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Modelling of fire behavior of RC girders strengthened by external prestressing

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Abstract

Since the 1950s, external post-tensioning has been a powerful tool for retrofitting existing bridges. In unbonded prestressing, the post-tensioning tendons are positioned outside the concrete cross-section, and the prestressing forces are transferred to the girder through end anchorages, deviators, or saddles. This implies that these could be critical elements for the bridge performance against fire. This paper examines reinforced concrete bridges strengthened by external prestressing and subjected to fire, using a mixed experimental and analytical approach. Initially, load tests are conducted on a model of a bridge beam strengthened by external prestressing to assess the effectiveness of the intervention until failure. The ultimate limit state (ULS) capacity of the beam is then evaluated both before and after rehabilitation. Subsequently, the fire performance is investigated through different scenarios defined by the fire load curves of Eurocode, evaluating the cross-sectional temperatures resulting from fire, estimating the loss of external prestressing efficiency, and assessing the beam's overall fire resistance. This methodology is then applied to a real girder bridge with adjacent beams of the same type, extending the approach used on the experimental scaled beam to an actual bridge deck. Results are presented in terms of performance levels for different traffic load values and residual capacity over time from fire ignition, providing a safety assessment of girder bridges under fire exposure. The procedure is straightforward and immediately applicable to many existing reinforced bridges. The findings demonstrate how quickly external prestressing degrades, leaving the beam unprotected, and highlight the fundamental importance of the protective sheath for external tendons in limiting this degradation. Evaluating load levels based on exposure time also allows engineers to estimate the service time before rescue arrives, given the expected traffic level.

Keywords: Concrete bridges; fire performance; external prestressing; experimental test

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1. Introduction

The fire behavior of reinforced concrete bridges is a rather debated topic in literature, as it may result in the achievement of the Service Limit State or even the Ultimate Limit State, according to the accident that caused the fire and the characteristics of the structure. Among the numerous cases of fire that struck the bridges, documented in the literature (Garlock et al., 2012; Kodur et al., 2010; Lee et al., 2013; Zhang et al., 2022), those that have generated significant structural deficiencies after the event generally occurred in steel bridges or steel-concrete bridges, with high prevalence respect to RC bridges. The vulnerability of steel bridges to fire is in fact known and concrete bridges are generally considered with less vulnerability and therefore a lower risk (Kodur & Naser, 2013). When the event is prolonged and the fire load is such that the prestressing tendons of prestressed structures are invested by high temperatures until they deteriorate instead, concrete bridges can show sudden performance reductions due to the degradation of prestressing steel and the overall loss of prestressing on the structure (Jeyashree et al., 2022; Liu et al., 2022). Moreover, nowadays increasingly interest is attracted by the assessment of seismic vulnerability (Colajanni et al., 2023), and multirisk vulnerability due to the combined effect of earthquakes and fire (Khiali & Rodrigues, 2023).

The increasing use of external prestressing, that is, of non-bonded tendons inside the new box bridges or outside the concrete structures, for strengthening existing bridges (Recupero et al, 2014, Granata et al., 2021; Granata et al., 2023), significantly modifies the fire behavior. Very few studies on the behavior of prestressed bridges with external prestressing are considered in the literature. These bridges have the typical characteristics of reinforced concrete sections where the element of greatest importance for the structural behavior in service till failure is the external tendon, generally protected only by a rubber or PE sheath (Jeanneret et al., 2021). As a result, the steel element that provides the greatest contribution of structural strength is also the most vulnerable to fire, which is the most exposed.

When the use of the external tendon in an existing bridge serves to strengthen the reinforced concrete or initially post-tensioned girder (Colajanni et al. 2017a, Colajanni et al. 2014), then the structural performance in the new configuration depends on this element and its degradation in the event of fire, on the actual load of the fire or the temperature curve achieved, together with the causes of the fire and the location of the fire source or the area struck by the flames.

Most of the analyses in the literature concern steel or composite bridges, and they take into account the degradation of the steel (Del Prete et al., 2015; De Silva et al., 2023); therefore the behavior of the single prestressing strand or tendon (Atienza & Elices, 2009; Kotsovinos et al., 2020) or the prestressed bridge with embedded strands or with post-tensioned tendons (Bamonte et al., 2018; Eamon & Jenssen, 2012; Liu et al., 2019; Wu et al., 2020) are analyzed. Less investigations are carried out on the structure that initially retains its external strengthening (Ellobody & Builey, 2009; Gales et al., 2009; Gales et al., 2011; Zhou et al., 2018) and then changes its properties due to the degradation of the steel placed outside, returning to the original structure which continues to degrade, without relying on the contribution of prestressing. This is the case of RC bridges strengthened by external tendons. The time range in which this structural change occurs and the safety factors with respect to collapse may be of fundamental importance to ensure structural safety during fire or post-fire, making it the difference for saving human lives.

Particularly sensitive to these events are road intersections where the reinforced structures are above a road where the event occurs, especially in the presence of fires whose load is represented through hydrocarbon curves, because the sudden evolution of temperatures tends to unload the tendon of the original pretension drastically and quickly, reducing the performance and thus reducing the overall resistance of the structure.

This study presents the evaluation of the behavior of an existing reinforced concrete bridge, strengthened by external prestressing. The scale model of the bridge beam was used for an experimental campaign that provided the post-elastic behavior during the cracking stage till failure in the naked and prestressed configurations together with the load-displacement curves of bending tests. The fire analysis was performed first on the experimental beam, analyzing what reduction can be expected in strength by applying the fire load curves of literature (CEN, 2002) both standard and hydrocarbon ones. Afterwards, the same procedure was applied on the real bridge, repeating the thermal analysis and the performance degradation trend, correlated with the Service Limit State and the Ultimate Limit State or with the collapse values of the structure. The methods used for fire analysis and the results obtained in the various scenarios and the load curves used are reported, making it significantly in the assessment of existing bridges to fire, having regard to the number of bridges that are now being reinforced by external prestressing and, above all, the possibility that this will happen in motorway overpasses, subject to fire from below. Conclusions on the fire performance for actual bridges in such scenarios are given for engineering practice, in a perspective of large-scale fire assessment of existing bridges belonging to networks.

2. Experimental approach to load bearing capacity of beams with external prestressing.

The beam used for the experimental campaign is a reinforced concrete beam with a T-shaped cross-section, 5.40 m long and with a span length of 5.0 meters between the supports. The cross section has a maximum width of 600 mm, height 600 mm, thickness of the slab 150 mm and web thickness 200 mm and it is reinforced with 2 bars ϕ14 mm at the bottom chord, and 4+4 ϕ12 mm in the top slab. Four-point bending test is performed with two symmetrical loads in the center by a jack and a contrast frame. The beam is simply supported, and it is designed for failure in bending with ductile behavior. Subsequently, the beam is equipped with anchoring plates at the ends and intermediate saddles, in order to accommodate two symmetrical strands, placed side-by-side with the concrete web and post-tensioned through jacks on one end. Figure 1 shows the bending test scheme with the introduction of the strands and their layout as well as the test pictures.

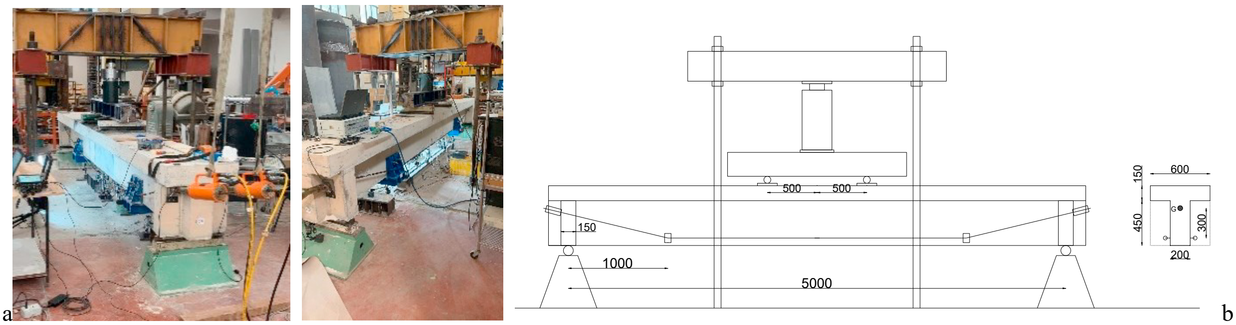


Fig. 1. Experimental test on prestressed beam. a) General views during prestressing phase. b) Bending test setup

Figure 2 illustrates the load-midspan deflection response of the bending tests on the beam for the case without prestressing and for the case with prestressing, respectively.

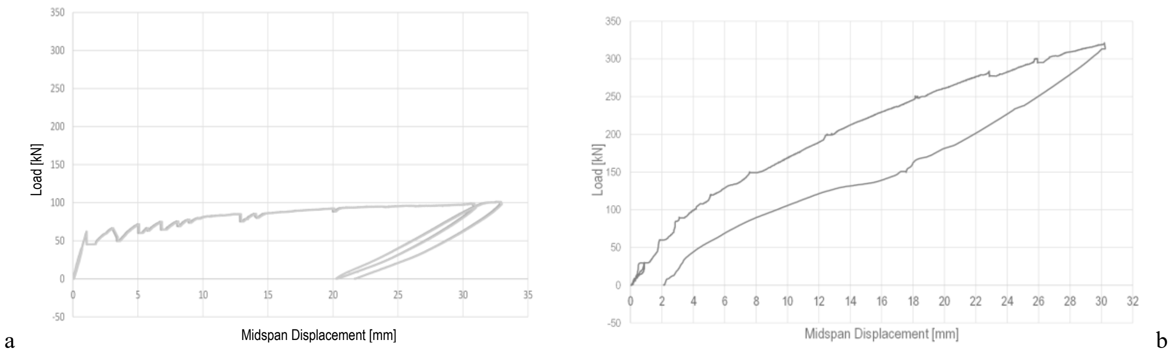


Fig. 2. Load-displacement curves of tests. a) RC beam without prestressing; b) beam strengthened by external prestressing.

The ultimate bending moment of the naked RC beam, computed with unitary partial safety factors (PSF) on material strength γ_M, is M_u = 97.8 kNm, while the ULS moment with code safety factors is M_{ULS} = 82.2 kNm. The ultimate value is confirmed by the bending test on the RC beam (Fig. 2a). When prestressing is added to the beam, the value of axial force at the end of prestressing phase, after sudden losses, is N = 320 kN, for which the ultimate moment, computed with γ_M =1, is $M_{u,p}$ = 182.5 kNm while the ULS value is $M_{ULS,p}$ = 164.8 kNm. Considering that the vertical component of the prestressing is V_p = 74.7 kN, the estimation of the actual bending moment is affected with benefit and the useful moment for external load till failure is $M_{u,L} = M_{u,p} + M_{Vp}$ = 182.5 + 82.2 = 264.7 kNm. The load test shows how the increasing axial stress due to the deflection of the beam, increases the ultimate moment for external load; in fact, for a vertical displacement at midspan of about 30 mm, the axial force in the tendon increases from the initial value of 320 kN to about 450 kN (with this value of axial force the tendon remains in the elastic field soon before achieving its yielding strength). The ultimate moment corresponding to this axial force and due to the external load becomes $M_{u,L} = M_{u,p} + M_{Vp}$ = 195.4+115.5 = 310.9 kNm, which agrees with the maximum value achieved during load test (Fig. 2b). Hence the above values of ultimate bending moment can be assumed for the RC beam with and without prestressing, in the undamaged condition.

3. Fire performance of the single beam

3.1. Fire loads.

The time-temperature curves adopted for fire analysis of the beam are reported in Figure 3. These are the curves provided by Eurocode 1 (CEN, 2002) for standard fire, open areas (external curve) and hydrocarbon fire.

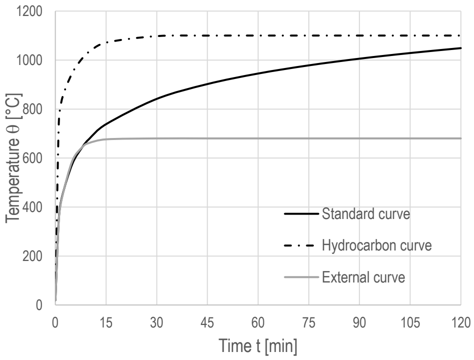


Fig. 3 Time-temperature curves of fire loads.

In this case, only the standard and the hydrocarbon curves will be investigated, given that the external curve is the one with the lowest and slowest increase in temperature and does not provide significant results in terms of safety assessment; hence, it was chosen to operate in two scenarios:

1) fire on the beam from below with standard curve, simulating the persistence of the flame within a confined space below a road overpass.
2) Fire from below due to a tanker accident and then due to hydrocarbons, just below the central span length of the overpass beam.

3.2. Evaluation of the beam performance

The assessment of the performance for the experimental beam is carried out through the degradation due to the temperature in the time of the useful ultimate moment $M_{u,L,}$ reached first by the strengthened beam with the sheathed tendon and due to the relaxation of prestressing, and then by the original reinforced concrete section, without prestressing, when the strengthening is no longer considered effective with the strength degradation of mild reinforcements within concrete cross-section.

In other words, the first assessment is that of the loss of prestressing due to the heating of the steel tendon, taking into account the section of tendon used and the thick and type of protection sheath. Equation 4.27 of Eurocode 3 (CEN, 2005b) was applied where the heating time delay of the protected steel element is provided, by inserting the protection characteristics of the plastic sheath. The standard PE sheath thickness was fixed to 5 mm. The reduction of prestressing was done in a simplified way through a linear correlation with temperatures, matching the thermal elongation to a loss of initial pretension. At each step, the ultimate moment of the beam for the corresponding value of axial stress due to the residual external prestressing was evaluated. Since the zero prestressing value was attained at time t_1, before than the achievement of the isotherm to 400°C of the most exposed reinforcements, occurring at time t_2, in the time range [$t_1 < t < t_2$] the curve shows the behavior of the original beam; for $t > t_2$ the ultimate moment of the original beam begins instead to degrade by reducing of the steel yielding strength f_{yd}, according to Eurocode 3 (CEN, 2005b).

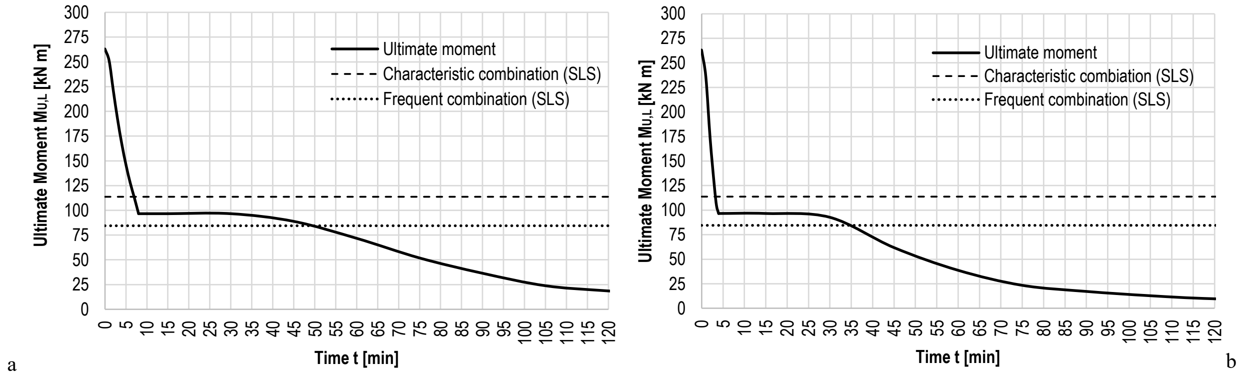


Fig. 4 Results of fire analysis on the beam. Ultimate moment and Service loads with PE sheath 5 mm thick. a) Standard curve; b) hydrocarbon curve

Figure 4a shows the degradation curve for a standard fire while Figure 4b shows the corresponding curve for a hydrocarbon fire. The two load levels shown, intersecting the curves, are those related to a load proportional to the traffic load of the two service combinations: characteristic and frequent. The SLS characteristic load combination is immediately reached by the decay of the performance, leading to the failure of the beam after a few minutes, that is for a sudden damage on the tendon and for a load level equal to that of the characteristic combination. For the frequent combination, which is more realistic in a probabilistic assessment of the structural safety of the bridge in case of a fire scenario, instead, the beam could be able to support the load even in the absence of prestressing and for that load level a longer time could pass before failure.

The hydrocarbon curve shows the worst results with a failure corresponding to the presence of a characteristic value of load after less than 5 minutes while with a failure by frequent combination corresponding to 35 minutes. This confirms the unreliable behavior of the external steel for this type of fire and the reliable behavior of the reinforced concrete section which guarantees a time delay before reaching the critical structural condition.

4. Fire performance of RC bridge

The procedure followed above for the beam of the experimental campaign was repeated for a case-study overpass bridge with comparable properties of the scaled model, to compare the results in terms of fire safety assessment.

Figure 5 shows a typical configuration of an existing bridge strengthened by external prestressing.

Fig. 5 Example of bridges strengthened by external prestressing.

Figure 6 shows instead the geometric properties of the case-study bridge for which the procedure of fire analysis was applied. In this case two tendons of 4 strands with diameter 15 mm are applied for each beam of the deck.

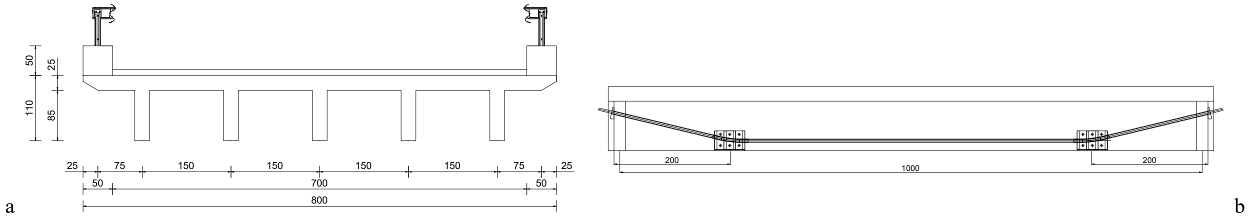


Fig. 6 Properties of case-study bridge

Figure 7 shows the isotherms in the section of the bridge beam for the standard curve at 30 (Figure 7a) and 60 minutes (Figure 7b), and for the hydrocarbon curve, at 30 (Figure 7c) and 60 minutes (Figure 7d).

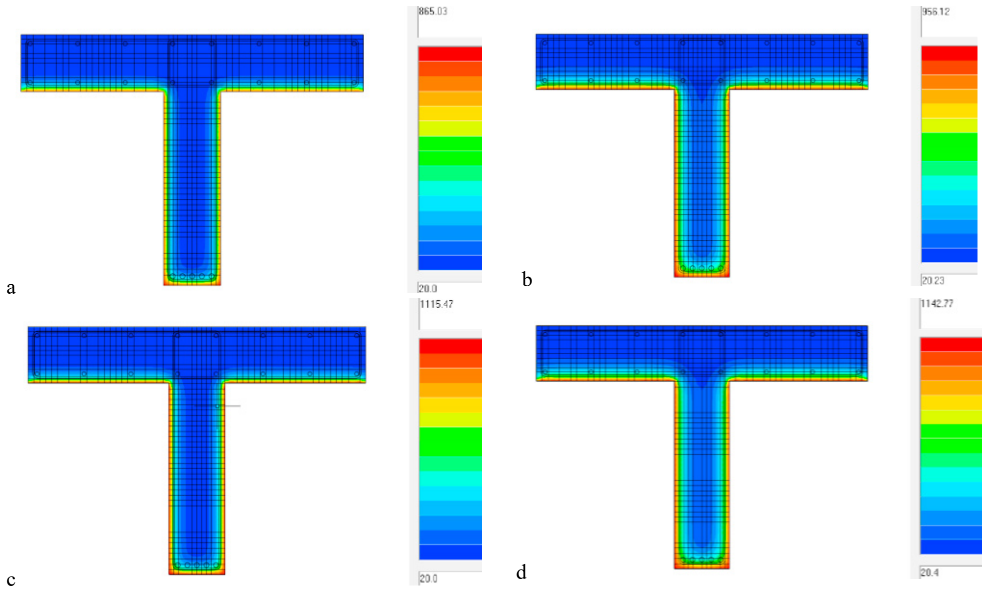
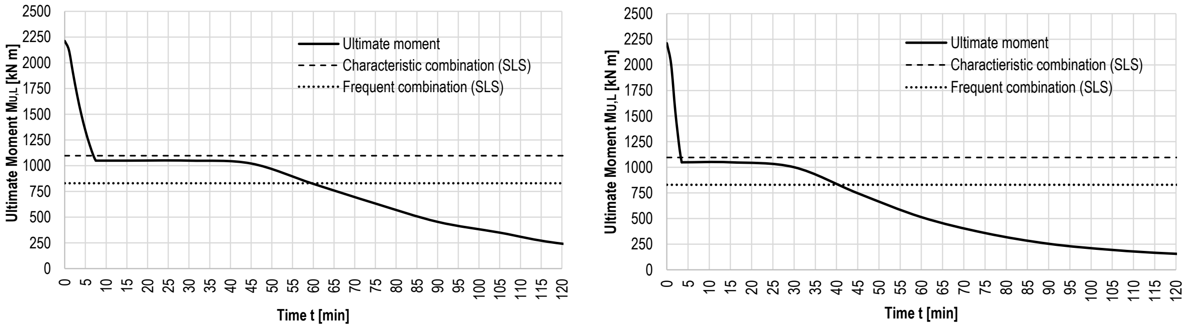


Fig. 7 Isothermal curves for the bridge beam. a) Standard curve at 30 min. b) Standard curve at 60 min. c) Hydrocarbon curve at 30 min. d) Hydrocarbon curve at 60 min.

Figure 8 shows the corresponding diagrams of useful ultimate moment $M_{u,L}$ with time for the RC bridge with external prestressing subjected to the standard and hydrocarbon curves, with a protective sheath of 5 mm for each tendon. The trend for the actual bridge is similar to the one of the experimental beam, considering the difference between the tendon diameter and cross-section properties. The characteristic and frequent combinations of loads are evaluated according to the moving loads defined by Eurocode 1 (CEN, 2005a), transversely distributed to the outermost beam, considered as the one most loaded, through the rule of Courbon-Engesser.

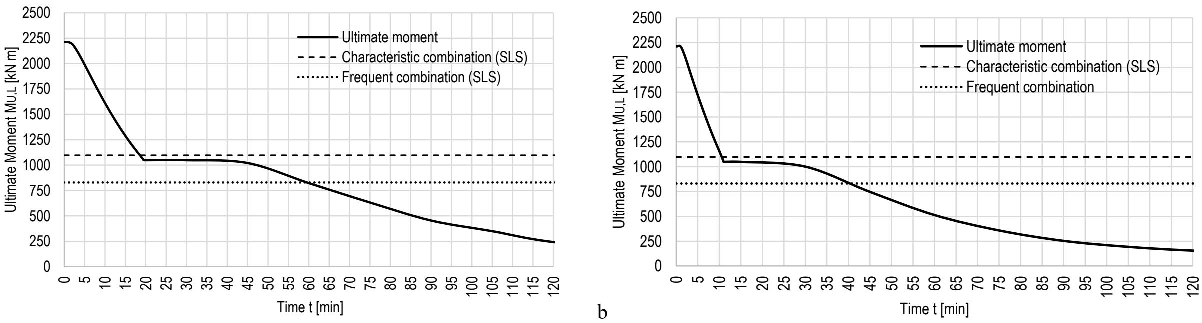
The failure under the characteristic combination of loads is achieved at about 7 minutes from the beginning of the fire event when the standard curve is considered, while less than 5 minutes are sufficient for the hydrocarbon fire. This result does not change between the experimental model and the actual bridge. The curve of the RC section without prestressing, coinciding with the curves represented in the figure starting from the beginning of the horizontal branch, shows instead a better behavior in the actual bridge due to the greater number of bars that are placed at the bottom chord, and are involved in the heat transfer in time. For this reason, the standard curve shows the failure for a frequent combination of loads after about 60 minutes while the experimental beam achieved the same critical point after 50 minutes. For hydrocarbon fire, the actual bridge fails for a frequent combination after 40 minutes while the experimental beam achieved this condition at 35 minutes. It is worth noting that the frequent combination of moving loads is supported by the bridge for a sufficient time to assure safety conditions after the accident and the fire ignition, while the characteristic combination of loads is not supported for a sufficient time for the rescue team arrival on-site and for stopping traffic.



a b

Fig. 8 Results of fire analysis on the bridge. Ultimate moment and Service loads with protective PE sheath 5 mm thick. a) Standard curve; b) hydrocarbon curve

Since the performance for maximum moving loads is determined by the sudden degradation of the properties of the external prestressing tendon and by the sudden prestressing loss, the above evaluations were repeated with a greater thickness of the protective sheath. Figure 9 shows the curves of the ultimate moment for the standard fire curve (Fig. 9a) and the hydrocarbon curve (Fig. 9b) with a PE sheath thickness of 15 mm.



a b

Fig. 9 Results of fire analysis on the bridge. Ultimate moment and Service loads with protective PE sheath 15 mm thick. a) Standard curve; b) hydrocarbon curve

It is evident that the curve of the prestressed structure is, in this case, much smoother and therefore allows a longer time before reaching the critical situation by the characteristic combination of loads: 18 minutes for the standard curve and 10 minutes for the hydrocarbon curve. The values, doubled compared to the previous ones, allow for a minimum reaction time before the damage of the structure can have fatal effects for high levels of traffic loads in the infrastructure. This measure therefore affects the maximum load values but has no effect on the strength of the RC structure, which does not change its behavior in the frequent combination of loads. As a result, to improve the fire performance of the strengthened structure, it is possible to simply equip the external tendons with a more effective protective sheath that is not sized only for the sliding of the tendon and protection against atmospheric agents, as it is common in engineering practice, but it is thickened in order to obtain a higher performance at any unexpected fire events.

Table 1 reports the values of time corresponding to the main events during the fire on the bridge: t_1 time of total prestressing relaxation (zero prestressing), t_2 time for the achievement of the isotherm at 400°C in the mild reinforcement of RC cross-section, $t_{f,char.}$ time of failure with a load corresponding to the characteristic combination, $t_{f,freq.}$ time of failure with a load corresponding to the frequent combination.

Table 1. Times of events during fire on the bridge.

	protective sheath thickness	t_1 [min]	t_2 [min]	$t_{f,char.}$ [min]	$t_{f,freq.}$ [min]
Standard curve	5 mm	8	40	7	60
	15 mm	19	40	18	60
Hydrocarbon curve	5 mm	4	25	3	40
	15 mm	11	25	10	40

5. Conclusions

The fire behavior of existing reinforced concrete bridge decks strengthened by external prestressing, that is by tendons applied externally to the beams, was evaluated. In these conditions the performance, depending on the load level on the bridge, during the fire event, is of fundamental importance to establish the level of structural safety. Load levels lower than those at the Ultimate Limit State, and particularly those corresponding to the Service Limit State combinations, could lead to the failure of the structure, due to the sudden damage of the tendon and to the prestressing loss.

The structural assessment was first carried out through the application of an analytical procedure of fire safety on a single beam of an experimental campaign, for which the post-elastic structural behavior is known, and then the same procedure is repeated on a case study of a typical highway overpass with fire from below. The behavior and degradation in terms of the reduction of the ultimate moment of the beam and of the bridge deck for standard and hydrocarbons fire curves supplied by Eurocode were evaluated. From the results, it can be said that the load level of the characteristic combination can cause critical situations for the strengthened beam, while the frequent combination is kept below the ultimate moment value of the RC beam with good performance and a sufficient range of time for rescue teams and traffic limitation on the infrastructure. The protection of the external prestressing tendons remains, however, a crucial element in the safety assessment in relation to the level of load and residual prestressing force, during and after the fire event.

The presented procedure is simple and easy to use, based on an analytical approach that is very useful for a quick analysis of the problem and for designing protective sheaths of prestressing tendons. A numerical approach can assist in verifying the results on actual bridges, considering fire load curves specifically conceived for the hypothesized scenarios and a more detailed examination of the temperatures reached within the structural elements based on the exposure time and type of fire.

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