Damage evaluation of cable-stayed bridges subjected to blast loading

Cyrille Denis TETOUGUENI [1], Paolo ZAMPIERI [1], Carlo PELLEGRINO [1]
[1] University of Padua, Padua, Italy

Contact e-mail: cyrille.tetougueni@dicea.unipd.it

ABSTRACT: One of the major issues for a structure is to control the collapse mechanism. In particular, it is important to understand the consequences of actions to the structure. The safety requirement can easily be checked for well-known exerting actions on the structure, but there are, on the other hand, hazards which are often difficult to be predicted and then increase the complexity in the design. The aim of this study is to evaluate the damage induced by an accidental load such as blast loading to a cable-stayed bridge. In the first part of the paper, the non-linear pushover analysis of the structure has been performed to evaluate the limit state of the structure for a random position of blast loading, then through non-linear dynamic analysis, it was possible to collect the stress and strain states of the structure in the action time history and finally, the frequency of observed damage was presented in term of loading parameters. The major finding shows that the cable-stayed bridge presents a different structural behavior in relation to the position of load. In addition, through the analysis, it is possible to predict the possible damage state during hazards such as blast loading.

1 INTODUCTION

Several existing structures nowadays are facing the risk of destruction under explosions since primarily most of them were designed to satisfy the limit state under current use while extreme events have not been considered in the design. Jenkins (Jenkins, 1997) has identified in his work no fewer than 550 attacks against existing buildings and infrastructures over the past 60 years. These figures are not about to stop there in view of the number of terrorist attacks and intimidation situations recently observed around the world. For these reasons, structures are increasingly exposed to early failure compared to their initial expected lifetime because the observed degradation of a structure depends on several parameters related to both the structure and the intensity of the event. For this reason, many scientists, organizations in recent decades have addressed the issue in order to study or provide new insights into existing work on the structural performance of structures subjected to blast loads.

The blast loading and its impact on building studied extensively (Ngo, *et al.*, 2007; Subramaniam, *et al.*, 2009). Dass and Matsagar (2014) presented an overview of how the blast loading is induced after an explosion. They derived a framework to study the structural response of buildings under the shock wave released by the explosion. Throughout a probabilistic approach, Olmati et al. (2014) were able to present the structural performance of cladding wall panels subjected to blast loading. The study of a structure under blast loadings is more complex since it includes several parameters related to the structure's geometry, the material used and the description of the blast load model. Elsanadedy et al., (2014) assessed through numerical analysis the expected progressive collapse of a typical steel building in the situation of blast attacks. Pan et al. (2017), studied the damage induced by a high-intensity blast loading on Highway Bridge. Other relevant issue associated to blast loading is that the load does not belong to other conventional loads such as live load, permanent loads etc… According to Eurocode 1991-1-7 (2002), explosions or blast loading induced by explosions may be considered as accidental loads. For this reason, it may be hard to check the basic safety requirement. The most common methodology under this situation is the perform reliability assessment of the structure exposed the hazard in order to be able to predict the damage state in function of the loading parameters.

From what emerges in the literature review, it is clear that many scientists nowadays pay more attention to both the implementation of charges due to blast loading and the structural consequences observed after such acts. In this study, the reliability assessment of a cable-stayed bridge has been perform through an extensive non-linear analysis. In the first stage, the limit state of the bridge has been defined from the non-linear pushover analysis, then the stress and strain states observed in the load time history are presented and finally, it was possible to highlight the frequency of observance each damage state. The main results show that the probability to reach each damage state strongly depends on the loading parameters.

Nomenclature

$G_{k,j}$	permanent loads
P	pretension force in cable
A_d	accidental loads
$Q_{k,1}$	leading variable loads
$Q_{k,1}$	accompanying variable loads
R	Stand-off
W	weight of
f_{yd}	dynamic yielding stress
f_y	yielding stress

2 NUMERICAL MODEL OF BRIDGE

2.1 Bridge description

The structural analysis of a cable-stayed bridge displaying Harp pattern has intensively been analysis through strand7 software (2005). The total span of the cable-stayed bridge is 403 m supported by four pylons with a height of 51 m, each pair resting on two 30 m square piers. The bridge consists in one central part with 204.6 m span and two lateral parts with 99.20 m span. The 13m wide deck is supported by 64 circular stay cables are equally distributed along the deck and both pylons to contribute in load transfer. Figure 1 shows the longitudinal view of the bridge under study. The cable anchorage is 12.4m equidistant each other on the deck and 6.2 m equidistant along to the pylon. The deck is made by an optimized closed box section; additional stiffeners are used to increase the buckling strength of the deck. Piers and Pylons are also made of square close box section. Two transversal beams are used to increase the out-of-plane stability of the pylons and piers. Details related structural elements used in this study are presented in Table 1.

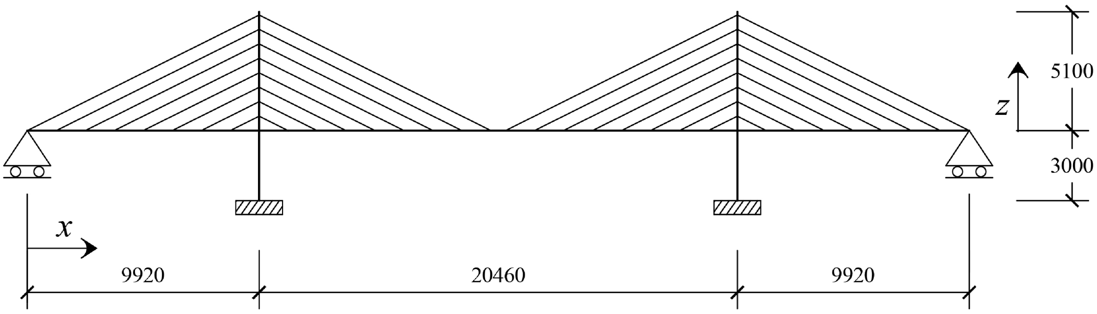


Figure 1. Longitudinal view of the bridge

Table 1. Geometric characteristics of structural sections

	Pylon Base	Pylon	Transversal beam	Deck	Unit
b	1800	2800	3200	13000	[mm]
h	2800	2800	3200	1150	[mm]
t	40	40	40	20	[mm]
A	441600	521600	585600	642331,3	[mm^2]
i_{xx}	4,88×10^{11}	6,41×10^{11}	9,47×10^{11}	1,59×10^{11}	[mm^4]
i_{yy}	2,37×10^{11}	6,41×10^{11}	9,47×10^{11}	9,37×10^{11}	[mm^4]

2.2 *Numerical modelling*

The actual numerical model has been made by 1D fiber beam element. Rigid connections are considered between structural elements. Cable stays are directly connected to pylons while rigid links are used to create the proper connection between the deck and cables. For simplicity, abutments were not modeled and were replaced by fixed constraints. Then elastic links with high vertical stiffness are used as bearing at both bridge end and above the transversal beam. High strength steel material was considered for deck, pylons, and piers whereas the stress distribution in the cables was limited to $0.55\times f_u$ to satisfy the fatigue criteria. In order to consider the cable's sag effect due to the change in the shape under varying stresses, Ernst's formula has been used to derive the equivalent elastic modulus. Perfectly elastoplastic stress-strain diagrams for different elements excepted cables are used in the analysis. Figure 2 shows the 3D numerical model of the bridge with the stress-strain relationship of structural sections. The observed structural response under high impact load is always nonlinear. In this study, both material and geometrical non-linear behaviors of structural elements are considered.

3 FEM ANALYSIS

3.1 *Load analysis*

In this paper, blast loads are considered as the principal live or variable load whereas traffic loads are taken as secondary loads. The self-weight is directly provided and is a function of the material density and the characteristics of the section. A uniformly distributed load G_2=48.7 kN/m is applied along the deck which represents asphalt layer, safety barrier, and waterproofing layer. The pretension force on stays is calculated throughout an optimization process in order to compensate 95% of the permanent loads. Traffic loads comprise a uniformly distributed load and tandem system are defined according to EN 1991-2 (2004).

3.2 Blast loading model

Two waves are generated after blast loading (Son, *et al.*, 2005). The incipient and the reflected pressure wave. In this study, we will focus only on the reflected pressure since it is considered as the one through which very high pressure is released. The reflected pressure is generated when the pressure wave encounters the solid surface of the exposed object such as bridge deck, building façade,etc...

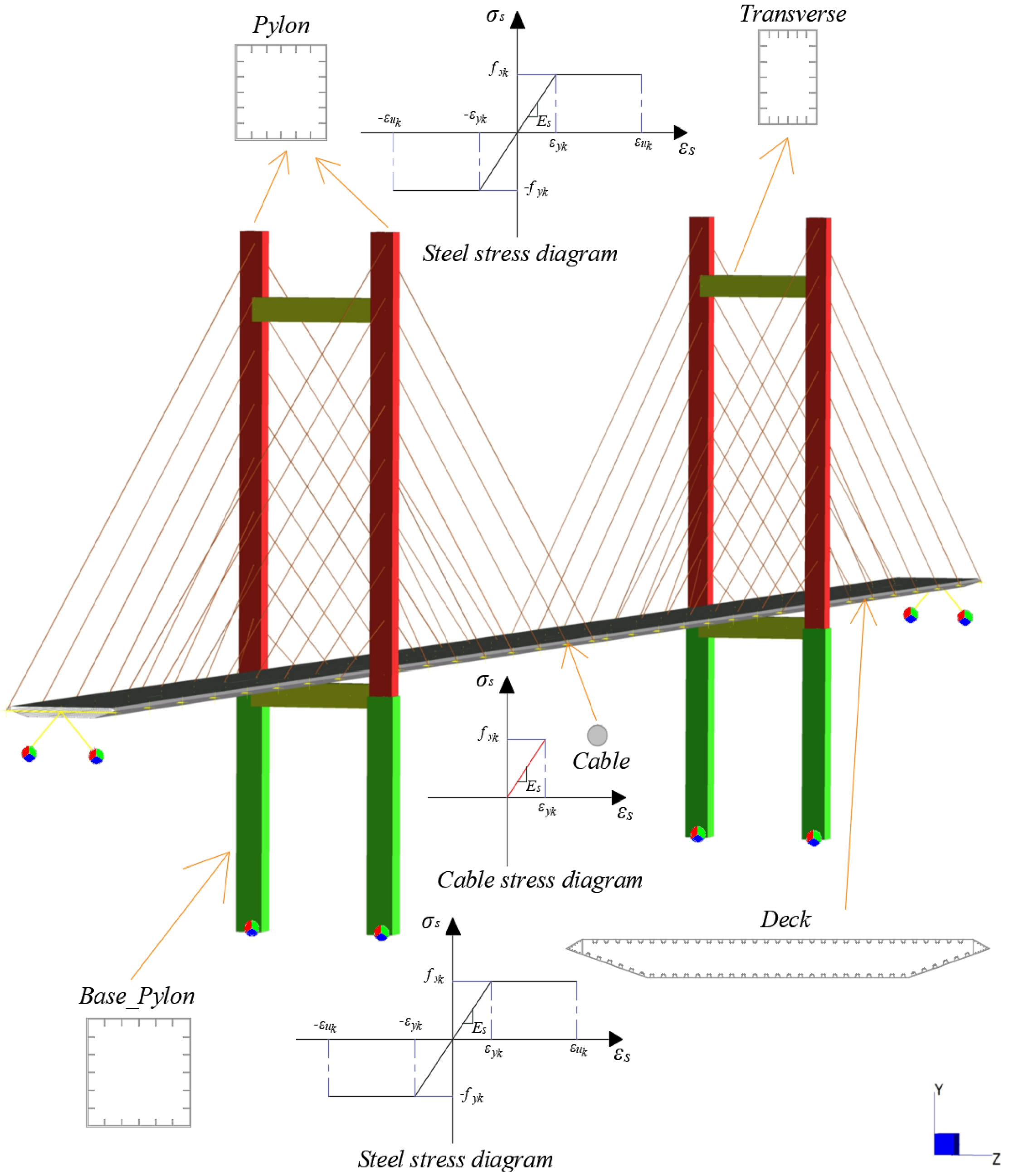


Figure 2. 3D Numerical model of the cable-stayed bridge under study

Table 1. Parameters under consideration for blast analysis

R (m)	1	1.5	2			
W (kg)	100	250	500	750	1000	1500
Location	Middle span	Close to abutment				

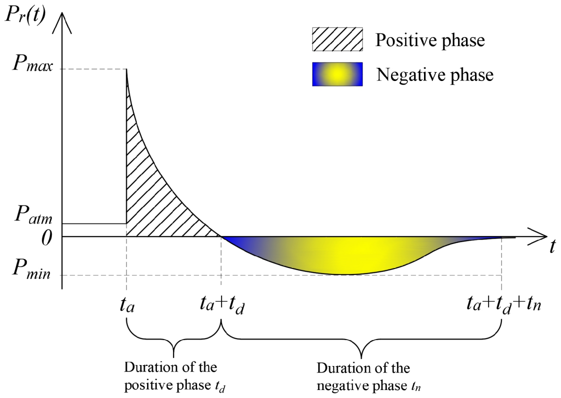


Figure 3. Characteristic of reflected pressure during a blast loading

The impact of blast loading to a structure is characterized by three periods as described in Figure 3. Usually, the pressure induced by blast loading depends on ambient pressure, the equivalent mass of the TNT representing the magnitude of the explosion and the stand-off distance. Several authors derived empirical formulas, to find the pressure distribution for a given bomb's mass and stand-off coefficient (Olmati, *et al.,* 2014). In this paper, the software RC Blast (2014) will be used to find out the pressure distribution for a given parameter of blast loading. finally, since the software used in the structural assessment doesn't integrate the blast load model, the peak overpressure is collected from RC Blast and then converted into a point load to study the local effect induced the blast.
Different stages were built in the analysis to study separately the effect blast loading under a bridge in service. Indeed, the first stage consisted of determining the actual stress of the bridge in the traffic condition. Then this state is solved and considered as the initial stage during blast analysis. To reach our goal, load combinations (Eq. 1) for accidental design situations as defined in EN 1990 (2002).

$$E_d = E\left\{G_{k,j};\, P\,;\, A_d;\, \left(\psi_{1,1} \text{ or } \psi_{2,1}\right)Q_{k,1};\, \psi_{2,i}Q_{k,j}\right\}\ j\geq 1\ ;\ i>1 \tag{1}$$

3.3 Material Properties

For steel and concrete materials, the behavior of these structural materials differs in correspondence to the strain rate. Typically, hazards such as blast loading, earthquake involve high strain rate since they occur over a short period. Under very high strain rates, the modulus of elasticity changes whereas the ultimate strain remains almost the same. In this paper, only the performance of the deck is studied. Indeed, the dynamic yield stress of the deck is obtained throughout the Cowper-Symonds equation (Eq. (2)). In this equation, C and q are coefficients depending on the steel material. Table3 shows the adopted yield and ultimate stress for different structural sections.

Table 2. Material properties of structural sections

Structural Element	Density Kg/m^3	Modulus Of Elasticity MPa	Yield Stress MPa	Ultimate strength MPa
Deck	7850	210000	585	585
Pylon	7850	210000	420	420
Cable	7850	157692	///	1023

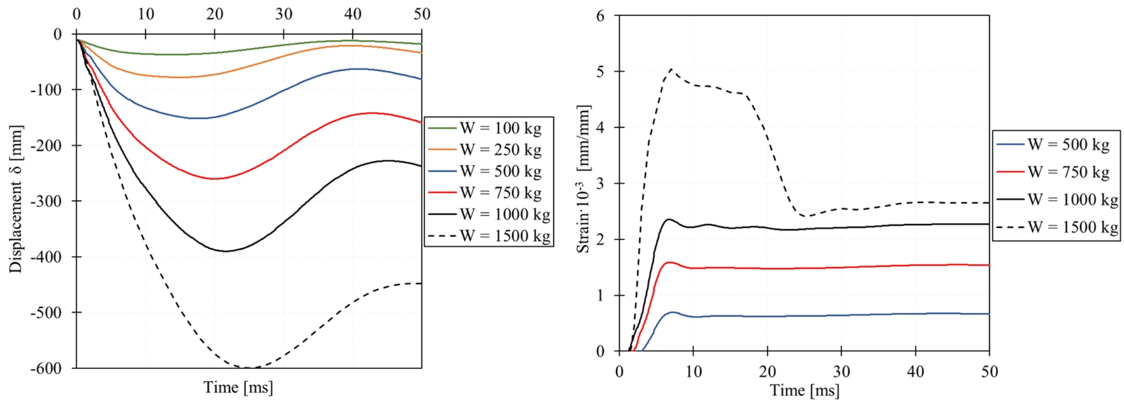
$$\frac{f_{yd}}{f_y}=1.0+\left(\frac{\dot{\varepsilon}}{C}\right)^{1/q} \tag{2}$$

4 FEM RESULTS AND COMMENTS

The structural response of the deck to blast loading is assessed throughout the dynamic analysis of the structure with loading condition following the pattern described in Figure 3 and considering parameters defined in Table2. The distribution of displacement, strain, and stress is captured after a short period following the blast loading.

4.1 Blast loading close to the abutment

For any position where the impact of the explosion takes place on the bridge deck, we observe different structural responses of the bridge depending on the weight of the bomb and the stand-off. The first observation is that the effect of the bomb is more damaging as the explosion takes place close to the deck. Indeed, stress, displacement, and strain increase with the reduction of the stand-off.
For R=2 m, close to the impact, the stress in the bridge's deck will reach the dynamic yielding for bomb weight from 500kg to 1500kg, but the stress will decrease in the few seconds after the impact (Figure 5). In this case, heavy damage is induced for 1000 – 1500 kg of bomb weight whereas for other bomb weight, superficial to moderate damage is observed.

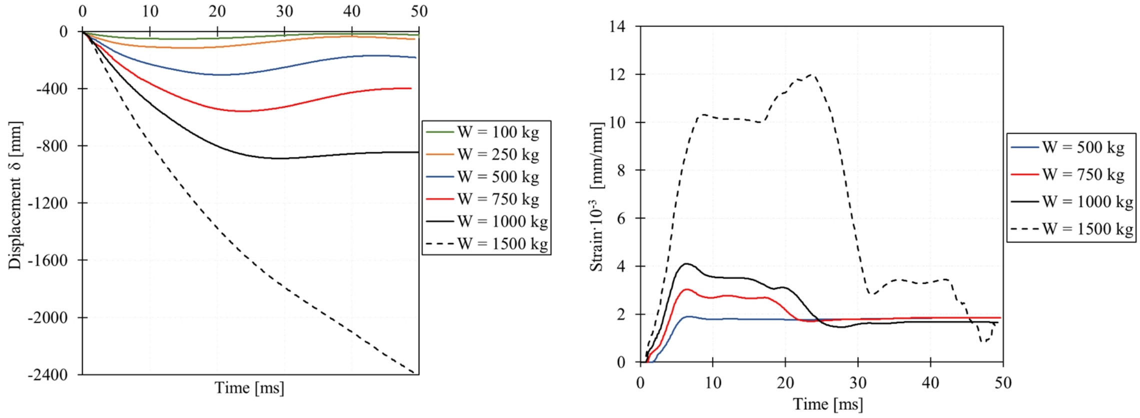


a) Displacement b) strain

Figure 5. Displacement, plastic strain and Stress time histories of the deck near the blasting point for stand-off R=2 m: Blast loading in the middle span

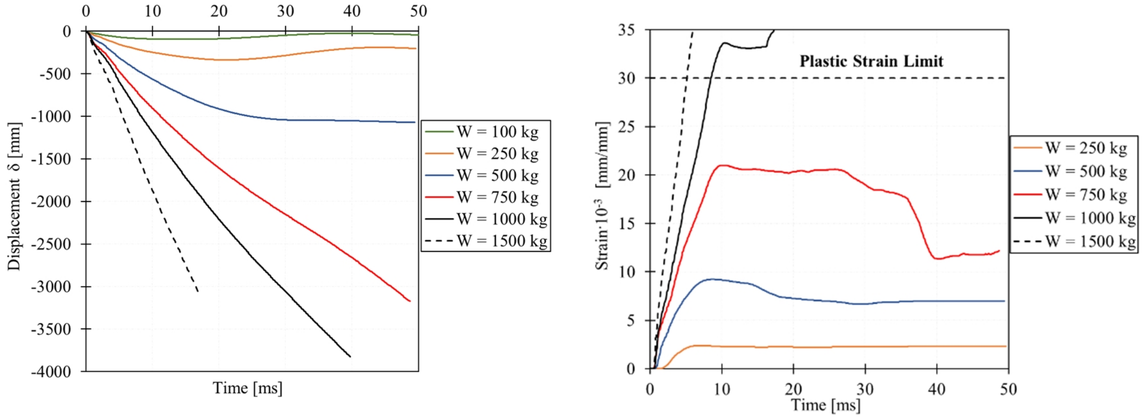
Similarly to the case where R=2 m, it is observed that small bomb weight is not able to induce severe damage to the deck for R= 1.5 m. For service purpose, the bridge will probably be closed a blast loading provoked by 1500 kg of TNT. The displacement observed is unacceptable for service purpose and even a few seconds after the blast occurs, the displacement is still increasing probably due to the fact that large plastic strain is developed within the section. In the meantime, in all the remaining cases, the bridge's deck is able to regain its original position or part of it (Figure 6). Hazard failure will be observed for probably for this TNT weight whereas the failure state for the other bomb weight is still minor to moderate. However, in these cases, the section at the impact point will not totally split up since the maximum plastic strain is not yet reached.
Finally, for the explosion, which takes place at 1m above the deck, the damage is much more severe for almost all the weight. The dynamic yielding stress is reached for all the cases (Figure 7). Blow out failure will be observed for blast loading induced by 1000-1500 kg of the TNT. A dramatic break down of the deck will be observed less than 10 ms after the impact since the maximum plastic strain will be reached in a short period. Although the 750 kg of TNT is likely to

produce also a dramatic failure, the deck, in this case, will not probably split up into two parts since it still exists a reserve before the complete breakage of the deck.



a) Displacement b) strain

Figure 6. Displacement, plastic strain time histories of the deck near the blasting point for stand-off R=1.5 m: Blast loading in the middle span



a) Displacement b) strain

Figure 7. Displacement, plastic strain and Stress time histories of the deck near the blasting point for stand-off R=1 m: Blast loading in the middle span

4.2 Limit state of cable-stayed bridge

Throughout an intensive material and geometric non-linear analysis, it was possible to determine the structural performance of the cable-stayed bridge. Indeed, from the analysis, the shear and bending failure modes were derived.

From Figure 8 and Figure 9, it is observed that the performance of the structure depends on the location of the load. In fact, the large ductility capacity in terms of shear when the load is applied close to the abutment. In addition, it is worth notating that the structure will fail with small displacement exhibited for load located close to the abutment while it can exhibit very large displacement before the rupture for the load at the midspan. This finding is quite opposed to the case of bending performance as shown in Figure 9. Then, it was possible to define three different damage states from moderate to collapse as presented in table 4 and table 5.

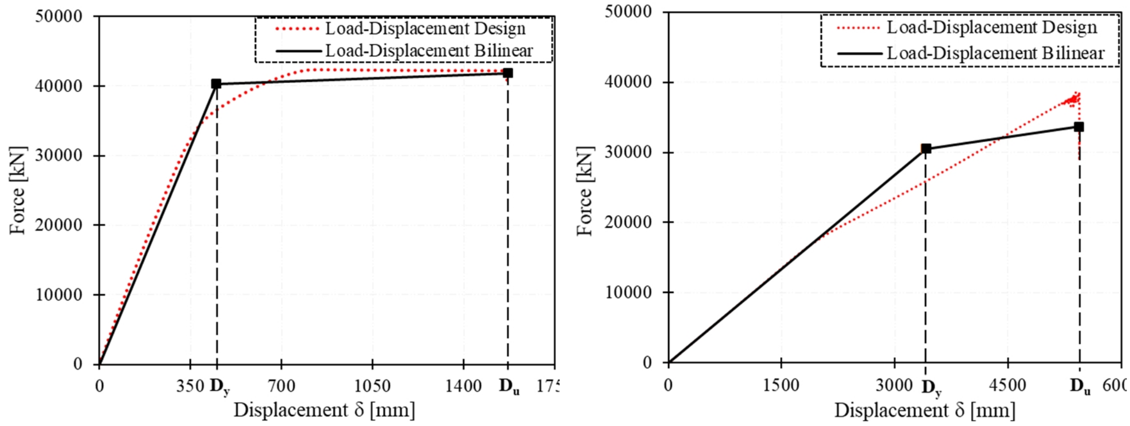


Figure 8. Shear performance of cable-stayed bridge

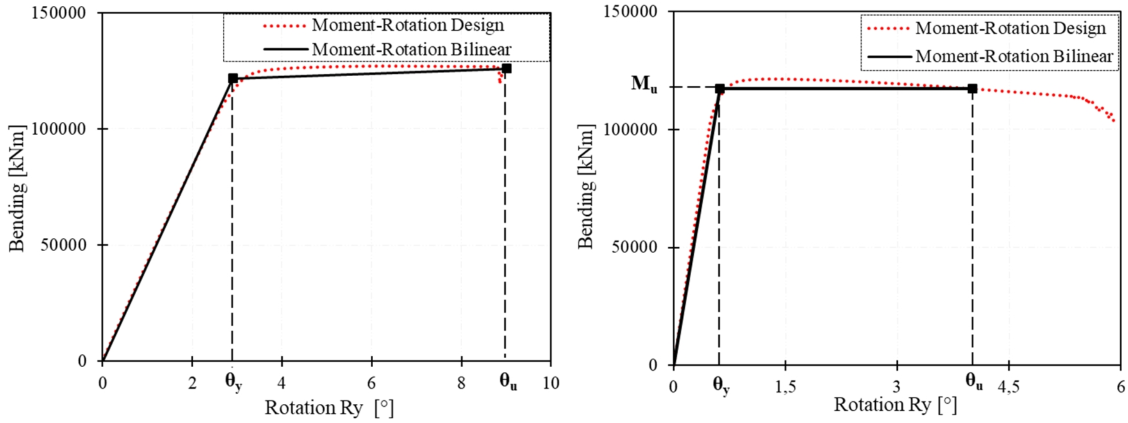


Figure 9. Bending performance of cable-stayed bridge

Table 4. Limit state of bridge for load at the abutment

Description	Physical phenomenon	Deck in service with repair	Need consequent repair	Deck out of service
component		Moderate	Major	Collapse
Deck	θ [°]	$2.9 < \theta \leq 5.6$	$5.6 < \theta \leq 9$	$\theta > 9$

Table 5. Limit state of cable-stayed bridge for load at the midpan

Description	Physical phenomenon	Deck in service with repair	Need consequent repair	Deck out of service
component		Moderate	Major	Collapse
Deck	θ [°]	$0.7 < \theta \leq 1.8$	$1.8 < \theta \leq 3.1$	$\theta > 3.1$

Finally, the nonlinear dynamic analysis has been performed to take into account the time history behaviour observed during a blast loading. Different loading magnitudes from small to very large were considered as presented in table 2. The results showed that as the blast detonation happens closed to the deck, the probability to observe the collapse of the deck increase whereas moderate damage state will be observed at the moment the blast detonation takes place far from the deck (Figure 10).

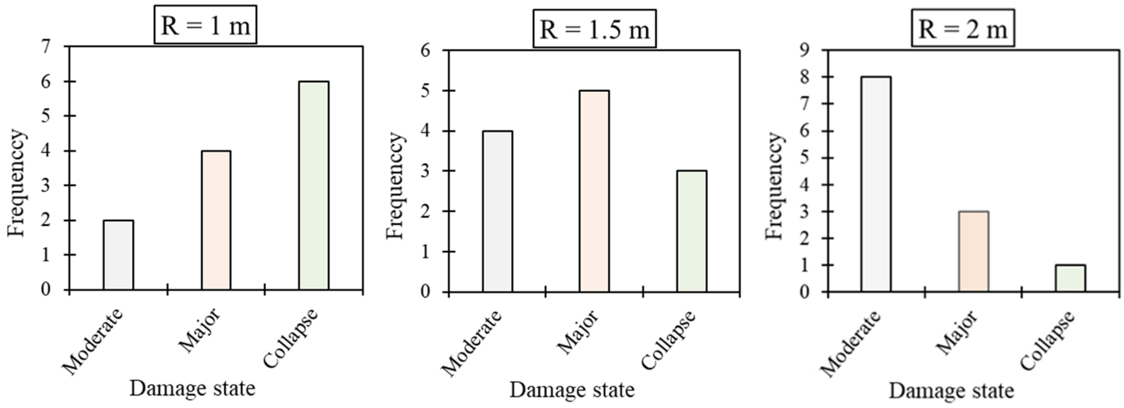


Figure 10. Damage state of cable-stayed bridge subjected to blast loading

5 CONCLUSION

The assessment of a generic cable-stayed bridge was performed throughout a thorough numerical analysis in order to study the structural performance of these bridge's typologies in the event of bomb attacks. Different bomb masses, as well as different stands-off were considered in this paper and the results from the analysis made it possible to produce the following conclusions.

The severity of the damage varies widely from one attack to another. As the explosion happens close to the deck, the observed damage becomes critical. The blast generated 2 m above the deck is unlikely to trigger the hazard failure of the deck.

In the case of blast loading analysis, the yielding stress is not the most important parameter to study since a high level of stress is released after an explosion. In almost all cases, the structure will certainly reach the yielding stress.

Permanent and traffic loads influence the structural performance of the cable-stayed subjected to blast loads. Major damages are observed when the blast happened at the middle span where an existing stress distribution is visible.

Steel material is likely to be appropriate for structure against blast loading. Indeed, the large plastic strain reservoir of the material prevents steel structure against moderate blast loading to the total failure.

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