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A NUMERICAL STUDY ON NETWORK ARCH BRIDGES SUBJECTED TO CABLE LOSS

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ABSTRACT: A numerical study is proposed to investigate the structural behavior of network arch bridges subjected to the cable loss accidental event. The main aim of the paper is to analyze the effects produced by potential cable loss scenarios on the main stress and kinematic design variables of the structure. To this end, a parametric study in terms of the main structural and geometric parameters is developed with the purpose to identify the key factors that contribute to reduce hazards produced by the cable loss event. The structural analyses are performed by using 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 the cable is reproduced by means of a proper damage law, developed in the framework of Continuum Damage Mechanics theory. Cable loss 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 are discussed.

KEYWORDS: Network arch bridges; Cable loss; Nonlinear analysis; Finite elements.

1 INTRODUCTION

Network arch bridges represent a valid solution in the field of short and medium spans, since they combine both structural and aesthetic advantages with respect to conventional bridge typologies [1, 2]. The structure consists of two parallel arch ribs that support a lower deck by means of several hanger cables. The hanger cables run diagonally and cross each other at least two times. This hanger arrangement provides a considerable degree of robustness to the entire structure. However, corrosion or fatigue phenomena may cause a loss of tension in the elements of the cable system. The loss of cables is an unsafe condition that may generate the collapse of the entire structure, causing social-economic damages. The cable loss is an accidental situation that should be taking into account in the design of network arch bridges. However, current codes on bridges do not provide explicit design guidelines to address the cable loss accidental event for network arch bridges. As a matter of fact, Eurocode 3 [3] reports a simplified method that can be applied to any cable structures to analyze the sudden loss of cable elements. 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 [4] of the Post-Tensioning Institute provide prescriptive guidance on the cable loss extreme event, but exclusively with reference to cable-stayed bridges. In the literature, several works have been developed to investigate the behavior of bridge structures subjected to the cable loss accidental event. A number of studies have analyzed the damage mechanisms of cable elements by means of experimental works and numerical simulations [5-9]. Analyses on the structural behavior of cable supported bridges subjected to cable loss have been performed mainly for cable-stayed configurations. Wolff and Starossek [10, 11] have developed nonlinear dynamic analyses on cable-stayed bridges subjected to the loss of multiple cables, concluding that the loss of more than two cables may lead to the overall collapse of the structures. Mozos and Aparicio [12, 13] have investigated the structural behavior of cable-stayed bridges by means of nonlinear dynamic analyses and simplified methodologies reported in the main code on cable supported bridges. They have revealed that simplified methodologies lead to unsafe results. Jani and Amin [14] have investigated the effects of cable loss due to increasing corrosion as well as sudden cable loss on fan and semi-fan type cable-stayed bridges. Greco, et al. [15] have evaluated the dynamic amplification effects on cable-stayed bridges produced by the cable loss and the moving load actions. In particular, the moving load action has been simulated taking into account for nonstandard inertial forces involved in the moving system mass description arising from Coriolis acceleration and centripetal acceleration. Zhou and Chen [16] have considered the effects produced by the cable loss and the traffic and wind loads. In particular, in order to simulate the traffic load in a more realistic manner, they considered stochastic moving traffic loads. Few studies have been published on others cable supported bridge configurations. The structural behavior of self-anchored suspension bridges subjected to the sudden loss of cables has been investigated by Qiu, et al. [17]. Lonetti and Pascuzzo [18] have analyzed the structural performance of hybrid cable-stayed suspension bridges subjected to damage mechanisms. They have revealed that the hybrid scheme guarantees a proper degree of robustness and safety against the cable loss event with respect to conventional suspension bridges. As a matter of fact, they have found that the effects produced by the cable loss is considerable reduced because of the coupled behavior between the cable-stayed and suspension systems.

Despite the great efforts developed to investigate the cable loss event on cable supported bridges, network arch bridges have received less attention and, to the Author’s knowledge, no detailed work is reported in the literature. Then, the main aim of the present paper is to analyze the structural behavior of network arch bridges subjected to the sudden loss of cables. At first, the evaluation of the effects produced by the cable loss event on the main stress and kinematic design variables of the structure is performed. Then, the identification of the structural parameters that may contribute to improve the degree of robustness and safety of the structure is developed. To this end, a generalized FE model and a proper damage law for cable elements are developed to perform advanced nonlinear dynamic analyses. Comparisons between nonlinear dynamic analysis and simplified methodologies prescribed by codes are developed. The main aim is to assess the applicability of the simplified methodologies in the case of network arch bridges.

The outline of the paper is as follows: in Section 2, a review of the simplified methodologies proposed by codes to analyze the cable loss accidental event is presented. The numerical implementation is reported in Section 3, whereas in Section 4 numerical and parametric results are proposed.

2 A REVIEW OF PRESCRIPTIONS PROVIDED BY CODES ON THE CABLE LOSS ACCIDENTAL DESIGN SITUATION

From a design point of view, network arch bridges are able to withstanding the loss of cables without the occurrence of the structural collapse. Unfortunately, current codes on cable supported bridges do not provide any specific prescriptions to assess the cable loss accidental event. Such analysis is treated by part 11 of Eurocode 3 [3] and by PTI recommendations of the Post-Tensioning Institute [4].

The PTI recommendations provide guidelines to perform the analysis of cable-stayed bridges subjected to the loss of one cable only. The cable loss accidental event can be investigated by means of two methods: The first method consists in a static analysis of structure, without the rupture cable, subjected to dead and possible live loads together with static forces, which takes account for the impact dynamic force resulting from the sudden failure. The magnitude of such static forces is equal to that of the cable prior the failure and, it shall be applied at the anchorage locations of the rupture cable. The second method is a nonlinear dynamic time-history analysis in which the structure is analyzed considering the full dynamic effect induced by the cable loss. However, PTI recommendations provide not exhaustive advices in how to perform such analysis. Moreover, it is specified that the effects due to the abrupt cable loss quantified by means of the dynamic analysis should be equal of 50% obtained by means of a static analysis. Finally, PTI recommendations provide a specific load combination for the analysis of the accidental situation caused by the cable loss:

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

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.

Eurocode 3 treats the replacement and loss of cable components in part 11 [3]. Contrarily to PTI recommendations, EC3 describes a general method to evaluate the effects produced by the sudden loss of cables, which may applied to any type of cable supported bridge. The dynamic effect of the sudden loss of cables (E_d) can be evaluated by means of the following expression:

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

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. In particular, E_{d1} and E_{d2} can be evaluated by means of a quasi-static analysis of the structure subjected to dead and possible live loads. Alternatively, to the simplified method, EC3 allows the use of advanced numerical analysis to evaluate the effects produced by the cable loss. However, recommendations on how to perform these advanced analyses are not reported. Finally, EC3 postpones load combinations and values of the partial coefficients to the National Annex.

3 NUMERICAL MODEL

The bridge scheme, reported in Fig.1, refers to a steel network arch bridge in which the arch and the girder are fixed at their extremities.

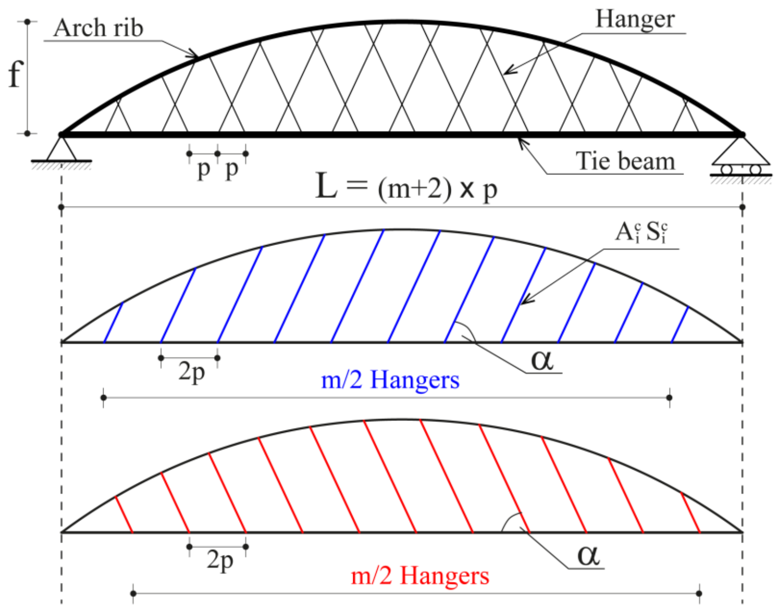


Figure 1. Structural scheme of the network arch bridge

The arch sustains the girder by means of the cable system, which is formed by a total of *m* hangers inclined of α. The hangers are arranged in two specular sub-systems of *m*/2 elements. The combination of the two sub-systems guarantees intermediate supports for the girder with a step equal to $p = L/(m+2)$. Finally, the bridge presents external boundary conditions, which are clamped or simply supported for each cross-section extremities.

The structural behavior of the bridge is analyzed by means of a 2D finite element model, as shown in Fig. 2. In order to reduce computational efforts, the arch and the girder are modelled as beam elements, whereas, hangers are modelled by means of truss elements. In particular, each single hanger is composed of a series of truss elements according to the Multi Element Cable System (MECS) approach, which permits to take into account for local and global vibrations of cables. Both arch and girder are connected to the cable system by means of explicit constraint equations defined at interceptions nodes of beams and truss elements. Each element of the model is characterized by means of cross-section and material properties. The material behavior is assumed to be linear elastic.

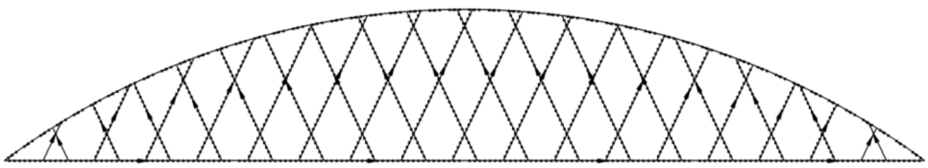


Figure 2. Finite element modeling of the structural scheme

3.1 Initial configuration of the bridge

Since the bridge deformability is mostly influenced by the cable system stiffness, the definition of the initial geometrical and stress configuration of hangers under the action of permanent and dead loads is an essential step that has to be performed before any other analysis. From a design point of view, it is required to identify cable stresses distribution and geometric shapes of the arch and the girder to obtain an undeformed configuration of the structure under the action of dead and permanent loads. Such an objective is achieved in the present study by means of a numerical procedure based on the “zero displacement method” [19, 20], which consists to solve a constrained optimization problem in terms of stress and displacements conditions. In particular, with reference to the network arch bridge scheme reported in Fig.3, with *m* number of hangers, the objective functions, which are minimized during the optimization procedure, are represented by the vertical displacements of the hangers at girder anchorages, $(U^C_{Z,1}, U^C_{Z,2}, U^C_{Z,3}, \ldots, U^C_{Z,\mathrm{m}})$, the horizontal displacement of the girder at x=L $U^G_{X,X=L}$, and the vertical displacement of the top arch cross-section $U^A_{Z,X=L/2}$.

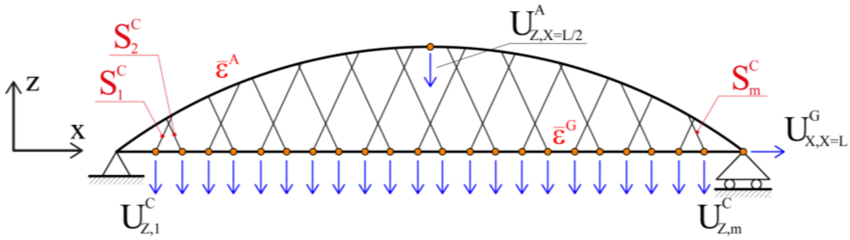


Figure 3. Displacement and stress variables for the evaluation of the initial configuration under dead and permanent loads

The design variables are the initial stress of the hangers $(S_1^C, S_2^C, S_3^C, \ldots, S_m^C)$ and the global average axial deformations of arch and girder, that is $\bar{\varepsilon}^A = {}^1\!/_{L^A}\int_0^{L^A}\varepsilon(s)ds$ and $\bar{\varepsilon}^G = {}^1\!/_{L^G}\int_0^{L^G}\varepsilon(s)ds$, respectively. Therefore, the required equations to achieve the initial configuration are expressed as follows:

$$\begin{aligned}
&\underset{\sim}{L}_C\left[(S_1^C, S_2^C, \ldots, S_m^C), (U_{Z,1}^C, U_{Z,2}^C, U_{Z,3}^C, \ldots, U_{Z,\mathrm{m}}^C)\right] = \underset{\sim}{0} \\
&L_A\left[\bar{\varepsilon}^A, U_{Z,X=L/2}^A\right] = 0 \\
&L_G\left[\bar{\varepsilon}^G, U_{\mathrm{X},X=L}^G\right] = 0
\end{aligned} \tag{3}$$

where L_C, L_A, L_G are operators, which collect variables concerning the stiffness properties of the cable, the arch and girder, respectively. Moreover, in order to avoid convergence problems arising from nonlinearities involved in the cable system, Eq.(3) is solved numerically by means of an iterative procedure. In particular, each design variable X is expressed as the additive combination of an initial constant value (X_0), and an incremental one (ΔX_0), which is evaluated during the iteration steps.

3.2 Damage formulation for cable elements

The cable loss is reproduced by means of a time dependent damage law, consistently with Continuum Damage Mechanics (CDM) theory. In particular, a one dimensional time dependent formulation based on a Kachanov's law is utilized to describe the dynamic damage variable by the following expression:

$$\xi_d(\mathrm{t}) = \left[1 - \frac{t - t_0}{t_f}\right]^{\frac{1}{m+1}} \tag{4}$$

where m is asymptotic parameter of the damage which controls the evolution of the damage function which is linear for m 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 m in the damage definition is typically assumed close equal to 0.98 [21].

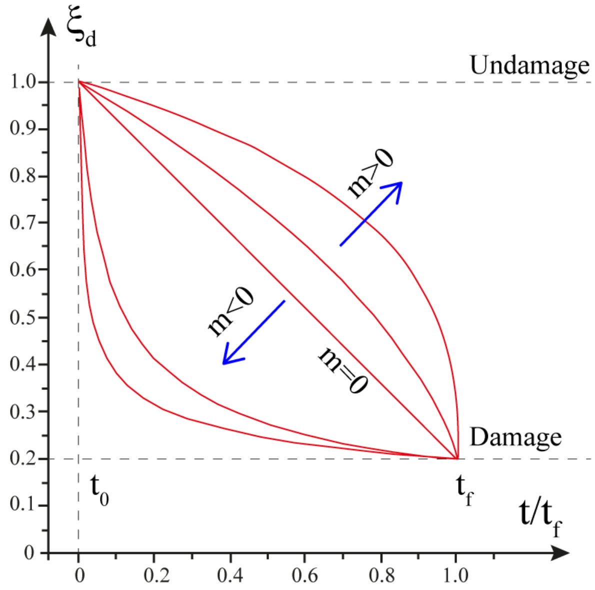


Figure 4. Damage law for cable element

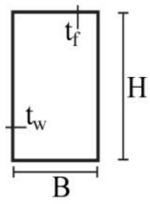
4 RESULTS

In the present paper, the structural behavior of network arch bridges subjected to the cable loss accidental event is investigated. The analyses focused on the evaluation of the effects produced by the cable loss under the action of dead loads. An advanced FE numerical model has been developed and a proper damage law has been formulated in order to reproduce the sudden loss of cables. Comparisons are proposed between nonlinear dynamic analyses and simplified methodologies reported in the main code on cable supported bridges (par. 2). A parametric study in terms of structural parameters is performed in order to identify geometric and mechanics properties that may increase the degree of robustness and safety of the structure.

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. Both the arch and the girder consist of a hollow rectangular cross-section whose dimensions are reported in Table 1. The cable system is composed of 34 hanger cables, which are organized in two specular sub-systems. Each sub-system is composed by 17 elements inclined of 65°, which are equally distributed along the girder with a hanger step of 10 m ($2 \cdot p$). However, the girder anchorages of the two sub-system do not coincide, but they are spaced of 5 m (*p*). This ensures that girder intermediate supports are equally spaced with a length of 5 m (*p*).

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

	B (m)	H (m)	t_w (m)	t_f (m)	A (m^2)	I (m^4)	W (m^3)
Arch rib	0.675	1.85	0.06	0.06	0.2886	0.1166	0.0315
Tie beam	1.56	2.00	0.013	0.013	0.0918	0.0567	0.0142



The cross-section area of the *i*-th hanger (A^H) is designed in such a way that under dead loads of the girder constant stresses are produced over all distributed elements. Such design stress, assumed to a fixed design value, namely S_g^C [22-24], is defined on the basis of the ratio between live (g^L) and dead loads (g^G) and allowable cable stress S_A^C by mean of the following relationship:

$$S_g^C = \frac{g^G}{g^G + g^L} S_A^C \tag{5}$$

The allowable cable stress (S_A^C), for fatigue requirements, is defined as the ratio between the ultimate cable stress (S_U^C) and a security factor of 2. The ultimate cable stress has been assumed equal to 1600 MPa, whereas a typical value of the live to dead load ratio for network arch bridges equal to 0.8 (g^L/g^G) is considered [25]. Therefore, the cross-section of the cable elements cab be expressed by means of the following expression:

$$A_h = \frac{g^G (p/2)}{S_g \sin(\alpha)} \tag{6}$$

The dead loads consist of girder self-weight, with a 0.30 m concrete slab thickness, whereas characteristics of ballast sub-ballast and utilities are reported in Tab. 2.

Table 2. Dead loads

Girder self-weight	7073	N/m
Sub-ballast	12250	N/m
Ballast	46000	N/m
Utilities	5000	N/m

4.1 Investigation of the effect produced by the cable loss on stress and kinematic variables

At first, the behavior of the bridge is investigated with reference to the loss of one hanger located at *x*=3/4 *L*. Fig. 5 shows the vertical displacements of the girder produced by the cable loss event. The graph reports the envelope of girder vertical displacements and the girder profiles obtained by means of the nonlinear dynamic analysis and the simplified methodologies proposed by PTI and EC3, respectively. Results show that the cable loss changes the initial configuration of the girder. The girder suddenly loses the undeformed profile producing vertical displacements whose amplitude varies along the girder profile. The portion of the girder from the midspan to the right end of the bridge is affected by the major vertical displacements. In particular, the maximum vertical displacement is observed close to the anchorage of the broken cable and it is almost 0.02% *L*. On the other hand, in the remaining part of the girder, i.e. from the left end to the midspan, the vertical displacements are quite reduced. The maximum vertical displacement of this portion is almost two times smaller than that observed one close to the anchorage of the broken cable. These results denote that the cable loss produces a local loss of stiffness in the cable system. Moreover, close to the damage zone, the structure loses the reticular effect provided by the cable system configuration and undergoes relevant deformations. The simplified approaches proposed by PTI and EC3 ensure an adequate degree of accuracy since results are quite similar to that obtained by means of the nonlinear dynamic analysis.

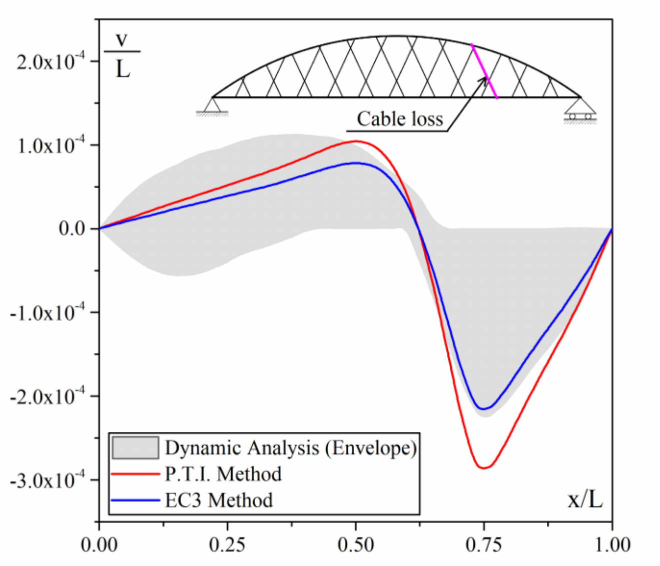


Figure 5. Girder displacements due to the loss of one cable located at x=3/4 L

However, the PTI method guarantees the highest degree of safety. As a matter of fact, the maximum vertical displacement predicted by PTI method is larger than that obtained by means of the dynamic analysis and EC3 method with errors equal to 20% and 24%, respectively. This highlights that, the simplified methodologies may ensure an adequate degree of approximation in the evaluation of displacements produced by the cable loss event.

Fig. 6 a-b illustrate bending moment distribution in the arch (Fig. 6-a) and the girder (Fig. 6-b), respectively. In particular, bending moment in the initial configuration of the bridge and results obtained by means of cable loss analyses, i.e. dynamic analysis and simplified methodologies, are reported. Results show that the cable loss causes an amplification of the bending moment in both the arch and the girder close to the anchorages of the broken cable. In these zones, results obtained by means of a dynamic analysis denote that both girder and arch are affected by a bending moment much larger than the one observed in the initial configuration. However, the girder is more vulnerable than the arch since the analysis bending moment is 42% of the yield bending moment. On the other hand, in the remaining part of the structure, the variation of the bending moment caused by the cable loss is practically negligible. The simplified methodologies proposed by PTI and EC3 ensure an adequate degree of approximation of the bending moment distribution in both the girder and the arch. For the arch, previsions provided by the PTI and EC3 methods differ from dynamic analysis results of 13% and -4.31%, respectively. For the girder, these differences are of 30% and 12% for PTI and EC3, respectively. This results denote that the PTI method provides conservative results, whereas EC3 method guarantees an acceptable approximation results.

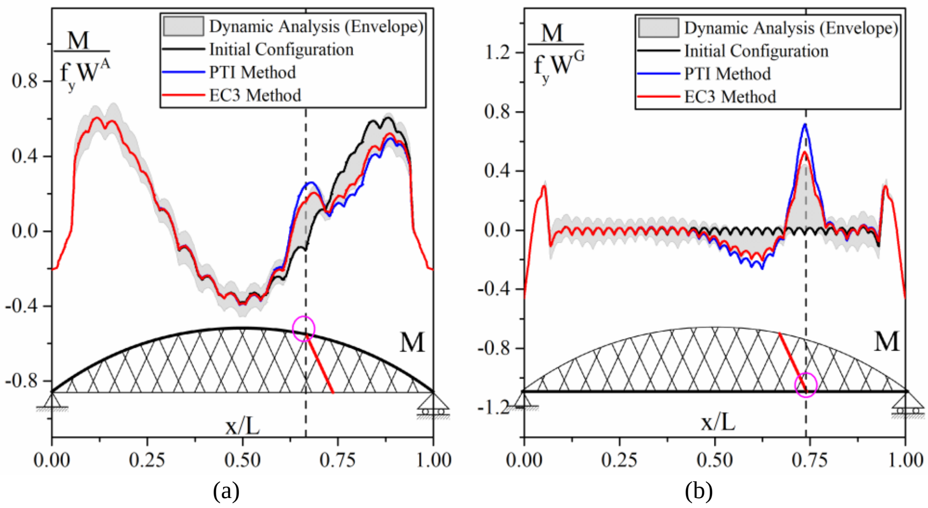


Figure 6. Bending moment distribution in the arch (a) and in the girder (b) due to the loss of one cable located at x=3/4 L

Fig. 7 illustrates the maximum stress in the hangers produced by the cable loss event. Results are organized as a function of hangers orientation. In particular, Fig. 7-a and Fig. 7-b show results of the hangers with right (R) and left (L) orientation, respectively. The results are expressed in a dimensionless form as the analysis stress (S) to the ultimate cable stress ratio (S_U^C). Comparisons results between cable loss analyses, i.e. the dynamic analysis and the simplified methodologies, and the stress distribution under dead load are reported.

Results show that the cable loss event produces a relevant amplification of stresses in the hanger elements located at x=3/4 L, i.e. near the broken one.

In particular, the stress in the hangers with left orientation increases about 63%, whereas the amplification observed for the hangers with right orientation is about 30%. This result highlights a decoupled behavior between the two sub-systems of cables in the case of cable loss event. This because the contribution of two sub-system of cables to the overall stiffness of the cable system varies along the girder development. At x=3/4 L, the hangers with right orientation are almost perpendicular to the arch profile and act in radial direction. Then, the primary function of these elements is to transfer the load from the girder to the arch. On the other hand, the hangers with left orientations works as stiffener members in order to limit the girder deformations. The loss of the stiffener member involves the major amplifications in terms of stress and kinematic variables. The simplified methodologies, i.e. PTI and EC3, ensure an adequate approximation only for cable element with left orientation. For hangers with right orientation, results show that the maximum cable stress predicted by means of the dynamic analysis is greater than that predicted by PTI and EC3 of 14.5% and 13%, respectively.

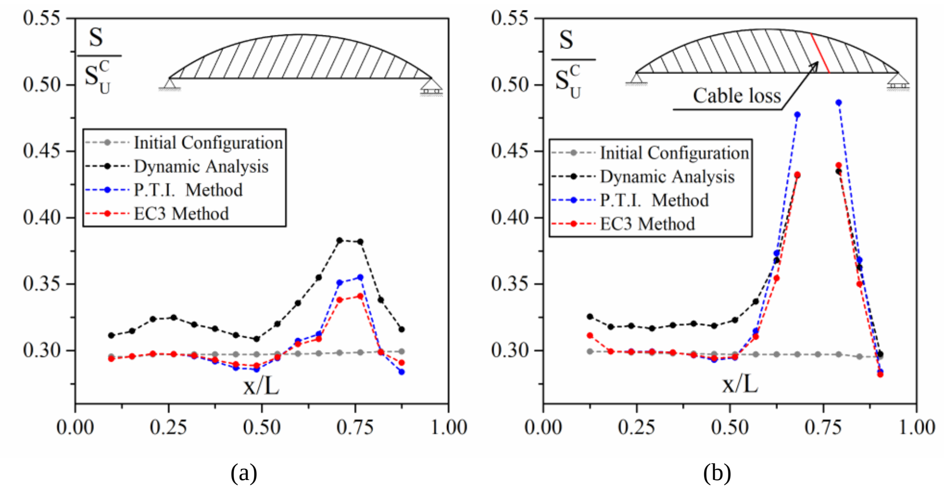
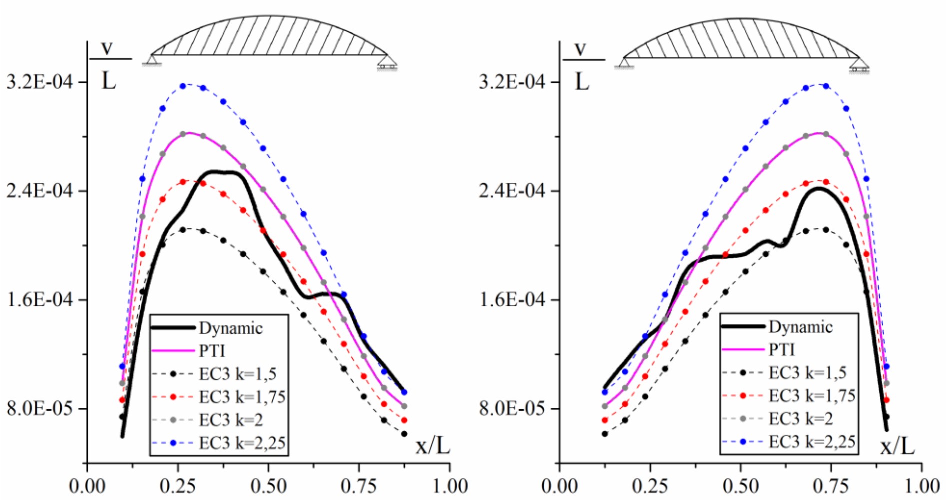


Figure 7. Stress distribution in the cable system produced by the loss of one cable at x=3/4 L

4.2 Parametric study

A parametric study is developed in terms of the main characteristics of the bridge structure to investigate the influence of the structural components on the amplification effects produced by cable loss events. The main aim of this study is to identify the key factors that may contribute to increase the degree of robustness and safety of the structure against the cable failure. The parametric study is developed with reference to the network arch bridge scheme analyzed in the previous section.

At first, the variability of the cable loss event is investigated. The main aim is to identify the worst cable loss event that may affect the bridge structure and, consequently, the most vulnerable hanger elements. Fig. 8 a-b shows the envelope of the maximum vertical displacements of the girder obtained by the cable loss analysis of each hanger that compose the cable system. In particular, Fig. 8-a illustrates results related to the hangers with right orientation, whereas Fig. 8-b refers to the hangers with left orientation. Note that, the horizontal axis of the graphs represents the position of the broken cable, whereas in the vertical axis the maximum vertical displacement achieved from the analyses is reported. Moreover, comparisons between dynamic analysis and the simplified methodologies, i.e. PTI and EC3, also are reported. Results show that the lateral zones of the bridge, i.e. close to x=1/4 L and x= 3/4 L, are the most vulnerable ones. With reference to dynamic analysis results, the maximum vertical displacements in the lateral zones of the bridge are larger than that observed at the midspan of 28%.



(a) (b)

Figure 8. Identification of the most dangerous cable loss event for the bridge structure in terms of girder vertical displacements

The comparisons between dynamic analysis and the simplified methodologies proposed by codes show that the PTI approach ensures a certain degree of safety for the most of the cable loss events. The maximum vertical displacement achieved by the PTI approach is bigger than that obtained by means of dynamic analysis of 10% and 14% at x=1/4 L and x=3/4 L, respectively. On the other hand, EC3 approach leads to unsafe results for the most of analyzed cable loss events. In order to ensure an adequate level of safety, the variability of the k-parameter defined by EC3 is investigated. Results highlights that a k-parameter of 2 may guarantee an adequate degree of approximation of the effects produced by the cable loss event. Moreover, EC3 results with k=2 coincide with the previsions provided by the PTI method.

Fig. 9 shows the variability of the maximum vertical displacement of the girder due to the loss of one cable at x= 3/4 L as a function of the hangers inclination (α). Results highlight that the variability of the vertical displacement of the girder as a function of the hangers orientation is almost linear for a large range of angle values. In particular, the vertical displacement increases as α decreases. In this context, the maximum vertical displacement of the girder for α=40° is almost 2 times bigger than that observed for α=75°. The variability of the structural response of the bridge as a function of the hanger inclination can be explained in view of the Dischinger's effect. In particular, the stiffness of a cable element is strictly related to the cable inclination and it increases as far as the cable element tends to the vertical configuration. Results also illustrate that the approximate methodologies provided by PTI and EC3 ensure an adequate level of approximations with the variability of α.

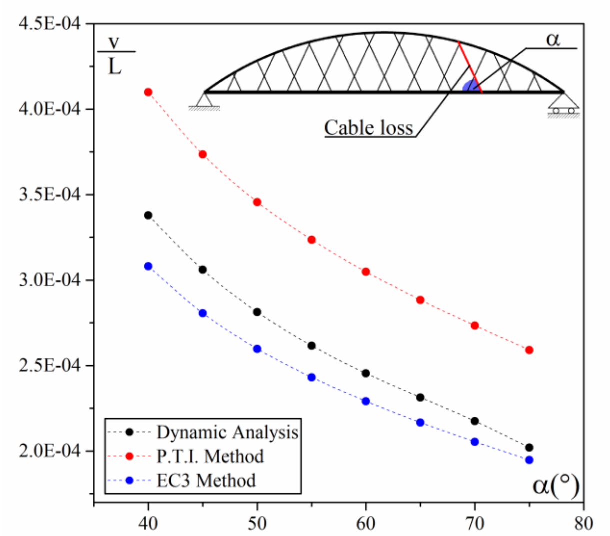


Figure 9. Influence of hangers inclination on the maximum vertical displacement of the girder

However, the PTI approach ensures the major degree of safety in the prediction of the structural response. As a matter of fact, the vertical displacements are always bigger than the ones observed in the frameworks of a dynamic analysis and EC3 method with errors equal to 18% and 51%, respectively.

Fig. 10 presents the analysis of the variability of the bending moment of the girder as a function of the number of cables (*m*).

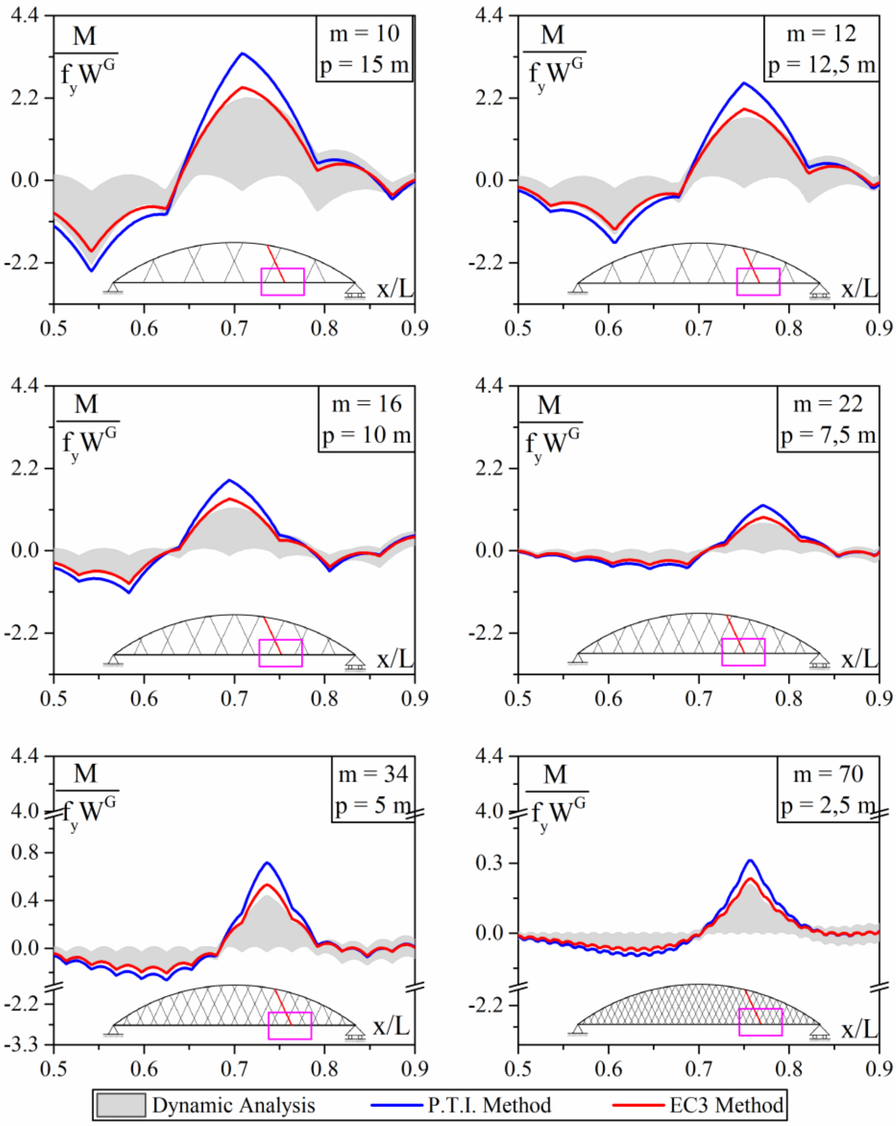


Figure 10. Influence of the number of hangers (*m*)

The loss of one cable located at x=3/4 L with right orientation has been investigated for six cable system configurations with a different number of hangers and a different hanger step (p). Table 3 summarizes the properties of the examined bridge configurations in terms of number elements (m) and hanger step along the girder profile (p).

Table 3. Geometric data of the arch bridge configurations

Scheme	**1**	**2**	**3**	**4**	**5**	**6**
m	10	12	16	22	34	70
p (m)	15	12.5	10	7.5	5	2.5

Each figure identifies the portion of the bending moment diagram close to x=3/4L, which is the location where it is reached the maximum values of the bending moment. Results are expressed in a dimensionless form, as the ratio between the bending moment obtained by means of the cable loss analyses and the yield bending moment of the girder ($M_y^G = f_y \cdot W^G$).

Results shows that the number of hangers highly influence the response of the structure affected by cable failure. The bending moment increases as far as the number of cables is reduced. For the first four cable configurations, i.e. from m=10 to m=22, the cable loss generates unsafe situations since the bending moment is larger than the yield one. The most dangerous situation is the case of m=10, in which results arising from dynamic analysis highlights that the cable loss generates a bending moment 2 times larger than the yield one. On the other hand, for m=34 and m=70, the amplification in terms of bending moment is about 40% and 25% of the yield value, respectively. Note that, the cable systems with m=10 and m=70 represent extreme configurations, which may be impracticable to use. Contrarily, the remaining cases, i.e. from m=12 to m=34, are the most common configurations for the considered span length.

This suggests that an adequate degree of safety for the structure may be obtained by means of cable systems composed by a large number of hangers equally spaced along the girder with a step smaller than 10 m. Comparisons between dynamic analysis and the simplified methodologies results reveal that for all examined cases the PTI and EC3 approaches ensure an adequate degree of approximation of the prediction of bending moment.

Fig.11 shows the analysis of the structural response of the bridge in terms of number of cables involved in the cable loss event. Three cable loss events at x=3/4 L are compared in terms of the maximum vertical displacement of the girder: (D1) the cable loss of just one cable; (D2) the cable loss of 2 cables; (D3) the cable loss of 3 cables. Results show that the maximum vertical displacement of the girder increases as far as the number of cable involved in the damage scenario increases. The vertical displacement for D3 is almost six times bigger than that obtained for D1.

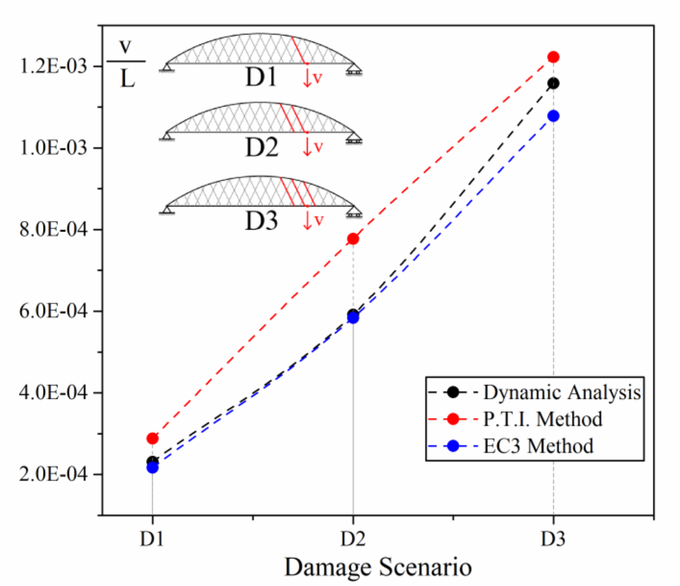


Figure 11. Vertical displacement of the girder produced by different cable loss scenarios

Moreover, results denote that the simplified methodologies provided by EC3 offer a good approximation of the effects also for the analysis of multiple cable loss event. This statement is also valid for PTI, although it provides quite conservative predictions. Note that, the PTI method has been developed to analyze the effects produced by the cable loss of just one cable in the case of cable stayed bridges. The obtained results have revealed that this approach can be applied in multiple cable loss scenarios in the case of network arch bridges.

Finally, a comparison in terms of arch bridge typologies is proposed in Fig. 12. The loss of one cable located at x=3/4 L is investigated with reference to three arch bridge schemes: (*i*) a tied-arch bridge; (*ii*) a network arch bridge whose hangers are inclined of 70°; (*iii*) a network arch bridge with orientation angle of hangers equal to 45°. For each arch bridge scheme, the volume of the cable system is reported in the figure. For the sake of brevity, only dynamic analysis results are proposed. However, also in this case, results obtained by means of the simplified methodologies, i.e. PTI and EC3, provide an adequate degree of accuracy in the evaluation of the effects produced by the cable loss event. Results shows that the tied arch bridge guarantees the highest degree of safety and robustness. For the tied arch bridge, the vertical displacement produced by the cable loss is 3 and 2 times smaller than that obtained for the network arch bridge with the hanger inclined of 45° and 70°, respectively. This result may be explained by the fact that the tied arch scheme offers better performances than network one under the action of dead loads. As a matter of fact, the cable system of the tied arch scheme transfers the dead loads to the arch rib as an evenly distributed vertical load. This load scheme produces large axial forces and low bending moments in the structural members of the bridge. Then, the structure is characterized by a considerable degree of stiffness, which contributes to reduce the effects produced by potential cable damages. However, the better performance of tied arch scheme with respect to the network ones may be not verified in the case of simultaneous action of moving load and sudden loss of cables. As a matter of fact, the tied arch bridges are affected by considerable deformations under the actions of unsymmetrical load schemes. In these situations, the network arch schemes may offer an adequate degree of robustness and safety much larger than tied arch bridges.

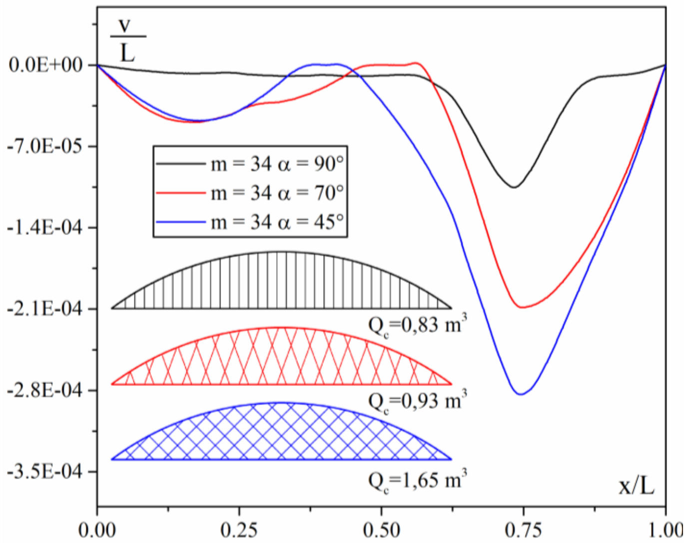


Figure 12. Comparisons between tied and network cable system configurations

5 CONCLUSIONS

In the present study an investigation on the behavior of network arch bridges is developed. In particular, the main aim of the paper is to verify the vulnerability behavior of network arch bridges in presence of accidental failure mechanisms arising from cable loss events. Such task is achieved by means of a proper definition of the structural model based on a FE formulation and an accurate description of the damage mechanisms on the basis of CDM theory. Parametric results are proposed to quantify the amplification effects on both kinematic and stress quantities for several bridge configurations, in terms of the main geometrical and mechanical bridge characteristics. Comparisons with respect to conventional configurations based on vertical cable system show the enhanced behavior of the network schemes. Moreover, the results show how conventional approaches provided by existing codes, i.e. PTI and EC3, on cable supported bridges may be utilized to predict the vulnerability behavior of network arch bridges affected by cable loss events.

REFERENCES

[1] Bruno, D., Lonetti, P., and Pascuzzo, A., "An optimization model for the design of network arch bridges", Computers and Structures, Vol. 170, pp. 13-25, 2016.

[2] Lonetti, P., Pascuzzo, A., and Davanzo, A., "Dynamic Behavior of Tied-Arch Bridges under the Action of Moving Loads", Mathematical Problems in Engineering, Vol. 2016, pp. 17, 2016.

[3] European Committee for Standardisation, Eurocode 3: Design of Steel Structures, in Part 1-11: Design of steel structures, Design of structures with tension components. 2006

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

[5] Lepidi, M., Gattulli, V., and Vestroni, F., "Static and dynamic response of elastic suspended cables with damage", International Journal of Solids and Structures, Vol. 44(25), pp. 8194-8212, 2007.

[6] Mapelli, C. and Barella, S., "Failure analysis of a cableway rope", Engineering Failure Analysis, Vol. 16(5), pp. 1666-1673, 2009.

[7] Mahmoud, K.M., "Fracture strength for a high strength steel bridge cable wire with a surface crack", Theoretical and Applied Fracture Mechanics, Vol. 48(2), pp. 152-160, 2007.

[8] Li, C.X., Tang, X.S., and Xiang, G.B., "Fatigue crack growth of cable steel wires in a suspension bridge: Multiscaling and mesoscopic fracture mechanics", Theoretical and Applied Fracture Mechanics, Vol. 53(2), pp. 113-126, 2010.

[9] Materazzi, A.L. and Ubertini, F., "Eigenproperties of suspension bridges with damage", Journal of Sound and Vibration, Vol. 330(26), pp. 6420-6434, 2011.

[10] Wolff, M. and Starossek, U. *Robustness assessment of a cable-stayed bridge*. in *Proceedings, 4th International Conference on Bridge Maintenance, Safety, and Management (IABMAS'08), Seoul, Korea*. 2008.

[11] Starossek, U. *Collapse resistance and robustness of bridges*. in *The 4th International Conference on Bridge Maintenance, Safety, Management.* 2008.

[12] Mozos, C.M. and Aparicio, A.C., "Parametric study on the dynamic response of cable stayed bridges to the sudden failure of a stay, Part I: Bending moment acting on the deck", Engineering Structures, Vol. 32(10), pp. 3288-3300, 2010.

[13] Mozos, C.M. and Aparicio, A.C., "Parametric study on the dynamic response of cable stayed bridges to the sudden failure of a stay, Part II: Bending moment acting on the pylons and stress on the stays", Engineering Structures, Vol. 32(10), pp. 3301-3312, 2010.

[14] Jani, H. and Amin, J., "Analysis of cable-stayed bridge under cable loss", International Journal of Bridge Engineering, Vol. 5(1), pp. 61-78, 2017.

[15] Greco, F., Lonetti, P., and Pascuzzo, A., "Dynamic analysis of cable-stayed bridges affected by accidental failure mechanisms under moving loads", Mathematical Problems in Engineering, Vol. 2013, 2013.

[16] Zhou, Y. and Chen, S., "Numerical investigation of cable breakage events on long-span cable-stayed bridges under stochastic traffic and wind", Engineering Structures, Vol. 105, pp. 299-315, 2015.

[17] Qiu, W., Jiang, M., and Huang, C., "Parametric study on responses of a self-anchored suspension bridge to sudden breakage of a hanger", Scientific World Journal, Vol. 2014, 2014.

[18] Lonetti, P. and Pascuzzo, A., "Vulnerability and failure analysis of hybrid cable-stayed suspension bridges subjected to damage mechanisms", Engineering Failure Analysis, Vol. 45, pp. 470-495, 2014.

[19] Wang, P.H., Tseng, T.C., and Yang, C.G., "Initial shape of cable-stayed bridges", Computers and Structures, Vol. 46(6), pp. 1095-1106, 1993.
[20] Wang, P.H., Tang, T.Y., and Zheng, H.N., "Analysis of cable-stayed bridges during construction by cantilever methods", Computers and Structures, Vol. 82(4-5), pp. 329-346, 2004.
[21] Petrini, F. and Bontempi, F., "Estimation of fatigue life for long span suspension bridge hangers under wind action and train transit", Structure and Infrastructure Engineering, Vol. 7(7-8), pp. 491-507, 2011.
[22] Lonetti, P. and Pascuzzo, A., "A numerical study on the structural integrity of self-anchored cable-stayed suspension bridges", Frattura ed Integrita Strutturale, Vol. 10(38), pp. 359-376, 2016.
[23] Lonetti, P. and Pascuzzo, A., "Design analysis of the optimum configuration of self-anchored cable-stayed suspension bridges", Structural Engineering and Mechanics, Vol. 51(5), pp. 847-866, 2014.
[24] Lonetti, P. and Pascuzzo, A., "Optimum design analysis of hybrid cable-stayed suspension bridges", Advances in Engineering Software, Vol. 73, pp. 53-66, 2014.
[25] Hedgren, A.W. and P.E., J., Arch Bridges, in Structural steel designer's handbook, I. McGRAW-HILL, Editor. 1994: New York.