Performance-based fire design of complex structures

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Abstract: The problem of structural fire safety in the recent years has gained a predominant position within the engineering design, with the affirmation of performance-based structural codes and standards, replacing more and more the traditional prescriptive ones. This is because nowadays, structures always bigger and more complex are designed and built. In modelling such complex structures, there are important aspects and relevant uncertainties that need to be taken into account. This paper focuses on the application of the performance-based fire design to this kind of structures; the systemic approach is identified as the proper tool to manage all the aspect related with the problem. A general framework is presented for this purpose and it is applied to a facility made of steel for the storage of helicopters, with a relatively complex geometry subject to fire. The structure is of interest since, due to its occupancy, it is prone to elevate fire risk. The modelling of the problem proposes the use of non-linear analysis that includes thermo-plastic material, geometric non-linearity and the representation of fire action are done according to a standard parametric curve.

Keywords: performance-based design; PBD; system approach; steel structures; truss structures; thermo-plastic material; non-linear analysis; fire safety.

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1 Introduction

Modern codes endorse two different ways for the design of structures subjected to fire: either by means of a prescriptive approach or with a performance-based approach. A prescriptive code provides for fire safety by prescribing some combination of specific requirements, without referring to the desired safety level or how it is achieved. In comparison, a performance-based code allows any solution that can lead to an *a priori* imposed safety level. In some cases, for example when dealing with complex structures where it is impossible to comply with all the architectonical prescriptions of a prescriptive code, a performance-based approach is more appropriate in obtaining the optimal structural behaviour under fire, and a prescriptive code proves to be inadequate. With specific reference to fires, structures can be defined ‘complex’ if they are characterised by aspects like: complex geometry, complex or large fire compartments, made by innovative materials, high structural redundancy, large spans.

Performance-based design (PBD) has been extensively introduced in fire engineering standards in the last years (Buchanan, 1994; Hadjisophocleous et al., 1998; Beyler et al., 2007) and many research studies on this topic have been produced. In this context it has been recognised that advanced analysis methods are essential in performance-based fire design (PBFD) evaluations (Gentili et al., 2010). However the studies focused on PBFD generally refer to simple structures or single structural elements (Yang et al., 2011; Dwaikat and Kodur, 2011) and only some applications are focused on complex structures (NIST NCSTAR 1, 2005). Complex structural systems need to be deeply investigated in order to consider all the aspects affecting the structural performances.

Furthermore, in the case of complex structures, additionally to the structural fire resistance, some other broadly recognised design problems, issues and trends must also be faced (Bontempi, 2005; Bontempi et al., 2008), summarised below:

- due to the advantages arising from the decomposition of complex structures in simpler parts (called substructures or components) having specific functions, it is useful to adopt a system approach in designing and assessing this kind of structures
- a hierarchy order is needed for determining the connections and the dependences between different parts
- non-linear behaviours usually play a prominent role in the response of complex structures to the actions induced by the surrounding environment
- mutual interactions between different components and between the structure and the actions cannot be disregarded in the structural analyses
- the structural behaviour must be investigated both in terms of components and in terms of the structure as whole
- the uncertainties regarding both measures (epistemic) and physical phenomena (aleatory) clearly affect the structural behaviour and, as a consequence, the more complex is the structure, the more uncertainty affects the structural behaviour.

In this study, the design scenario that considers the development of a fire inside a structure is referred as a low probability-high consequence event (Starrosek, 2009; Brando et al., 2012). This classification identifies such events with two features:

1 they occur rarely

2 can lead to severe consequences on the structural safety performance.

The use of common probabilistic methods usually fails in the treating of such events due to the lack of data for the characterisation of their intensity.

On the basis of the above premises, this paper focuses on the application of the PBFD for complex structures, with the main goal being to outline some specific issues related with this kind of problems. In particular, the systemic approach is adopted as proper tool to govern all the complexities related to performances definition and evaluation.

Conceptually the paper is organised as follows: Section 2 introduces the concepts of PBD for complex structures; Section 3 focuses on issues related with the treatment of complex structures under fire, while in Section 4 an example of performance assessment of a complex structure subjected to fire load is presented. Finally, some concluding remarks are given.

2 The PBD approach for complex structures

The PBD is a modern approach that allows designers to consistently take into account all the aspects related to the serviceability and safety of both existing and new structures without enforcing any limitation to the available design solutions. PBD has been mainly formalised and specialised for earthquake engineering applications. Several frameworks has been proposed for PBD in Earthquake engineering, and as declared in Krawinkler (1999) "they differ in details but not in concepts". Extensions to other design situations, like blast (Hamburger and Whittaker 2003; Crosti et al., 2012), wind (Petrini and Ciampoli, 2011), tsunami (Riggs et al., 2008) scenarios, have been recently proposed.

As previously stated, PBD is practically required in case of complex structures due to the fact that prescriptive approach is inadequate in dealing with non-ordinary configurations.

In general, a PBD approach must consider the whole life-cycle of the structural system including the decommission and the demolition phases (SEAOC, 1995). By neglecting these two phases from the subject of this paper, it is worth schematising the PBD approach for complex structures in two main steps:

- *Conceptual organisation of the design.* The first step regards:
  1. the qualitative definition of the performance requirements (generally related with structural safety, serviceability and robustness)
  2. the conceptual organisation of the structural system in its different parts and their reciprocal connections
  3. the individuation of the acting hazards and their intensities; the magnitude of the actions is expressed in terms of proper intensity measure parameters (IMs).

  At this stage, the performances requirements and limit states have to be taken into account in defining a suitable initial design configuration and in discriminating unfeasible configurations that are expected to being not able to fulfil the performance requirements. This task is particularly challenging for two reasons:

  1. the main choices in this preliminary design are essentially founded on the personal expertise of the designer and on the historical cases regarding similar structures
  2. in case of complex structures, mechanisms for stress transmissions and stiffness couplings between different structural elements and/or structural parts are really complex to predict, especially for safety of robustness evaluations.

  A suitable tool to govern the complexities arising in carrying out this phase is given from the structural system decomposition, represented by the design activities related with the classification and the identification of the structural system components, and by the hierarchies (and the interactions) between these components. The decomposition is carried out focusing the attention on different levels of detail: starting from a macro-level vision (related to whole structure) and moving on towards the micro-level details (related to connections between elementary structural elements); for more details the reader is referred to Petrini and Bontempi (2011). Under this point of view, specific performances have to be defined for single structural component (this is the philosophy of the so-called ‘element-by-element’ design), for the sub-structures and for the whole structural system.

- *Performance investigation of a structural design configuration*. After an initial structural design configuration has been defined in elements and in its system behaviour and the performance requirements, the quantitative definitions of some structural performance indicators and the respective limit states need to be carried out. The structural performance indicators are proper response parameters (often called Engineering Demand Parameters - EDPs) that have to be quantified under different loads with various intensities (IM values), the attaining of fixed thresholds for this EDPs defines the limit states of the related performance, while the overcrossing of these thresholds is assumed as the failure in fulfilling the performance requirements. The different limit states are usually associated with so-called damage states expressed in terms of proper damage parameters (DMs), DMs can be also assumed as structural performance indicators. The performances of the structural configuration must be quantified and eventually optimised by exploring alternative design configurations. Two main aspects have to be considered in this phase:

  1 In general, and especially for complex structures, the performance assessment must be carried out by the avail of advanced models adopted for both structural behaviour and actions. These circumstance increases the incidence of the modelling uncertainty with respect of ordinary models.

  2 Traditionally, two main philosophies can be adopted in assessing the performance:

     a probabilistic approach

     b heuristic approach.

  The probabilistic approach is usually feasible in case of performances investigation of structural configurations and actions whose can be statistically characterised by the avail of databases with a satisfying amount of data, while the heuristic approach may be preferable when the designer must deal with accidental actions and/or structures with raw statistical data.

2.1 Performance investigation by the probabilistic approach

In a probabilistic approach to the PBD, each structural performance is characterised by means of the probabilistic description of a (preferably scalar) Decision Variable (DV). Each DV is a measurable attribute that represents a specific structural performance (no collapse, occupant safety, accessibility, full functionality, admissible displacements or accelerations, etc.), The performance requirement is identified with an acceptable value of the probability G(DV) of exceeding a threshold value DV

$$G(DV) = \int\int\int G(DV|DM) \cdot f(DM|EDP) \cdot f(EDP|IM) \cdot \\ \cdot f(IM) \cdot dDM \cdot dEDP \cdot dIM \tag{1}$$

where G(•) is the complementary cumulative distribution function and G(•|•) the conditional complementary cumulative distribution function; f(•) is the probability density function, and f(•|•) the conditional probability density function. DM, EDP and IM have been previously introduced.

When possible, the DVs are expressed in monetary terms representing the economic losses due to the attained damage level and, under this point of view, this probability conventionally measures also the structural risk.

The performance (or risk) analysis is decomposed in its elementary components, which can be conducted in a subsequent order: the hazard analysis, in which the goal is assessing *f*(IM), the structural analysis, with the goal of assessing the probabilistic structural response *f*(EDP|IM) given IM, the damage analysis, with the goal of assessing the probabilistic damage measure *f*(DM|EDP) given EDP and, finally, the loss analysis, finalised to assessing the probabilistic loss in terms of decision variable *G*(DV|DM) given DM.

2.2 Performance investigation by the heuristic approach

As the complexity of the performance investigation problem grows, the need of more adequate methods to face this problem is evident. Beside the structural one, others component of the overall problem complexity arising in performances evaluation are given by:

1 the difficulties arising in characterising the IM from a probabilistic point of view

2 the needs of exploring extreme structural behaviours (e.g., progressive collapses or extremely damaged configurations or sudden changes of structural configuration).

The two components listed above are typical of those events that, due to both the possible induced structural collapse and their low occurrence, are called "Low Probability and High Consequence (LP–HC)" events (Ellingwood, 2009). As a matter of fact, the risk associated with these events is high but their probabilistic characterisation (e.g., the likelihood of the risk occurrence) is practically not reliably definable by adopting classical probabilistic methods.

In these cases, the more appropriate method to treat with the hazard may be the identification of pragmatic risk scenarios.

If the hazard is defined in terms of a scenario (or set of scenarios), equation (1) can be rewritten as in Ellingwood (2010):

$$\begin{aligned} G\left(\mathrm{DV}\middle|\mathrm{Scenario}\right) = \iint & G\left(\mathrm{DV}\middle|\mathrm{DM}\right)\cdot f\left(\mathrm{DM}\middle|\mathrm{EDP}\right) \\ & \cdot f\left(\mathrm{EDP}\middle|\mathrm{Scenario}\right)\cdot d\mathrm{DM}\cdot d\mathrm{EDP} \end{aligned} \tag{2}$$

and by collecting a suitable set of plausible scenarios, the unconditional probability G(DV) can be approximately assumed as the sum of the conditional probabilities computed by the (2) for all the scenarios among the set.

The equation (2) (and its convolution with respect to the domain of all possible scenarios), still represents a tool for both performances evaluation (each performance can be still conventionally identified as the structural risk with respect to a specific limit sate), but with respect to the (1), an heuristic component has been introduced in selecting a certain number of limited, pragmatic scenarios.

If the PBD regards a complex structure, additional heuristic considerations need to be made, for example the threshold levels for EDPs, DMs and DVs defining the lack of performance, are not rigorously definable for high redundant structures (e.g., the collapse of a certain number of structural elements does not necessary triggers the global collapse), as well as the connections between the EDPs, the DMs and the DVs (e.g., the collapse of a fixed number of structural elements produces a damage that depends both from the hierarchy of the collapsed elements with respect to the others and from the collapse sequence). In these cases all the elements of equation (2) may be hard to characterise from a probabilistic point of view, and a set of deterministic analyses can be preferable. Under these conditions, the performance given by a design configuration can be conventionally defined as the minimum resistance level given by the design configuration treated as deterministic under a set of deterministic load scenarios.

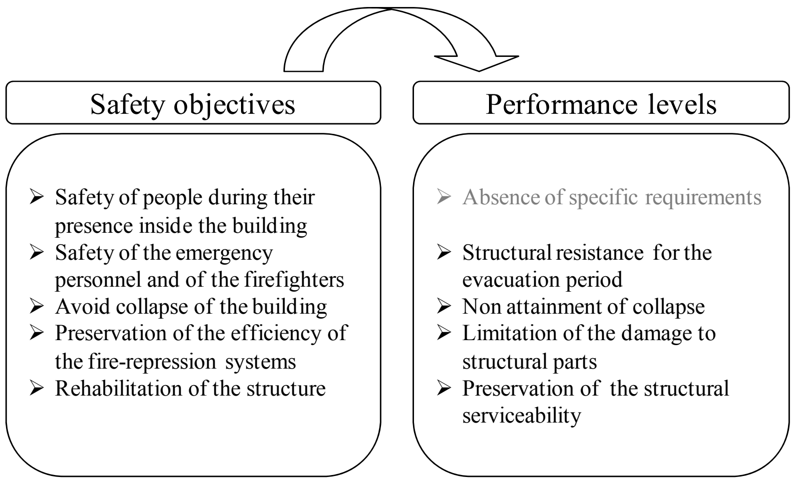
In this heuristic approach the performance is no more identified as equal to the structural risk with respect to a specific limit sate, but it is conventionally defined like an 'impact', as the consequences if the risk occurs.

3 Performance-based fire design

The PBD methodologies are widely used in fire engineering. Since fire can be viewed as LP-HC event, the heuristic approach is extensively adopted for this kinds of problems (ISO/PDTS 16733).

The performance of structures under fire is usually levelled with direct reference to the safety objectives as described in Figure 1, adapted from Crosti and Bontempi (2008) the reader is referred to Grosshandler (2007) for additional safety objectives.

Figure 1 Relation between safety objectives and performance levels



Generally, when assessing the performance of a structure subject to fire, a conventional collapse is defined, related to the typology of structural element and to the function that it must accomplish. In the particular case of steel beams, it is possible to define the conventional collapse when the maximum vertical displacement of one node of the element, reaches a certain limits (e.g., a ratio equal to L/20, where L is the length of the beam). The temperature and time of fire exposition that correspond to this displacement of the considered element, are defined respectively as the critical temperature and the critical time. As stated before, in case of complex structural systems, the conventional definition of the collapse may prove to be important and not trivial.

It is well known that, satisfactory performances of structures under fire can be achieved mainly by three strategies: active or passive protection and structural robustness. The case study presented in this paper focuses on the assessment of structural performance without any kind of protection in order to check the robustness of the structure under fire.

A fundamental aspect in conducting PBFD studies, regards the modelling activity. Three different numerical models have to be implemented (Buchanan, 2001):

1 a fire model that allows the study of the fire development (Gentili et al., 2012a)

2 a heat transfer model that allows to take into account the internal temperature of elements

3 a structural model for calculating the structure load bearing capacity, which takes as input the temperatures obtained from the heat transfer model.

The structural analysis has to take into account the effects of thermal expansion, loading and unloading, large deformations and thermo-plastic material. In the PBFD approach the check of performances is made by making use of analytical, physical or combined (analytical physical) models (Crosti, 2009).

3.1 Special issues in the PBFD of complex structures

In case of PBFD of complex structures, the arguments introduced above such as: use of the heuristic approach, difficulties encountered in conventional collapse and performance threshold definition, use of advanced models, assume a crucial importance. In this context, specific examples of these issues are listed below.

1 Methods for performance investigation. Probabilistic approaches are more suitable in case of PBFD of simple structures due to the relatively low number of variables.

2 Scale level for performance investigations. In simple structures the fire performance investigation can be usually conducted with reference to the structural key elements, which are limited in number, on the other hand for complex structures the fire resistance of single elements is not significant while the fire resistance of the system as whole is more relevant.

3 Performance thresholds and collapse definition. In simple structures the thresholds applied to proper response parameters to define the lack of performance are easily definable referring to the single structural elements, on the contrary for high redundant structures the collapse of a limited number of secondary elements do not necessarily imply the lack of performance.

4 Adopted models. Advanced structural models are needed in PBFD of complex structures due to the necessity of assessing global collapses rather than local ones, and advanced fire model are usually needed in order to assess the fire propagation.

5 Difficulties in determining proper fire scenarios. The fundamental fire scenarios are usually easily identifiable and limited in number for ordinary structures, while for complex structures this step is not so trivial.

6 Complexity of fire compartments. Beside the structural complexity, another parameter playing a prominent role is the configuration of the fire compartments, in fact the compartementation determines the size and the geometry of structure directly engaged by the fire. In case of simple structures, compartments are usually well defined and with simple geometries. In addition, complex geometries for fire compartments increase the uncertainty regarding the effectiveness of the compartments.

Arguments treated above are resumed in Table 1 where LP-HC events (like fires) are compared with ordinary events and in Table 2, where complex structures are compared with ordinary structures.

Table 1 LP-HC versus ordinary events

	Ordinary events	*LP-HC events*
Approach for performance investigation	Probabilistic	Heuristic
Statistics	Complete	Incomplete
Uncertainties	Low	High
Models	Ordinary	Advanced
Load scenarios	Simple	Complex

Table 2 Complex versus ordinary structures

	Ordinary structures	*Complex structures*	*Notes*
Design approach	Prescriptive – PBD	PBD	
Minimum check level	Element	Element – global	Investigations at a global level for robustness assessment
Models	Simple-ordinary	Advanced	Models are intended having same complexity both for structure and actions
Approach for Performance investigations	Probabilistic (Performance = structural risk with respect to a specific limit sate)	Heuristic (Performance = 'impact', identified with the consequences if the risk occurs)	Also semi-heuristic (Performance = structural risk with respect to a specific limit sate and to a specific scenario)
Fire scenarios	Easily identified and limited in number	Not trivial to define and great in number	
Definition of performance thresholds and collapse	Simple-ordinary	Not trivial	e.g., for high redundant structures the collapse of a limited number of secondary elements do not necessarily imply the lack of performance
Compartmentation	Simple	Complex	

3.2 A systemic-based framework for PBFD of complex structures

The framework shown in Figure 2 is proposed as general method for PBFD of complex structures. Given a structural configuration, and aiming at the evaluation of its fire performances, under this framework the analysis can be implemented by the following 6 steps:

1 Definition of global performance requirements. A total of P qualitative performance requirements are defined at global level for the whole structural system.

2 System decomposition. This phase has been previously introduced in Section 2: the structural system is decomposed in its parts (components) both at meso-scale level (substructures) and at micro-scale level (structural elements and connections). The components are hierarchy ordered. Here say that a total of N components are identified for the system.

3 Fire sources identification. The possible fire sources are identified and a picture of the intensities of corresponding fires is provided. Consider a total of S fire sources.

4 By the information provided at the steps 1 and 2, it is possible to define the required performances both for components and system. As previously introduced (see Section 2) proper performance indicators (EDPs or DMs) are identified for each component defined at the step 2. Proper performance thresholds (EDP* or DM*) are then fixed for each performance indicator by referring to the performance requirements fixed at the step 1. Performance In this phase, relations between the component performances and the global performances are established by referring to the hierarchies defined at the step 2. The relations between components and system performances can be profitably synthesised by the use of logical matrices as represented in Figure 2.

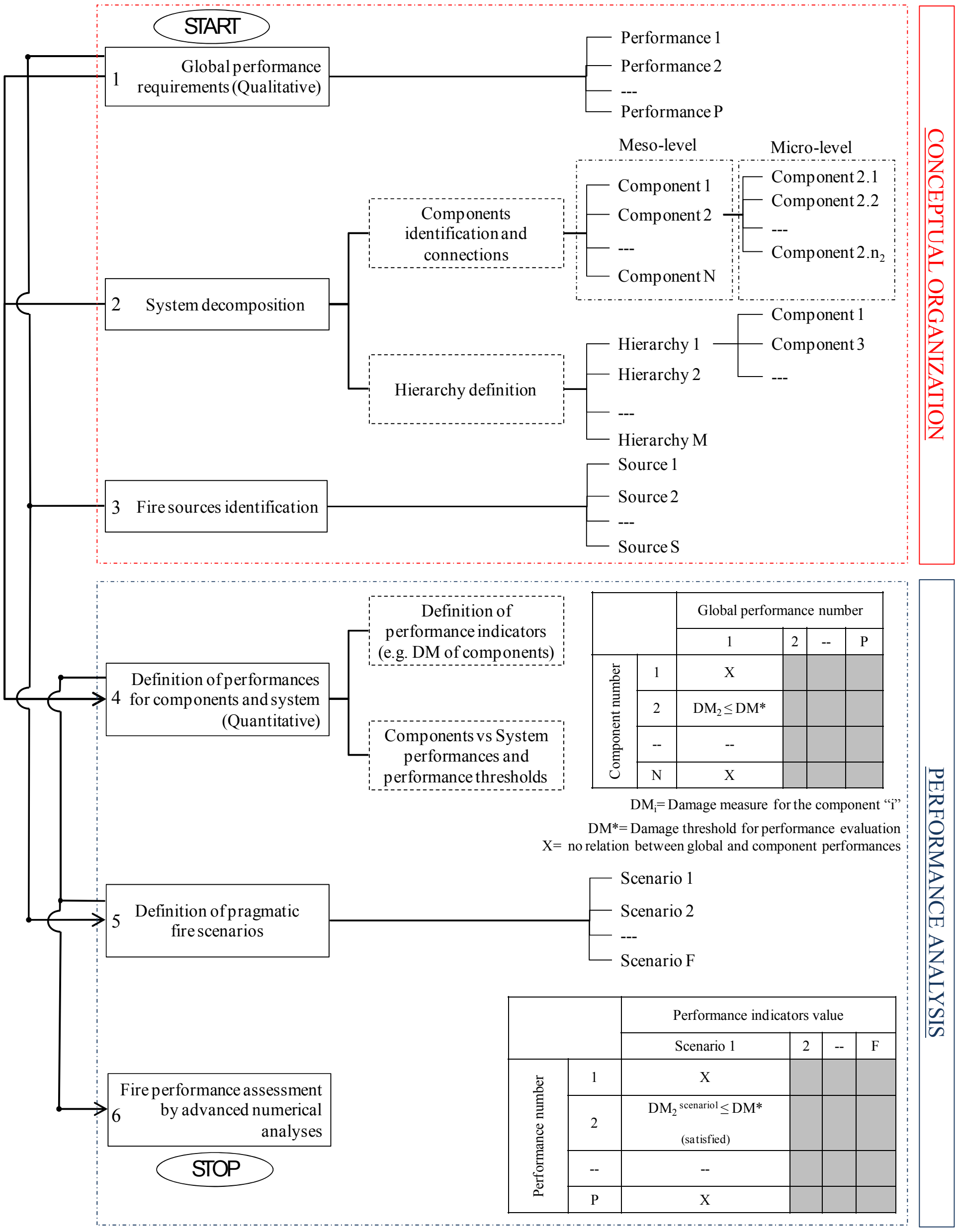
5 From the analyses carried out at the steps 1 and 3 it is possible to define a number F of pragmatic design fire scenarios.

6 By the implementation of advanced numerical analyses (Gentili et al., 2012b; Saviotti et al., 2012), the performances defined at the step 4 are assessed for the structural system under the fire scenarios defined at the step 5.

If the numerical analyses are carried out in deterministic way, a total of ‘F’ analyses are needed in order to evaluate a number ‘P’ of performances. By referring to the general aspect introduced in Section 2, the steps 1, 2 and 3 provide the conceptual organisation of the design, while steps 4, 5 and 6 allow the performance investigation of a structural design configuration.

In the following section, an application of PBFD to a complex structure is presented. The Heuristic approach is adopted and the investigations are carried out at a global level (point 1 and 2 of the above list) some arguments related to the issues 3 and 4 are treated.

Figure 2 General framework for PBFD of complex structures (see online version for colours)



4 Case study: a structure for helicopters storage made in steel

The structure under inquiry is a real life industrial facility in steel, used for the storage and maintenance of helicopters, therefore it presents with an elevated fire risk. The main goal in carrying out this application is:

- to show the utility of the system approach in dealing with the PBD of complex structures under fire
- to apply the framework introduced above
- to show the issues arising in evaluating the performance of complex structures under fire.

4.1 Structural system, fire scenarios and numerical models

The facility is 64.64 meters long, 32.85 meters wide and has a maximum height of 12.9 meters as shown in Figure 3. This facility presents a relatively complex geometry. The structure is isolated, it is symmetrical both in the x and in the y direction and it has a truss covering. Its six columns, made of steel elements, are founded on concrete blocks.

The triggering event considered is the helicopter caught fire. In this case, the fire remains localised if no nearby helicopters catch fire from the burning one. Fire ignition sites are selected among all possible locations of helicopters, on the basis of the most adverse locations (that would have the most severe affect on the structural performance of the facility). The fire due to a single helicopter burning is modelled using the ISO 834 nominal fire curve (time ‘t’ versus temperature ‘θ’, see Figure 4) found in Eurocode 3 (EN1993, 2004). The temperature is applied to the structural elements which are located along the total height of the structure and contained in the parking area of the ignited helicopter (of approximately 50 square meters), by considering that elements are not protected from the fire. Three pragmatic fire scenarios are considered, that are obtained by assuming three different locations as susceptible for fire ignition. The fire locations and the heated structural elements in the three cases are shown in Figure 4 while Figure 5 shows two pictures of the typical configuration for hypothised heated elements. By the way, no models for both room fire development and heat transfer inside the steel have been implemented. Both structural and non-structural vertical permanent loads are considered acting on the structure when the fire develops.

The finite element model of the structure is developed in the commercial code Straus7/Stand7 (http://www.hsh.info, http://www.strand7.com). All the steel members are modelled by beam elements and at least a mid-node has been inserted in each bar in order to compute the single bar buckling. Material non-linearities have been taken into account since thermo-plastic characteristics of steel has been modelled by considering the decay of the material with the temperature in terms of elastic modulus and yielding stress (initial value is set equal to 206,000 and 235 N/mm^2 respectively). Standard decay laws has been assumed from Buchanan (2001). Geometric non-linearities have been considered by adopting in the analysis the large displacement assumption, the structural response has been investigated by a transient analysis, setting the structural damping ratio near to 85% for the first 10 vibration modes.

Figure 3 Geometry of the facility (see online version for colours)

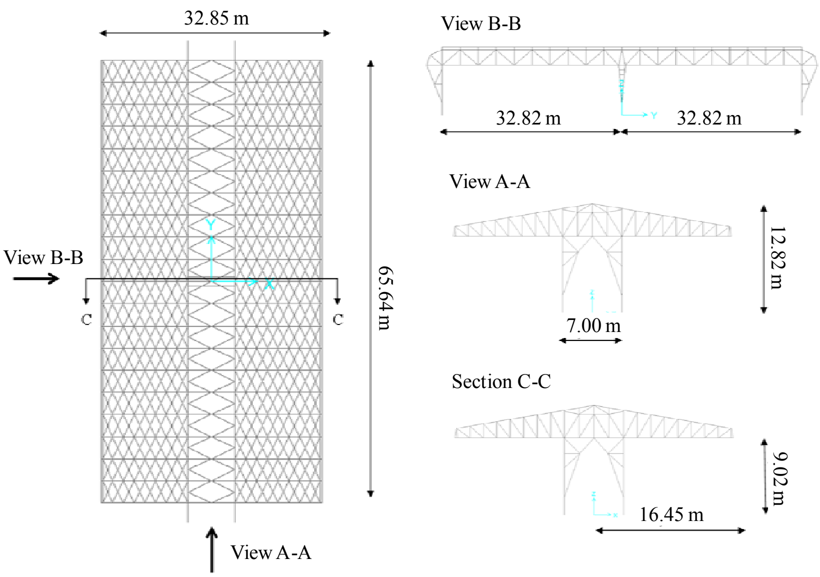


Figure 4 Depiction of the fire scenarios and of the ISO 834 curve (see online version for colours)

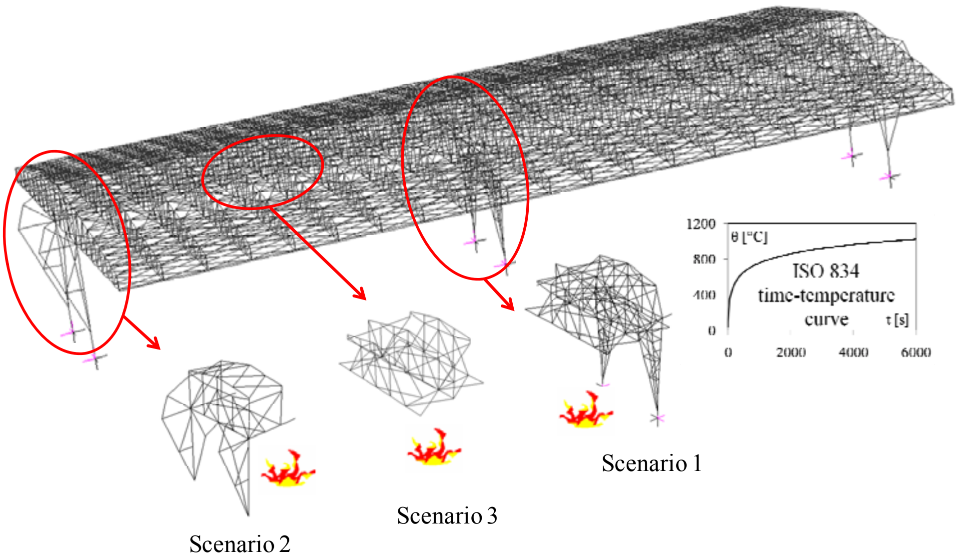


Figure 5 Photos of the steel elements (see online version for colours)

4.2 Structural decomposition and performance requirements

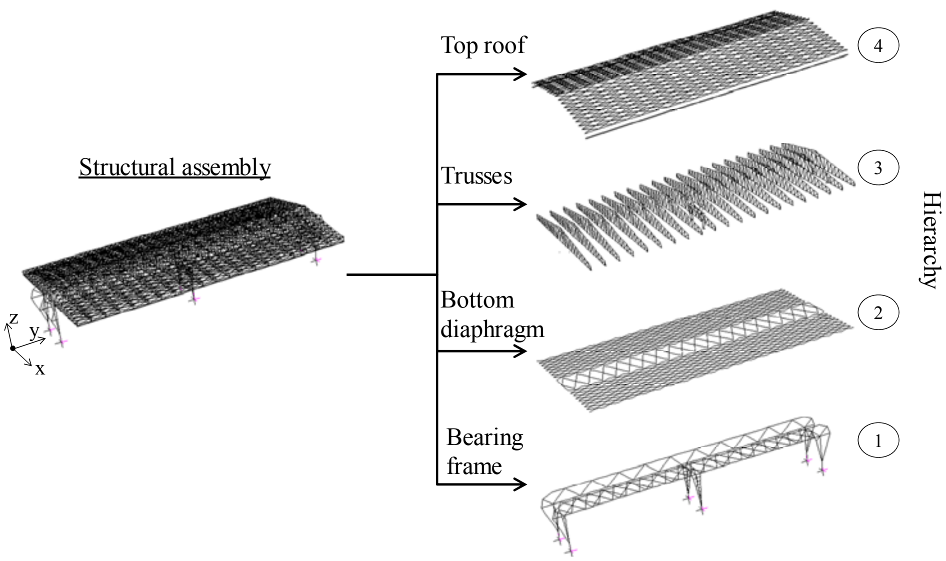
A decomposition of the structure in its main sub-structures is shown in Figure 6, where four principal components are identified and hierarchically ordered. A global or local failure of such substructures can be directly connected with the lack of performances hierarchically ordered in the same manner:

- Hierarchy level 4. The 'Top roof' component has the double function of carrying roof loads and furnish an upper diaphragm for horizontal loads connecting the sustaining trusses. The top roof component is formed by elements having circular cross sections with a diameter equal to 0.012 meters. A local failure of the this component does not affect the global stability of the structure or the local stability of other sub-structures, while a global failure of the top roof component is possible only in case of contemporary failure of other components having higher hierarchies. For this reason it is considered at the lower level of the structural hierarchy and failures of this component are associated with temporary and partial service interruptions for the rehabilitation of the facility.
- Hierarchy level 3. A total of 20 trusses sustaining the roof form the 'Trusses' component located at the third structural hierarchy level. The trusses are formed principally by coupled elements having L-shaped cross sections with different dimensions ranging from 0.03 to 0.08 meters and thickness between 5 and 8 millimetres. The local collapse of some elements forming truss component causes definitely an out-of-service period and the failure of a considerable amount of these elements could compromise the local stability of the structure.
- Hierarchy level 2. The 'Bottom diaphragm' component is formed by structural elements constituted by four joined bars having L-shaped cross sections and with the same dimensions. In the central alignment the dimensions of the cross sections are 0.08×0.08 meters and the thickness is 8 millimetres, while in the external alignments the dimensions are 0.04×0.04 meters with thicknesses of 6 millimetres. The local collapse of the Bottom diaphragm can compromise both the vertical and horizontal stability of the structure, while the global collapse of this component can cause the progressive collapse of the whole structure. For these reasons the integrity

of the bottom diaphragm is essential to the structural robustness and safety performance.

- Hierarchy level 1. Finally, the most important sub-structure is identified as 'Bearing frame' and is formed by reticular columns connected together with two reticular beams. All the structural elements forming this component are obtained by joining two or four bars having various cross sections (L- or C- shaped) whit dimensions falling in the same ranges chosen for the components previously described. The local or global collapse of the Bearing frame can be identified to the collapse of the whole structure.

Figure 6 Structural decomposition with different hierarchies (see online version for colours)



On the basis of the above premises, the performance requirements introduced in Figure 1 can be associated with the response or with the state of the different substructures as shown in Table 3. In the same table, the performances are also associated with proper performance indicators (EDPs and DMs) adopted for their quantitative evaluation. The description of these parameters is given in the next section.

The damage for the application of the requirements is defined by the percentage of structural elements which experiment the collapse (yielding or instability) among a certain sub-structure. Thus, for example, the damage of the component having hierarchy 2 (Bottom diaphragm) related to the fire scenario 2 is obtained as the percentage ratio between the number of collapsed structural elements belonging to the Bottom diaphragm and the total number of elements forming the Bottom diaphragm, namely

$$DM_2^{\text{scenario 2}} = \frac{\left(n^\circ_{\text{collapsed elements}}\right)^{\text{scenario 2}}}{\left(\text{total } n^\circ \text{ of elements}\right)_2} \cdot 100 \tag{3}$$

Concerning the definition of the damage levels, here the following levels are assumed: irrelevant damage (0 < *DM* < 1%), moderate damage (1% < *DM* < 5%), average damage (5% < *DM* < 10%), significant damage (10% < *DM* < 15%) and unacceptable damage (*DM* > 15%).

On the basis of these DM levels, the fire scenarios will be graded from the most dangerous to the less one.

4.3 Performance indicators

One of the main tasks in assessing the performances is the choice of proper quantitative performance indicators. As introduced in Section 2, the final indicators for risk or performance would be proper DVs which are conceived to quantify the losses in monetary terms. When the losses evaluation are neglected, a simplified evaluation of the performances can be made on the basis of EDP or DM evaluations. In case of complex structures the choice of proper EDP or DM and the setting of opportune performance thresholds (EDP*, DM*) for these parameters are particularly problematic. In this application the following performance indicators have been defined:

- EDP_v. Vertical displacement evaluated along the alignments shown in Figure 7 during the fire (computed relatively to the extremes of the alignments). These EDPs can indicate both the local collapse of a component, if the maximum of these displacements, named EDP_v_max, is greater than the established performance threshold, and the progressive collapse, if the initial local collapse drives the successive unconfined increasing of other displacements along the alignment (progression of the run-away phenomenon) (Usmani, 2003). Concerning the definition of local collapse, the collapse threshold for EDP_v_max has been assumed as derived by the criterion given in (Petterson et al., 1976) steel beams

  $$\mathrm{EDP\_}v\mathrm{\_max}^* = \frac{L^2}{800 \cdot H} \tag{4}$$

  where *L* and *H* are the beam length and height respectively. By computing the (4) with the dimensions of the reticular beams forming the 'Bearing frame' component (*L* = 32.82 m, *H* = 2.85 m) the threshold value EDP_v_max* = 0.472 m, the same threshold has been assumed to checking the collapse of components of hierarchies 1, 2 and 3.
- EPD_o. Lateral displacement of the nodes of the columns during the fire. This EDP can reveal instability of the column by showing a sudden change in its time trend (e.g., a change in the incremental direction during the column expansion under monotonic fire curve).
- DM. Measure of the structural component damage during the fire. Computed as indicate in equation (3). The level of damage suffered by a certain structural component determines the acceptability of the structural performance.

Table 3 Relations between performance requirements, component hierarchies and EDP and DM parameters

No	*Qualitative performance requirement*	*Quantitative performance requirement*	*Performance indicators (EDP or DM)*	*Note 1*	*Note 2*
1	Structural resistance for the evacuation period	No collapse for components of hierarchies 1 and 2 for 15 minutes	• EDP_v_max. Maximum vertical displacement among those evaluated along the alignments shown in Figure 7 • EDP_o. Horizontal displacements of the nodes of the columns.	15 minutes is the time period which is necessary to extract from the facility the helicopter nearest to the burning one.	Collapse is assumed in case of: • EDP_v_max greater than EDP_v_max* = 0.472 m (see equation (4)) • instability of a column (defined by a suddenly change in the trend of the EDP_o)
2	No attained of collapse	No collapse for components of hierarchies 1 and 2 during the all duration of the fire	NOT APPLIED	Components of hierarchies 3 and 4 are completely sustained by components having higher hierarchy	Fire models considering descending phase for fire are needed
3	Limitation of the damage to structural parts	a moderate damage (DM < 5%) for components of hierarchies 1, 2, average damage (DM < 10%) for components of hierarchy 3	a DM_i i = 1, 2, 3. Damage evaluated as described by equation (3)		
		b avoidance of progressive collapse	b EDP_v. Vertical displacement evaluated along the alignments shown in Figure 7		
4	Preservation of the structural serviceability	No damages (DM < 1%) for components of hierarchies 1, 2 and 3 and moderate damage (DM < 5%) to component of hierarchy 4	DM_i i = 1, 2, 3, 4. Damage evaluated by equation (3)		

Figure 7 Alignments of the structure (see online version for colours)

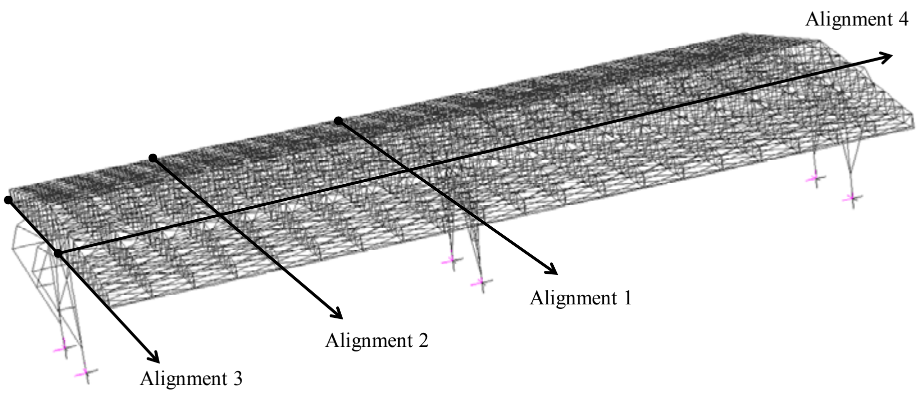
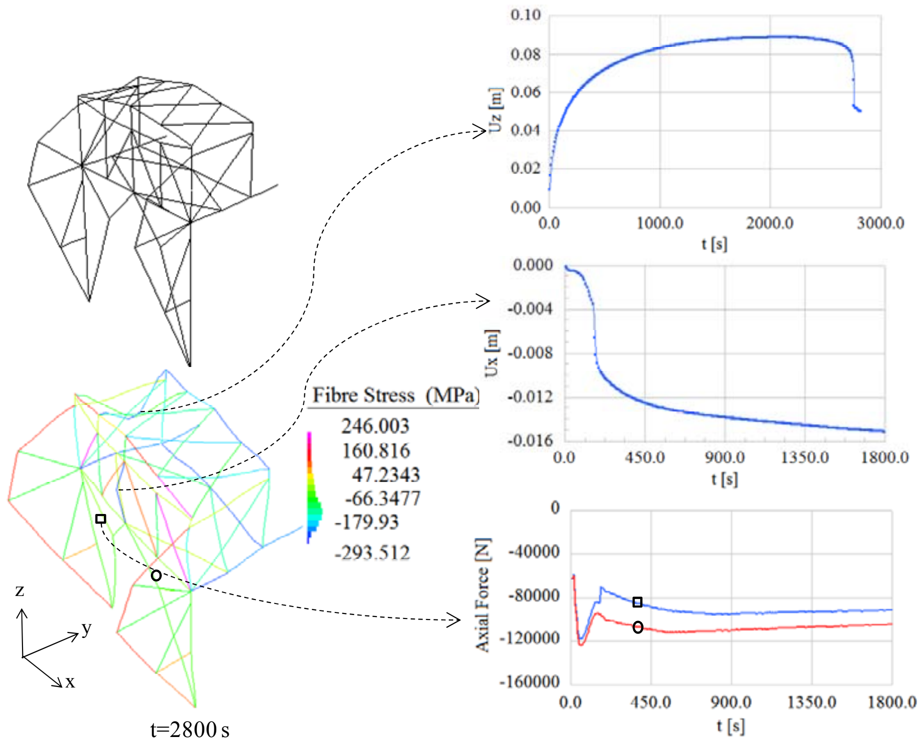


Figure 8 Scenario 3 detailed investigation (see online version for colours)



4.4 Results of the performance investigations

As previously stated, in the case of complex structures, the failure of individual structural elements either due to buckling or yielding does not imply the failure of the entire structure (where failure here is intended as the lack of performance). Failures of individual elements always occur in the conducted analyses. An example of this kind of structural response is depicted in Figure 8, corresponding to the analysis of the fire scenario 3. On the left side, the deformed shape of the heated elements at time 2,800 s is shown, with the indication of the maximum fibre stress attained in the structural elements; on the right side of the figure, some time histories of proper response parameter are shown. The upper and middle charts show the mid-point displacement time histories (in z and x direction) for elements that experiments the buckling, while the lower diagram shows the axial stress time histories of undamaged and stable elements that experiment sudden trend-changes due to the failure of the sustained elements.

Figure 9 Compared configurations of the three considered scenarios (see online version for colours)

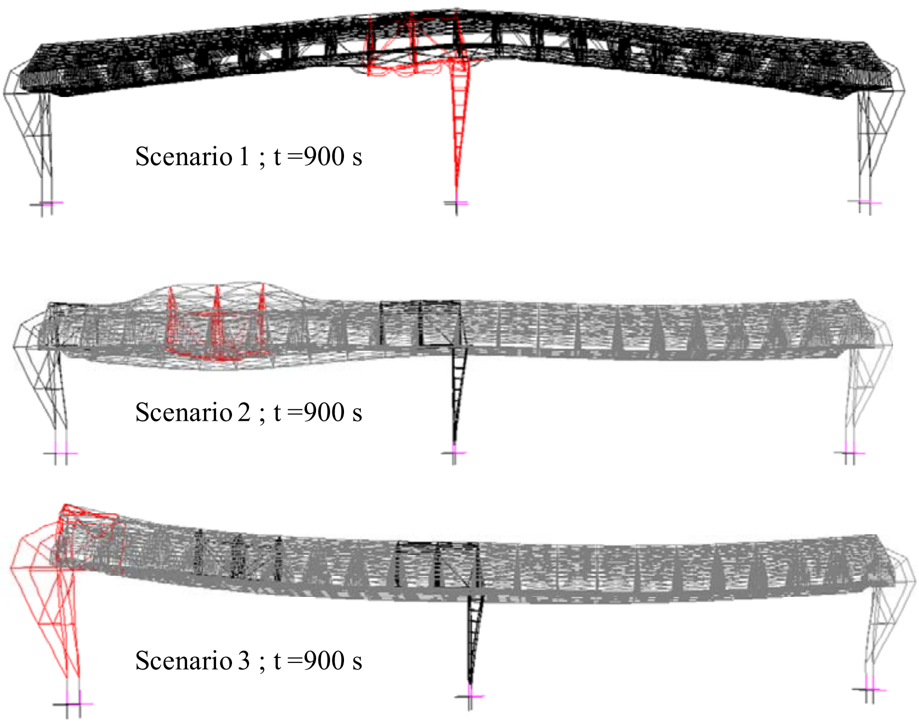


Figure 9 shows the deformed shapes obtained by the three scenarios at the time step which is significant for the first performance listed in Table 3 (no collapse for components of hierarchies 1 and 2 for 15 minutes), in this figure, the buckling of some structural elements can be appreciated. In Figures 10 and 11 the vertical displacements EDP_v obtained for scenarios 1 and 3 in nodes along the alignments shown in Figure 7 are reported. From Figure 11 can be deduced that both local and global collapses do not occur for the considered alignments under the fire scenario 3. In fact the value of the EDP_v_max remains under EDP_v_max*, and it is shown that the deformed configuration along the alignments maintains its general expanding shape only increasing in magnitude as the time (as well as the temperature) increases. The same considerations can be made concerning the Figure 10 for scenario 1, in this case the response shows a certain local damage denoted by a central drop of the curves in the zone of the scenario, this drop increase with the time.

Figure 10 Vertical displacements along the alignments 1 (upper) and 4 (lower) for scenario 1 (see online version for colours)

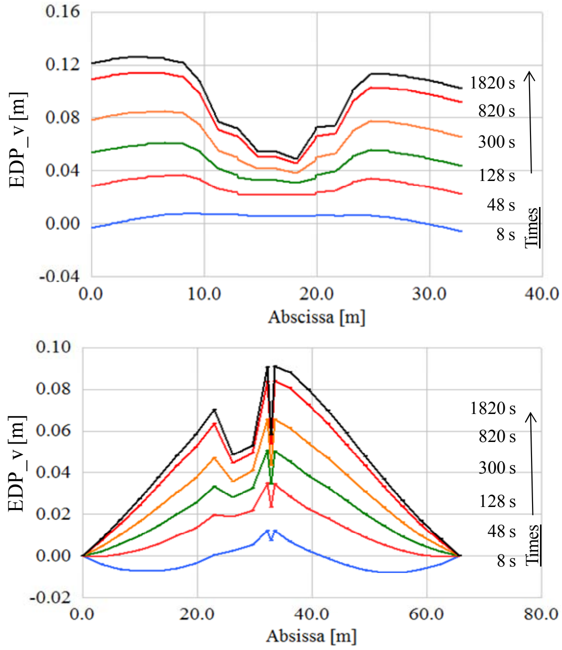


Figure 11 Vertical displacements along the alignments 3 (upper) and 4 (lower) for Scenario 3 (see online version for colours)

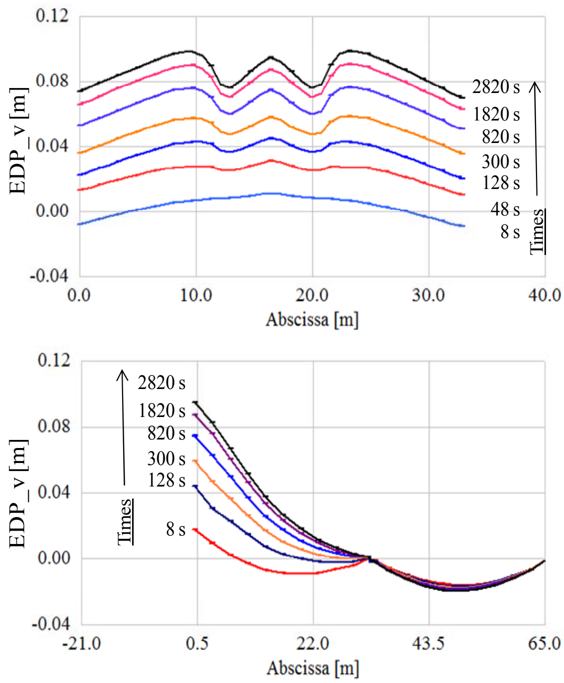


Figure 12 shows the values computed for the components damage obtained by the three scenarios at different times by the equation (3). The scenario 2 contains only two structural elements belonging to the component of hierarchy 1 (Bearing frame) and do not experiment failure both, since the scenario 2 does not appear in the upper left figure. From the Figure 12, the performance requirements 3a and 4 of Table 3 can be evaluated: the two performances requirements are not satisfied for scenario 1 due to the damage occurring to the components of hierarchies 1 and 2.

Finally, in Table 4, the performances evaluation is resumed. Referring to Table 3, it can be concluded that the structure fails in achieving the performance requirements 3 and 4, while at the present state the first performance requirement is satisfied, the investigation of the adequacy of the structure to the second performance criteria needs to be checked by the avail of fire models considering cooling phase.

As general consideration, it can be stated that the fire scenario 1 is the most challenging for the structure at the present state. This consideration suggests focusing on this zone the design strategies for improving the performance of the structure under fire.

Figure 12 Damage Measures (DM) of the three scenarios for structural components of different hierarchies: Bearing frame (hierarchy 1) (upper-left), Bottom diaphragm (hierarchy 2) upper-right), Trusses (hierarchy 3) (lower-left), Top roof (hierarchy 4) (lower-right)

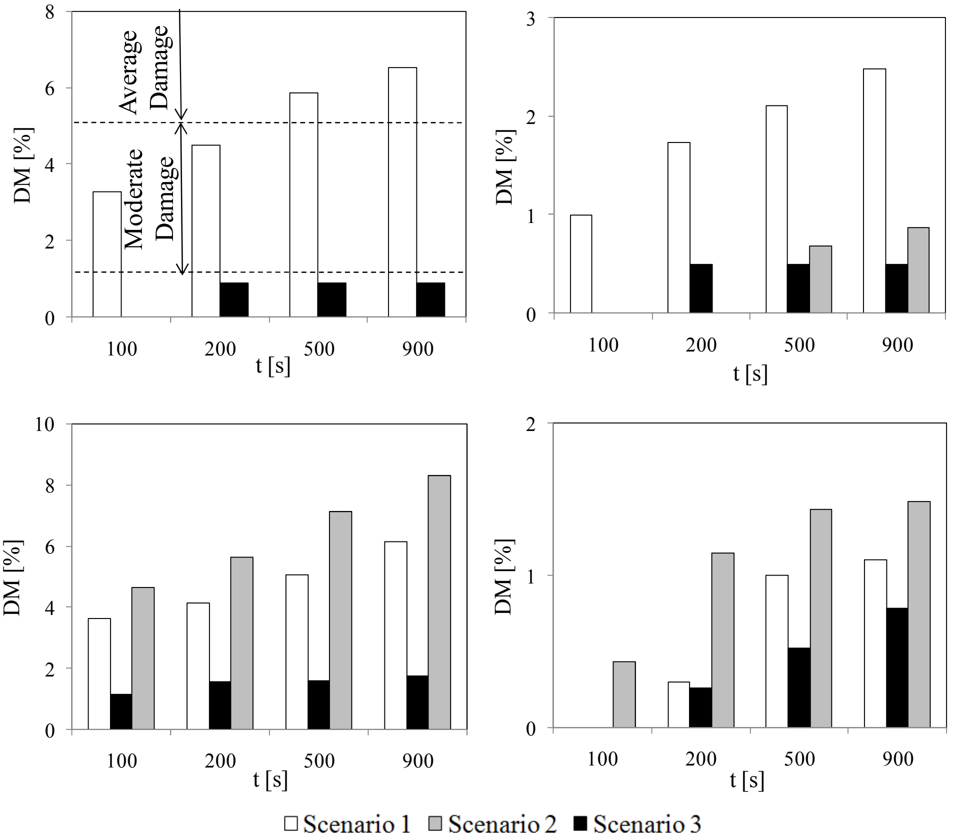


Table 4 Performance evaluation of the considered structure

No.	*Quantitative performance requirement*	*Performance indicators value*			*Performance result*
		Scenario 1	*Scenario 2*	*Scenario 3*	
1	No collapse for components of hierarchies 1 and 2 for 15 minutes	EDP_v_max (15min) = 0.128 m EDP_o (15min) does not show the columns instability Satisfied	EDP_v_max (15min) =0.057 m EDP_o (15min) does not show the columns instability Satisfied	EDP_v_max (15min) = 0.102 m EDP_o (15min) does not show the columns instability Satisfied	Satisfied
2	No collapse for components of hierarchies 1 and 2 during the all duration of the fire	Not applied	Not applied	Not applied	Not applied
3	a moderate damage (DM < 5%) for components of hierarchies 1, 2, average damage (DM < 10%) for components of hierarchy 3 b avoidance of progressive collapse	$DM_{1,}$ >5% at t = 500 s EDP_v does not show the progressive collapse *FAIL*	$DM_2 < 5\%$ $DM_3 < 10\%$ EDP_v does not show the progressive collapse Satisfied	$DM_{1,2} < 5\%$ $DM_3 < 10\%$ EDP_v does not show the progressive collapse Satisfied	*FAIL* for scenario 1
4	No damages (DM < 1%) for components of hierarchies 1, 2 and 3 and moderate damage (DM < 5%) to component of hierarchy 4	$DM_{1,2,3} > 1\%$ at time t = 100 s *FAIL*	$DM_3 > 1\%$ at time t = 100 s *FAIL*	$DM_{1,2,3} < 1\%$ $DM_4 < 5\%$ Satisfied	*FAIL* for scenarios 1 and 2

5 Conclusions

In this paper, issues related to the application of the PBFD of complex structures have been discussed. First, the general framework of the PBD approach and some complexities related to the design of complex structures have been separately introduced without making specific reference to fire engineering, while the connections between the PBD and the risk assessment analysis has been also outlined. Then, some specific issues related to the application of the PBFD to complex structures have been treated. The key issues are:

- the use of the heuristic approaches for PBFD rather than the probabilistic ones
- the need of advanced models to carry-out PBFD
- in the case of PBFD of complex structures, since the local collapse of a single or a limited number of structural elements do not necessary cause the lack of performance, there is the need to carry-out structural analyses that are able to reproduce high damaged structural configurations with marked non-linear behaviour
- the difficulties arising in defining both synthetic quantitative performance requirements and proper performance indicators in case of complex structures.

Some of these issues have been practically investigated with reference to a case study: a structure for helicopters storage made in steel. A general framework to manage these issues has been proposed, the system decomposition of the structure in components and association of different performance levels to various components behaviour has been performed. Quantitative performance requirements and proper performance indicators have been successively assumed and numerical analyses have been carried out to demonstrate the validity of the approach.

General conclusion is that the system approach is a powerful tool to rationally carry-out the PBD of complex structures. Concepts of these two frameworks can be profitably integrated in PBFD approach.

Classical probabilistic methods seems to be inadequate in dealing with such as complex problems at present state and research efforts need to be done in this direction

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