Conceptual design of precast industrial structures for fire resistance

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Summary

Precast industrial structures are often prone to fire spread due to the presence of combustible material stored or to specific manufacturing activities associated with fire risk. The typical structural checks for fire action are carried out at sectional level assuming a nominal fire curve. However, this standardised approach does not allow the designer to make an aware performance-oriented structural conception. Indeed, the development of fire and the performance of the precast structure to the exposure to high temperature depend upon several factors, including shape of roof and horizontal members, arrangement of cladding panels, exposure of edge and corner columns, joint geometry and connection system, etc. This paper presents findings from advanced fire safety engineering analysis and field observation on precast industrial structures in fire, providing design hints for a better performance. Design hints to allow for an implosive collapse of such structures are also provided.

1 INTRODUCTION

The vast majority of the industrial buildings in the latin countries, including Italy and Brasil, as well as many others, typically employ precast concrete elements (a prototypal precast industrial building is shown in Fig. 1). These buildings are made for various purposes, including heavy production industry, process industry, exhibition centres, warehouses, logistic hubs, etc. As such, precast concrete is often selected as the best technological solution also accounting for its fire resistance, in buildings with large fire load potential.

Fig. 1 Prototypal precast concrete industrial building

Indeed, precast concrete structures can, if correctly concepted and designed, provide superior performance under fire load with respect to alternative materials and technologies. Nevertheless, the typical methodologies employed to design structures in fire [1] do not allow the designer to carry out a correct conceptual design of such structures. Table methods only focus on the concrete cover, simplified methods such as isotherm500 and zone only focus at the level of the cross-sectional resistance. Even more advanced methods employing more complex numerical tools with stress integration from sophisticated constitutive laws typically focus and are limited to the evaluation of the cross-section resistance. This approach neglects two fundamental concepts to operate a good estimation of the performance of precast industrial structures in fire: indirect actions, and deformations.

These concepts are not decoupled: indirect actions derive both from static undeterminacies (which in precast frame structures are typically limited), and from hampered deformations and subsequent contact between elements. Precast concrete frame structures suffer from a problem of deformability, due to their typical remarkable dimensions [2,3]. Considering the current limitations of ordinary structures, roof elements can span up to 42 m [4], beam elements can span up to 50 m, columns can get up to 20 m of height, panels can span up to 16 m. Under these premises, the effect of thermal curvature induced by an asymmetric exposure of the element to the fire may induce very large deformation, which, if not correctly dealt within the design phase, can cause displacement incompatibility and therefore indirect actions.

Moreover, it is a recurrent finding in advanced fire simulations in industrial environments [5,6,7,8,9] that the nominal standard ISO 834 [1] curve is remarkably on the safe side for what concerns temperature load on concrete structures. This is mainly because of the large volume of the typical compartments. Indeed, the nominal standard fire curve was calibrated on the basis of typical anthropic environments such as apartment or office buildings, which are characterised by a limited plan surface and especially a height which typically does not overcome 4 m, which is much less than a typical industrial environment. The transposition of the nominal standard curve in an industrial compartment was shown to not overestimate the maximum temperatures, but to large overestimate the duration of the peak phase of the fire, limited in the advanced analyses to few dozens of minutes even considering large fire loads. The nominal standard curve would simulate an unrealistically large release of thermal energy.

Under these premises, the present paper illustrates the outcome of the experience of several real fires occurred in precast industrial structures, as well as of advanced numerical analyses, collecting hints for their proper conceptual design in situations of high fire hazard.

2 SHAPE OF ROOF ELEMENTS

Contrarily to intermediate slab elements, roof elements can have very different shapes, since the requisite of flat extrados is not required. Indeed, elements with various cross-sections are spread in the market and produced everyday. The field observation reports that typically ribbed elements perform less satisfactorily than open-section elements such as wing-shaped, as shown by the emblematic pictures of Fig. 2.

A recent research [5] focused on the comparison of the performance of the most common typologies of roof members, including narrow-rib and large-rib double-T, omega-shaped, wing-shaped, and Y-shaped. Elements with the same span and depth, subjected to the same external load, were designed and analytically investigated when exposed to the ISO834 [1] fire. The outcome of this comparison is resumed in Fig. 3, where the residual strength relative to initial at 60 min and 120 min of exposure, as well as the absolute values of concrete and reinforcement volume employed, are compared and plotted as deviations from the mean value. The main reason for the worse performance in fire of ribbed elements is to be attributed to the faster heating of the prestressing lower bulb, exposed on three sides, while the position of the prestressing reinforcement in wing-shaped elements strongly benefits from their vicinity to the outer air, making the temperature in prestressing tendons rise much lower and much slower than in ribbed elements. Moreover, which was not considered in the research [10], the possible failure of narrow ribs as shown in Fig. 2 was not accounted for, due to the complex interaction of self-equilibrated thermally-induced stresses and explosive spalling [11], fostered by the use of high-strength concrete [12], which could make the performance of narrow-ribbed elements in fire even worse. Similar considerations apply to the beams.

Fig. 2 Different performance of ribbed (left) and wing-shaped (right) precast elements in real fires

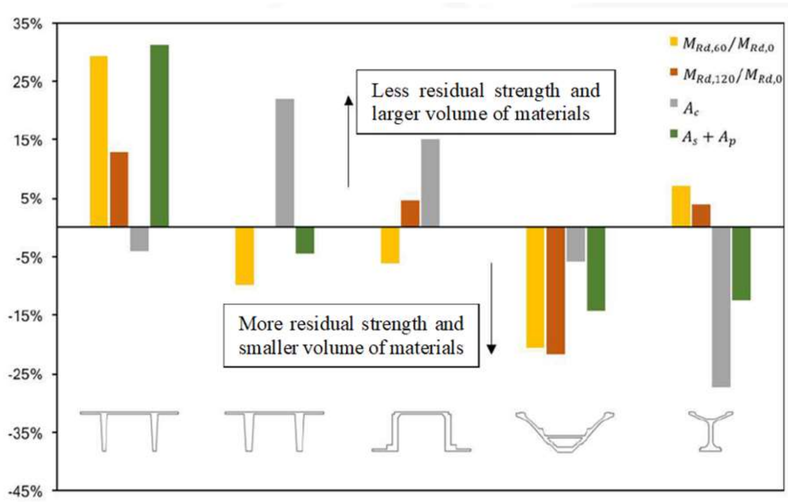

Fig. 3 Comparison of roof elements in fire having different cross-section

3 COLUMNS AND FRAME CONNECTIONS

Concerning the columns, their performance in fire is strongly influenced by their exposure. Typically, there are internal columns exposed on all the four sides, and external columns with three (edge columns with external panels), two (corner columns), or even one (edge columns with infill walls) exposed sides.

When proportioning the columns following the classical sectional approach, the internal columns typically result the most critical under compression (beams and roof elements are simply supported), due to their higher heating rate, typically failing for lateral buckling. Nevertheless, when considering their deformation, internal columns are subjected to unhampered elongation, while asymmetrically exposed columns are subjected to the effect of thermal curvature. Considering a single isolated column cantilevering from the foundation, the heated surface will tend to elongate, making the column bend towards the outside of the building [6]. It is in this context that the loss-of-support of the horizontal members (beams and/or roof members) supported by the column can occur, as shown in the picture of a real fire and as sketched in Fig. 4.

However, this condition can be avoided if a proper mechanical connection is provided between the members. In case the joint is a hinge, rather than a slider, indirect actions arise during the fire for the hampered cantilever deformation. The joint can be proportioned on the basis of capacity design, considering the bending strength of the column base and dividing it by the clear height of the column to get the design shear load for the connection. Hidden connections such as traditional dowels are effective, since they are not directly exposed to the fire.

Fig. 4 Effect of thermal curvature on external columns (Fidenza fire – 2024) and sketch of loss-of-support mechanism for horizontal members

4 CLADDING AND COMPARTMENT PANELS

Both vertical and horizontal cladding/compartment cladding panels are typically made with reinforced concrete outer and inner layers with embedded light insulation blocks. Lightened or thermal-break panels are typically employed, depending on whether the insulation blocks are discrete, leaving full-depth reinforced concrete ribs, or a continuous layer of insulation is also placed beside the outer flat layer, respectively. The performance of such panels resulted indeed not very different from the fire resistance point of view. Moreover, the acting load on the panels during a fire event is basically the self-weight of the panel, only, and thus they are typically designed for fire resistance according to table methods or to simplified calculation methods available in the standards [1], when subjected to very low stress.

Anyhow, the collapse of precast cladding panels was observed in several fires, including the one shown in Fig. 5, which also violates the principle of “implosion” of the building. The issue is once again related with deformation. All standard configurations of modern external cladding panels are simply supported, thus they tend in fire to bend towards the fire itself. The picture in Fig. 5 clearly shows deformations of the order of magnitude of dozens of centimetres, and the collapse of one of the panels induced by the failure of the connection device.

Fig. 5 Cladding panels strongly deformed and collapsed in the Taranto fire (2024)

As previously mentioned, the large deformation of the panels is often not accounted for in design, and thus the design of the joints neglects the possible contact between panel and structure, and the subsequent levering effect which tends to elongate the structural connections, imposing a horizontal displacement which typically leads to the failure of the connection device by displacement incompatibility. Typical design situations are sketched in Fig. 6 for vertical and horizontal panels, respectively.

It is to be reminded that panel deformations may be fully compatible with the connection system, given the connection devices are installed in correspondence of the centre of rotation of the panel, which also means that the metallic connections are in this case exposed to fire, and thus they would need to be protected from the temperature rise to allow for a satisfactory performance.

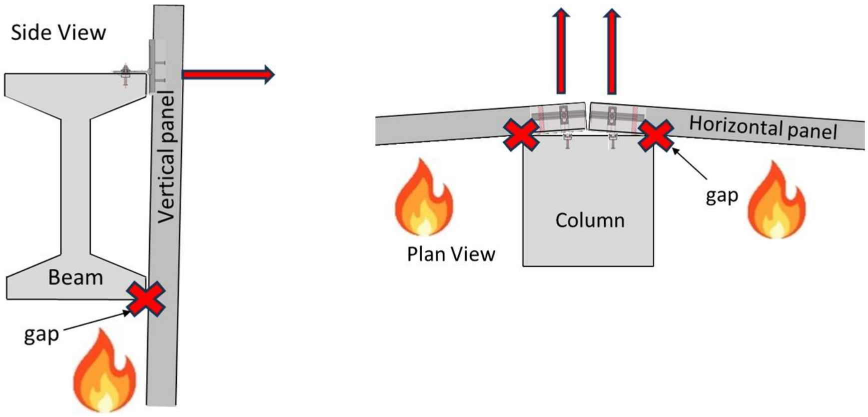

Fig. 6 Physical interference and unintended contact between panel and structure in typical vertical and horizontal panel configurations

Moreover, possible physical interferences may occur for those vertical panels installed in correspondence of columns, as well as for those horizontal panels partially covering the horizontal beam/roof member. In this case, unless the joint between the elements is designed well wider than the typical single centimetre to avoid the contact, it is recommended to install specific narrow vertical panels covering a width slightly larger than that of the column, or horizontal panels almost covering the beam/roof elements only, both sketched in Fig. 7. Typical male/female joints are also to be avoided with the adjacent panels, in order to leave the exposed panels to highly deform, while protecting those panels from the contact with the structural member which could likely cause their early collapse.

Possible alternative methods to avoid the violation of the concept of implosion of the building may concern the specific design of the joints to avoid unintended contact in case of fire, the replacement of typical cladding connection devices [13] with alternative ones providing large out-of-plane displacement capacity [14], or the installation of second-line backup loose tie connections that enter into service after the panels are detached from the structure, as a consequence of the collapse of their ordinary primary connection devices.

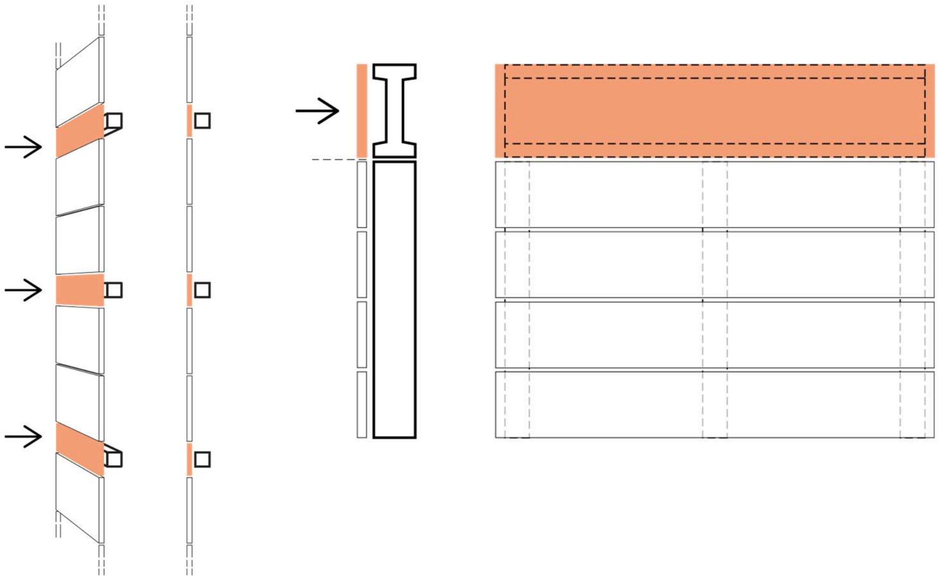

Fig. 7 Special panels to be installed to cover the columns in case of vertical panels, and the beam in case of horizontal panels

5 CONCLUSIONS

Precast concrete is among the technology allowing to build industrial facilities with large fire resistance. Nevertheless, the behaviour of precast concrete elements in fire is rather complex, and the current typical design methods do not account for the large deformability of precast members. This can cause precast concrete buildings to suffer from early failures when subjected to fire. Roof and beam elements can be adopted with various cross-sections, and it was shown that the most advantageous among the most typical concerning their performance in fire is the wing-shaped, where the prestressing reinforcement benefits from the vicinity to the outer air and the issue of the potential explosion of narrow ribs is not present. The roof-to-beam and the beam-to-column connections play a fundamental role in avoiding the loss-of-support mechanism, and they can be rather simply proportioned following capacity design with respect to the bending strength of the column base, which may be subjected to strong indirect bending actions induced by asymmetric expositions, typically of the peripheral columns. Finally, the large deformability of cladding panels may also cause displacement incompatibilities, leading to imposed out-of-plane displacements in the connection device and, consequently, their failure. The envisaged solutions encompass the proper design of elements and joints to avoid unintended contact, the installation of connection devices with large displacement capacity, or second-line beckup loose connections which avoid the panels to fall after they are detached from the structure, avoiding the violation of the implosive collapse criterion.

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References

[1] CEN-EN 1991-1-2. (2002) Eurocode 1: Actions on Structures, Part 1-2: General Actions – Actions on Structures Exposed to Fire. European Committee for Standardization, Brussels, Belgium.

[2] Bosio, M., Di Salvatore, C., Bellotti, D., Capacci, L., Belleri, A., Piccolo, V., Cavalieri, F., Dal Lago, B., Riva, P., Magliulo, G., Nascimbene, R., and Biondini, F. 2023. "Modelling and seismic response analysis of non-residential single-storey existing precast buildings in Italy." *Journal of Earthquake Engineering* 27(4):1047–1068. doi: 10.1080/13632469.2022.2033364.

[3] Capacci, L., and Dal Lago, B. 2024. "Structural modelling and probabilistic seismic assessment of existing long-span precast industrial buildings." *Bulletin of Earthquake Engineering*, paper in print.

[4] Dal Lago, B. 2017. "Experimental and numerical assessment of the service behaviour of an innovative long-span precast roof element." *International Journal of Concrete Structures and Materials* 11(2):261–273. doi:10.1007/s40069-017-0187-6.

[5] Dal Lago, B., Rizzo, F., and Tucci, P. 2024. "Thermo-fluid-dynamic simulation of cellulosic fires in precast industrial buildings with vaults or shed roofs", ACI Italy Chapter: The New Boundaries of Structural Concrete 2024, Rome, Italy, 19th-20th September, 93–98.

[6] Dal Lago, B., Nicora, A., Tucci, P., and Panico, A. 2023. "Precast concrete industrial portal frames subjected to simulated fire." fib Symposium 2023, Ilki, A., Çavunt, D., Çavunt, Y.S. (eds) Building for the Future: Durable, Sustainable, Resilient. fib Symposium 2023, Istanbul, Turkey, 5th-7th June. Lecture Notes in Civil Engineering, vol 350, 13–23. doi: 10.1007/978-3-031-32511-3_2.

[7] Dal Lago, B., and Tucci, P. 2023. "Causes of local collapse of a precast concrete industrial roof after a fire event." *Computers & Concrete* 31(5):371–384. doi: 10.12989/cac.2023.31.5.371.

[8] McGrattan, K., McDermott, R., Hostikka, S., Floyd, J., Vanella, M., Weinschenk, C., and Overholt, K. 2017. "Fire Dynamics Simulator." NIST Special Publication 1019, 6th Edition.

[9] Thunderhead Engineering. 2010. PyroSim: a model construction tool for fire dynamics simulator (FDS). PyroSim User Manual, Rel. 2010.2, Manhattan, USA.

[10] Dal Lago, B., and Mostachetti, M. 2024. "Application of the method of parallel sections (MPS) for the assessment of the thermo-mechanical performance of precast prestressed concrete roof/floor members in fire." *Structures in Fire*, Coimbra, Portugal, 19th-21st June, 359–370.

[11] Lo Monte, F., Felicetti, R., Meda, A., and Bortolussi, A. 2019. “Assessment of concrete sensitivity to fire spalling: A multi-scale experimental approach.” *Construction and Building Materials* 212:476–485.

[12] Bolina, F.L., München, R.M., Dal Lago, B., and Kodur, V. 2024. “A comparative study between ultra-high-performance concrete structures and normal strength concrete structures exposed to fire.” *Structures*, 68:107197. doi: 10.1016/j.istruc.2024.107197.

[13] Dal Lago, B., and Lamperti Tornaghi, M. 2018. “Sliding channel cladding connections for precast structures subjected to earthquake action.” *Bulletin of Earthquake Engineering* 16(11):5621–5646. doi: 10.1007/s10518-018-0410-0.

[14] Dal Lago, B., Biondini, F., and Toniolo, G. 2018. “Experimental investigation on steel W-shaped folded plate dissipative connectors for precast cladding panels.” *Journal of Earthquake Engineering* 22(5):778–800. doi:10.1080/13632469.2016.1264333.