effects of the initial configuration from E_{d3}: E_{dEC3} (EC3 only)
5. Combination of the effects
    5.1. Combine E_{d2} with E_{dPTI} (PTI only)
    5.2. Combine E_{d2} with 1.5 × E_{dEC3} (EC3 only)
6. END

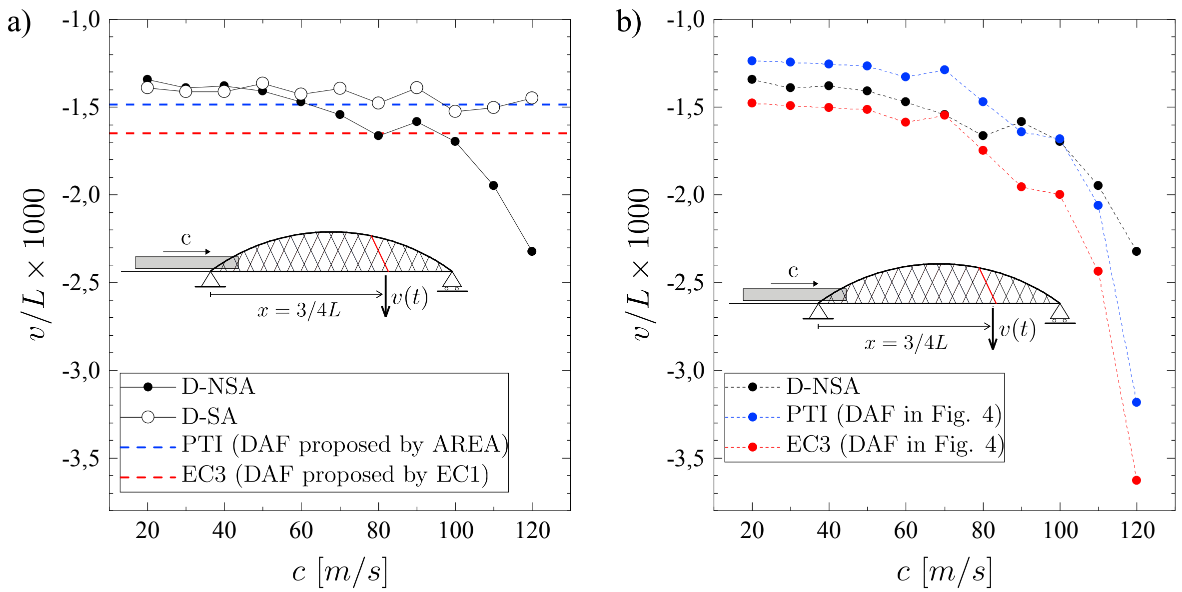

Fig. 5. Comparisons between nonlinear dynamic analyses and simplified approaches proposed by PTI (Post-Tensioning Institute (2007)) and EC3 (European Committee for Standardization (2006)). Estimations obtained by using simplified approaches Dynamic Amplification Factor proposed by (a) AREA (American Railway Engineering Association, (AREA) (1996)) and EC1 (European Committee for Standardization (2003)) and (b) those reported in Fig. 4

5. Conclusion

Network arch bridges are potentially exposed to unsafe conditions caused by cable loss hazards. As a matter of fact, moving loads subject the hangers of the cable system to considerable tractions and vibrations that may leads to yield or fatigue failure mechanisms. An accurate evaluation of the structural behavior produced by a cable loss event under the action of moving loads is then required to predict the effective distributions of stresses and deformations in the structure, thereby designing more robust and safety structures. In particular, the moving loads action should consider the contribution of nonstandard inertia forces arising from Coriolis and centripetal accelerations. The nonstandard effects involve considerable amplifications of stresses and deformations for increasing transit speeds. The results have shown that standard analyses, which usually do not consider the effect of nonstandard accelerations, involve notable underestimations in the bridge kinematic response. This aspect becomes more evident when the bridge structure is affected by cable loss events. The main codes on cable supported bridges report simplified quasi-static approaches for the quantification of the effects induced in the structure by the sudden loss of a hanger. The results highlight that these methods could provide acceptable estimations of the bridge response if proper dynamic amplifications factors for the moving loads are considered. In particular, the factors should account for amplifications induced by nonstandard contributions of the moving loads. Unfortunately, current codes on bridges structures provide proper values of amplification factors exclusively for modest bridge structures, such as simply supported or multiple span bridges, whereas not exhaustive guidelines are reported for network arch bridges.

References

American Railway Engineering Association, (AREA), 1996. Manual for railway engineering. [1996 ed.] ed., Washington D.C. : The Assocation. "Current until July 31, 1997"–Back cover.

Bruno, D., Lonetti, P., Pascuzzo, A., 2016. An optimization model for the design of network arch bridges. Computers and Structures 170, 13 – 25. doi:https://doi.org/10.1016/j.compstruc.2016.03.011.

Bruno, D., Lonetti, P., Pascuzzo, A., 2018. A numerical study on network arch bridges subjected to cable loss. International Journal of Bridge Engineering 6, 41–59.

COMSOL, 2018. COMSOL Multiphysics Reference Manual. COMSOL AB.

European Committee for Standardization, 2002. EN 1990:2002: Eurocode 0. Basis of structural design. European Committee for Standardization.

European Committee for Standardization, 2003. EN 1991-2:2003: Eurocode 1: Actions on structures. Part 3: Traffic Loads on Bridges. European Committee for Standardization.

European Committee for Standardization, 2006. EN 1993-1-11:2006: Eurocode 3. Design of steel structures. Design of structures with tension components. European Committee for Standardization.

Funari, M.F., Greco, F., Lonetti, P., 2018. Sandwich panels under interfacial debonding mechanisms. Composite Structures 203, 310 – 320. doi:https://doi.org/10.1016/j.compstruct.2018.06.113.

Funari, M.F., Lonetti, P., Spadea, S., 2019. A crack growth strategy based on moving mesh method and fracture mechanics. Theoretical and Applied Fracture Mechanics 102, 103 – 115. doi:https://doi.org/10.1016/j.tafmec.2019.03.007.

Greco, F., Lonetti, P., Pascuzzo, A., 2013. Dynamic analysis of cable-stayed bridges affected by accidental failure mechanisms under moving loads. Mathematical Problems in Engineering 2013. doi:10.1155/2013/302706.

Greco, F., Lonetti, P., Pascuzzo, A., 2018. A moving mesh fe methodology for vehicle–bridge interaction modeling. Mechanics of Advanced Materials and Structures 0, 1–13. doi:10.1080/15376494.2018.1506955.

Greco, F., Lonetti, P., Pascuzzo, A., 2019. Structural integrity of tied arch bridges affected by instability phenomena. Procedia Structural Integrity 18, 891 – 902. doi:10.1016/j.prostr.2019.08.240. 25th International Conference on Fracture and Structural Integrity.

Harshil, J., Jignesh, A., 2017. Analysis of cable-stayed bridge under cable loss 5, 61–78.

Lonetti, P., 2010. Dynamic propagation phenomena of multiple delaminations in composite structures. Computational Materials Science 48, 563 – 575. doi:https://doi.org/10.1016/j.commatsci.2010.02.024.

Lonetti, P., Maletta, R., 2018. Dynamic impact analysis of masonry buildings subjected to flood actions. Engineering Structures 167, 445 – 458. doi:https://doi.org/10.1016/j.engstruct.2018.03.076.

Lonetti, P., Pascuzzo, A., 2014a. Optimum design analysis of hybrid cable-stayed suspension bridges. Advances in Engineering Software 73, 53 – 66. doi:https://doi.org/10.1016/j.advengsoft.2014.03.004.

Lonetti, P., Pascuzzo, A., 2014b. Vulnerability and failure analysis of hybrid cable-stayed suspension bridges subjected to damage mechanisms. Engineering Failure Analysis 45, 470 – 495. doi:https://doi.org/10.1016/j.engfailanal.2014.07.002.

Lonetti, P., Pascuzzo, A., 2016. A numerical study on the structural integrity of self-anchored cable-stayed suspension bridges. Frattura ed Integrita Strutturale 10, 359–376. doi:10.3221/IGF-ESIS.38.46.

Lonetti, P., Pascuzzo, A., 2019. A practical method for the elastic buckling design of network arch bridges. International Journal of Steel Structures doi:10.1007/s13296-019-00282-8.

Lonetti, P., Pascuzzo, A., Aiello, S., 2019. Instability design analysis in tied-arch bridges. Mechanics of Advanced Materials and Structures 26, 716–726. doi:10.1080/15376494.2017.1410911.

Lonetti, P., Pascuzzo, A., Davanzo, A., 2016. Dynamic behavior of tied-arch bridges under the action of moving loads. Mathematical Problems in Engineering 2016. doi:10.1155/2016/2749720.

Mahmoud, K., 2007. Fracture strength for a high strength steel bridge cable wire with a surface crack. Theoretical and Applied Fracture Mechanics 48, 152 – 160. doi:10.1016/j.tafmec.2007.05.006.

Materazzi, A.L., Ubertini, F., 2011. Eigenproperties of suspension bridges with damage. Journal of Sound and Vibration 330, 6420 – 6434. doi:https://doi.org/10.1016/j.jsv.2011.08.007.

Pellegrino, C., Cupani, G., Modena, C., 2010. The effect of fatigue on the arrangement of hangers in tied arch bridges. Engineering Structures 32, 1140 – 1147. doi:https://doi.org/10.1016/j.engstruct.2009.12.040.

Post-Tensioning Institute, 2007. Recommendations for stay cable design, tensting and installation. Post-Tensioning Institute.

Sophianopoulos, D.S., Michaltsos, G.T., Cholevas, H.I., 2019. Static and dynamic responses of suspended arch bridges due to failure of cables. Archive of Applied Mechanics 89, 2281–2312. doi:10.1007/s00419-019-01576-3.

Sun, H., Xu, J., Chen, W., Yang, J., 2018. Time-dependent effect of corrosion on the mechanical characteristics of stay cable. Journal of Bridge Engineering 23, 04018019. doi:10.1061/(ASCE)BE.1943-5592.0001229.

Wickramasinghe, W.R., Thambiratnam, D.P., Chan, T.H., 2020. Damage detection in a suspension bridge using modal flexibility method. Engineering Failure Analysis 107, 104194. doi:https://doi.org/10.1016/j.engfailanal.2019.104194.

Wolff, M., Starossek, U., 2009. Cable loss and progressive collapse in cable-stayed bridges. Bridge Structures 5, 17–28. doi:10.1080/15732480902775615.

Wu, G., Qiu, W., Wu, T., 2019. Nonlinear dynamic analysis of the self-anchored suspension bridge subjected to sudden breakage of a hanger. Engineering Failure Analysis 97, 701 – 717. doi:https://doi.org/10.1016/j.engfailanal.2019.01.028.

Zhou, Y., Chen, S., 2015. Numerical investigation of cable breakage events on long-span cable-stayed bridges under stochastic traffic and wind. Engineering Structures 105, 299 – 315. doi:https://doi.org/10.1016/j.engstruct.2015.07.009.