

Full paper

Progressive collapse resistance of composite steel-concrete structures following local damage due to fire

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Abstract

Over the last decades, significant work has been developed worldwide into the field of structural robustness under extreme events. According to EN 1990, the design situations relevant to structural robustness refer to accidental actions, like blast, gas explosion, impact, fire, or localized failure. Although some of the results have been introduced into design practice codes and guidelines, there are still some gaps in knowledge and technical regulations. The study presented in the paper investigated the response of bare steel frames and steel frames with composite floor systems under column loss scenarios in the event of a localized fire. 3D models were calibrated against test data and employed in an extensive study using nonlinear static procedure. Two main parameters were employed, i.e., mechanical properties of beam-to-column joint (full-strength, partial strength) and steel-concrete interaction. The results showed that two-way bare steel systems with ductile behavior may provide alternative load paths and retain integrity in case of a localized fire. Also, special attention may be required for systems with composite floor deck and partial strength joints, due to the possible premature loss of steel-concrete interaction.

Keywords

Steel-concrete composite floor, localized fire, catenary action, tying resistance, joint ductility

1 Introduction

The robustness may be defined as the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause [1]. Over the last decades, intensive research into the structural robustness and progressive collapse resistance has been developed worldwide. Even if with significant delays, some of the results have been introduced into design codes and guidelines. For some structural typologies or hazard scenarios, however, the provisions are still limited or does not deal with specific accidental scenarios. In an extensive review on research and practice on progressive collapse and robustness of building structures [2], significant developments and future needs have been highlighted. Two of these needs are worth mentioning here, namely relevant experimental tests, especially on 3D models that can consider all the mechanisms that contribute to creating alternative load paths, and updated numerical modelling techniques. In a recent technical report issued by the JRC [3], scientific and technical background information related to

the robustness of structures have been summarised. Thus, a comparison between fire design and the design for robustness revealed that some structural requirements associated to a fire design are concomitant with some associated to a design for robustness. However, it has not yet been demonstrated that the level of requirements coincides, i.e., satisfying the requirements for fire design is not necessarily sufficient to ensure an appropriate level of robustness. The same observations can be found also in [4], where it is stressed out that there are substantial similarities between structural fire engineering and design against fire-induced structural (or progressive) collapse, but also some important differences. The progressive collapse resistance is generally checked using Direct Design (the Alternate Path method (i) or Key Element method (ii)), or Indirect Design (example Tie Forces method (iii)). However, if under general progressive collapse conditions adopted in (i), beam-to-beam continuity is assumed to be maintained across a removed column [5], the damage due to localized fire may not be limited to the column, but extended to the beam ends. This means that, even if the

structure has initially the capacity to bridge over the missing column (or columns), the degradation of material due to increase in temperature creates additional failures and may finally lead to the collapse of the structure, partial or total ([6], [7], [8]).

This study investigates the effect of a localised fire on the progressive collapse resistance of two-way steel frame systems, one with composite steel-concrete beams and one with composite steel-concrete beams and composite floor deck. For comparison, a similar bare steel frame system is used as reference. Beam-to-column joint characteristics varied from full-strength to different levels of partial strength. The localised fire is placed just next to the central column and is considered in two consecutive scenarios. First, the resistance of the column is lost, and the element is removed from the model. Second, the temperature at beam ends gradually increases, and the load above the missing column is incremented until failure. The analyses are done using a nonlinear static procedure. The numerical models were calibrated using combined results from column loss tests at ambient temperature and tests on materials and bolted T-stub specimens at ambient and elevated temperatures [9], [10], [11]. Numerical simulations were performed using SAP2000 program [12].

2 Summary of experimental tests on frames, materials and macro-components

2.1 Frame specimens tested under column loss and ambient temperature

The experimental program, carried within the CODEC research project [13], included three tests on two-way frame models and one test on a one-way frame model. All specimens were tested for internal column loss scenario. Because of laboratory restrictions, the models were downscaled, resulting in a 6.0 m by 6.0 m model (see Figure 1). The two-way specimens were made of IPE220 main beams, cruciform columns (two HEB260 with flanges reduced to 130 mm width), and bolted connections with 20 mm thick plates and 5 bolt rows M20, grade 8.8 (joints were designed as rigid and full strength). The BS-FS specimen was constructed and tested as a bare steel frame structure, without a concrete slab. Second specimen, CB-FS, used same steel elements and connections but had a concrete slab of 8 cm thick, reinforced with two layers of 6@100 by 6@100 mm. The third specimen, CD-FS, used same steel elements and connections, but had a composite concrete slab with 5.5 cm thick concrete, and trapezoidal profiled sheets acting as bottom reinforcement. The slab was reinforced with one layer of 6@100 by 6@100 mm at top side. Structural steel was S275 in beams and S355 in columns in all three specimens. For the composite specimens, the concrete was C20/25, the steel reinforcement was S420, and the headed shear studs used S235J2 + C450 steel. More details may be found in [14] and [9]. The testing of the three systems was made so that it can be observed and compared directly the contribution of the floor system and the influence of the steel-concrete interaction in the response to column loss scenario. When compared with two-way specimens, the 2D bare steel specimen (noted as EP-2D) employed same geometry (2 spans of 3.0 m), same beam sections (IPE220), and columns HEB260 with flanges reduced to 130 mm width (a single

section, not cruciform). The main difference was the partial strength bolted connections, made with 16 mm thick plates and 5 bolt rows M16, grade 8.8 (more details are given in [10]). The aim was to determine whether partial strength and semirigid connections can provide the capacity (strength, ductility) required for arresting the progressive collapse after the loss of a column. In all tests, the load was applied statically at the top of the central column, and gradually increased until failure.

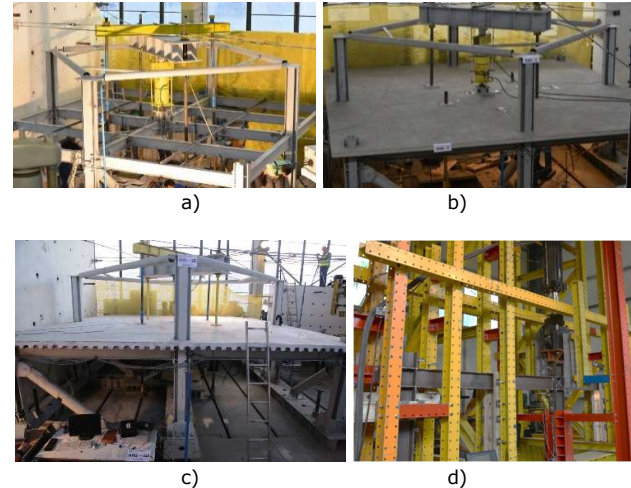


Figure 1 Specimens before testing: (a) bare steel BS-FS; b) composite beams CB-FS; c) composite beams and composite floor deck CD-FS; d) bare steel EP-2D (one-way)

The failure modes and vertical load versus vertical displacement curves are shown in Figure 2 and Figure 3, respectively. The two specimens with concrete slab, CB-FS and CD-FS, have larger initial stiffness and yield strength compared with the bare steel specimen, BS-FS. After the elastic stage, the structures show differences in response. The CB-FS specimen exhibits a significant increase due to composite action of the beams and ultimately fails due to the fracture of two beam ends near the central column.

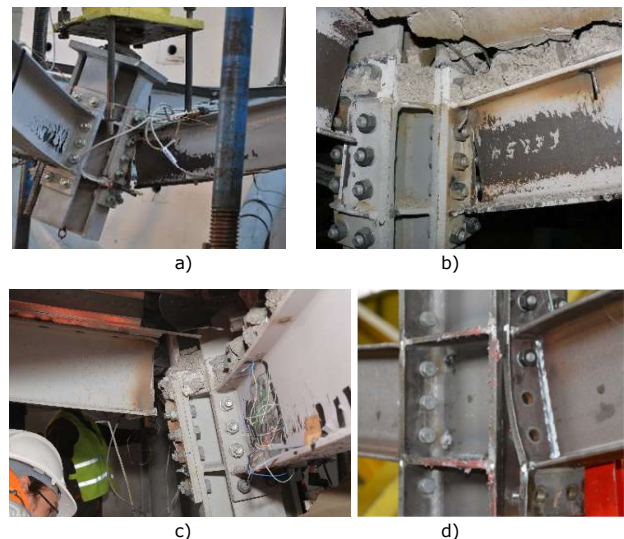


Figure 2 Photos with the failure modes: a) BS-FS; b) CB-FS; c) CD-FS; d) EP-2D

The CD-FS specimen follows initially the same path as CB-FS but shows a premature softening due to the loss of the slab - corrugated sheeting interaction. The resisting force

approaches a plateau, then a sharp decrease due to the fracture of a beam end near the central column. The deformation continues with no increase in capacity, until a second beam end fractures. In case of bare steel specimen BS-FS, a significant catenary force develops, leading to a significant increase in the ultimate force when compared with the yield point. BS-FS specimen exhibits the largest deformation capacity, while the CB-FS exhibits the largest resisting force. The composite slab specimen CD-FS shows a poor behaviour, with a very slight increase in ultimate resisting force compared to yield one. In all three cases, the failure is associated with the rupture of beam ends. The EP-2D specimen showed a moderate rotational capacity, see Figure 3. The progressive fracture of bolt rows near one central beam end ultimately led to the failure of the specimen. Bolts failure limit, however, the ultimate capacity of EP-2D, with no distinctive catenary behavior.

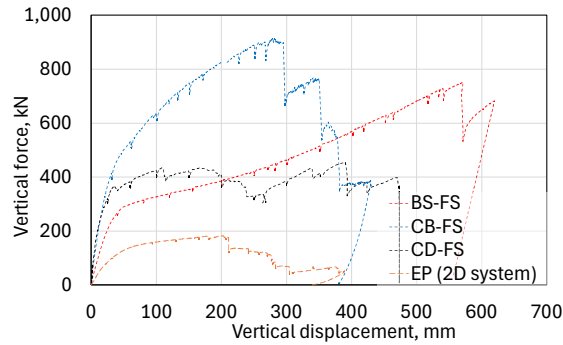


Figure 3 Force - displacement response of frame specimens

2.2 Tests on materials and macro-components (T-stubs) at elevated temperature

The second series of tests comprised monotonic tests on steel materials and bolted T-stub macro-components at ambient temperature and elevated temperature (542° C), respectively. For the present study, only the 8.8 grade M16 bolts, 100 mm bolt distance and three end plate thickness T-stubs (10 mm, 12 mm, and 15 mm) are of interest. More data about test program are given in [11]. Figure 4 shows the force-displacement curves for monotonic tests at the two temperatures. Note that three material specimens were tested at each temperature, but only a single specimen from each configuration. As seen from Figure 4a, the peak strength reduces significantly when the steel is heated at 542° C. Same effect of temperature is also visible in T-stub tests (see Figure 4b), with a very significant reduction in peak strength, but also in ductility due to the bolt fracture. The lower ductility of T-stubs at high temperatures is caused by a reduction in the relative resistance of the bolts compared with the flanges [11].

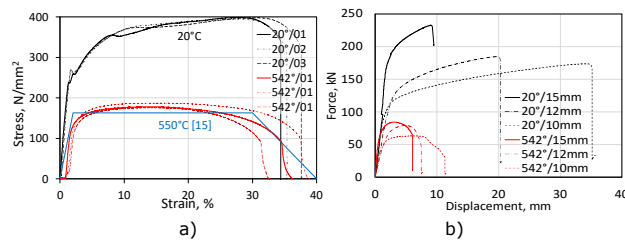


Figure 4 Force-displacement curves at ambient and elevated temperatures for material (a) and T-stubs (b)

3 Parametric study

3.1 Numerical models at ambient temperature

The two-way steel and composite numerical models were first calibrated against column loss tests at ambient temperature, see Figure 5. The material characteristics were defined based on the results obtained from coupon tests. The modelling of nonlinear behaviour for columns was done using concentrated hinge models considering the axial-flexural interaction ($P-M_2-M_3$). For the beams, fibre-type hinge models were adopted. The additional features for composite action (CB-FS, CD-FS) were modelled using the nonlinear layered shell approach for concrete slab, and multilinear-plastic link elements for shear studs (Figure 5b). The numerical analyses were performed using a nonlinear static procedure. The load was applied by pushing down the central column until failure using displacement control. The numerical results show a very good correlation with the experimental results, see Figure 6. All the phenomena that occurred during the test can also be traced on force-displacement curves (elastic and post-elastic behaviour, and failure modes).

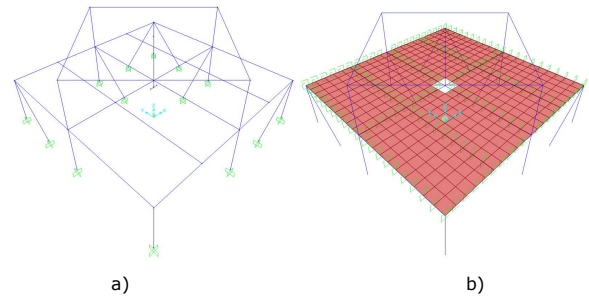


Figure 5 Numerical models: a) BS models; b) CB/CD models

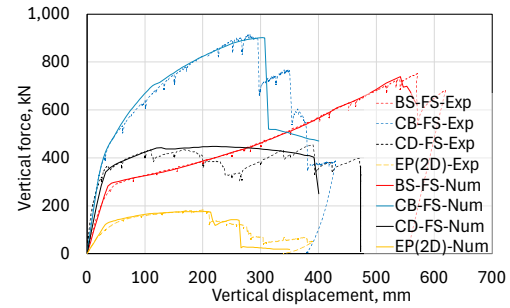


Figure 6 Force-displacement curves, experimental vs numerical

3.2 Parametric study

The parametric study aimed at investigating the response of two-way steel and composite systems under column loss scenarios in the event of a localized fire. Two main parameters were employed, i.e., mechanical properties of beam-to-column joint (full-strength, partial strength) and steel-concrete interaction (bare steel, composite beams, composite floor deck), resulting in 12 different configurations, see Table 1. Note that the frame configurations marked with bold text have been tested experimentally at ambient temperature. For elevated temperature conditions, the steel degradation followed provisions from [15], with the reduction factors defined as follows:

- effective yield strength, relative to yield strength at 20 °C: $k_{y,\theta} = f_{y,\theta} / f_y$ (1)

- proportional limit, relative to yield strength at 20°C:

$$k_{p,\theta} = f_{p,\theta}/f_y \quad (2)$$

- slope of linear elastic range, relative to slope at 20 °C:

$$k_{E,\theta} = E_{a,\theta}/E_a \quad (3)$$

Table 1 Structural parameters of investigated configurations

Name	Joint strength	End plate [mm]	Bolt [mm]	Comp. beam	Comp. deck
BS-FS	Full	Partial	20	-	-
BS-PS1	Partial	16	16	-	-
BS-PS2	Partial	12	16	-	-
BS-PS3		10	16	-	-
CB-FS	Full	Partial	20	+	-
CB-PS1	Partial	16	16	+	-
CB-PS2	Partial	12	16	+	-
CB-PS3		10	16	+	-
CD-FS	Full	Partial	20	+	+
CD-PS1	Partial	16	16	+	+
CD-PS2	Partial	12	16	+	+
CD-PS3		10	16	+	+

As seen from Figure 4a, analytical curve correlates very well with test data. Note that no degradation has been considered for concrete slab, shear studs, and steel deck. Figure 7 to Figure 12 provide push-down curves at ambient and elevated temperatures, and vertical displacement

with increasing temperature for different load levels and for all configurations. Since fire resistance design and critical temperature depend on the load level, for reference, a design load level of DL = 200 kN, which corresponds to the yield limit for BS-PS1, is also indicated on the graphs. It can be seen (Figure 7) that the bare steel system with full-strength joints (BS-FS) continues to provide significant resistance at elevated temperatures after the column is lost, benefiting from the large deformation capacity of the part remained at ambient condition. This resistance reduces dramatically for systems with partially strength joints, which fail very rapidly once the column is lost; the critical temperature for DL reduces from over 1000 °C for BS-FS to 550 °C for BS-PS2 (see Figure 8). At same temperature of 550 °C, the weakest configuration, BS-PS3, can sustain less than half of DL. Thanks to the two-way action in the slab, the systems with composite beams (CB) provide much greater fire resistance (see Figure 9), with the critical temperature for DL over 1000 °C for all four cases (see Figure 10). A less beneficial contribution of concrete slab has been observed for composite deck system (CD), with lower peak forces due to the premature loss of interaction between steel deck and concrete slab (see Figure 11, Figure 12).

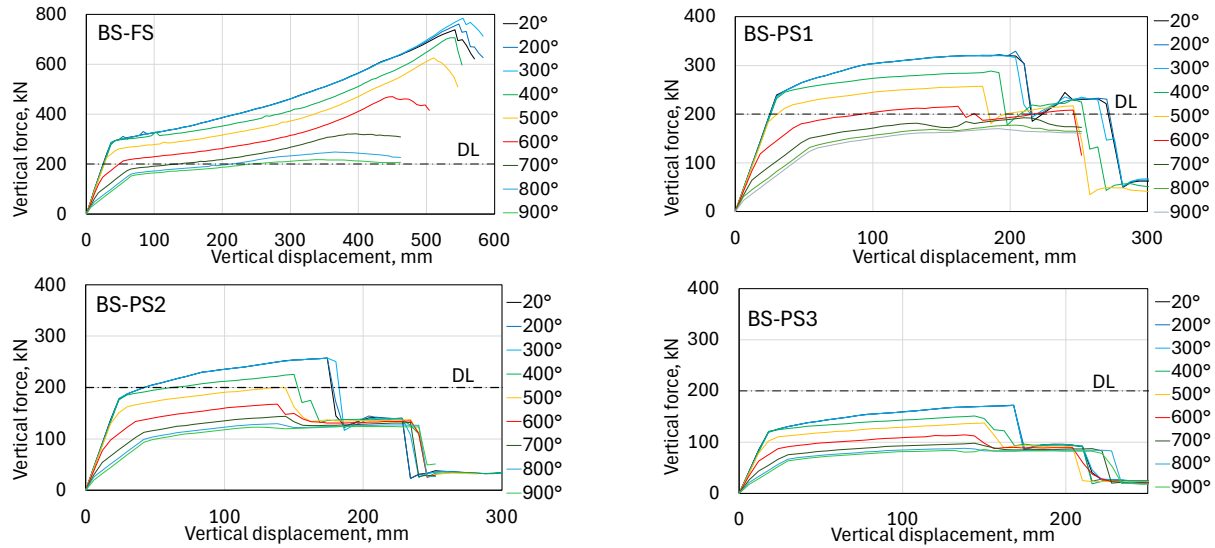


Figure 7 Force-displacement curves in push down tests for BS frames

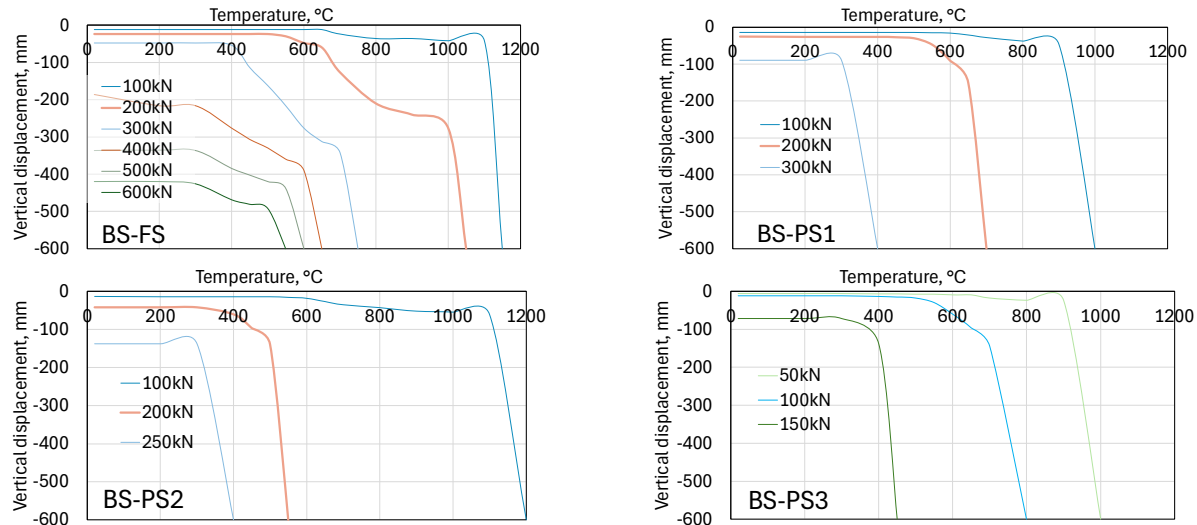


Figure 8 Displacement vs temperature curves for BS frames

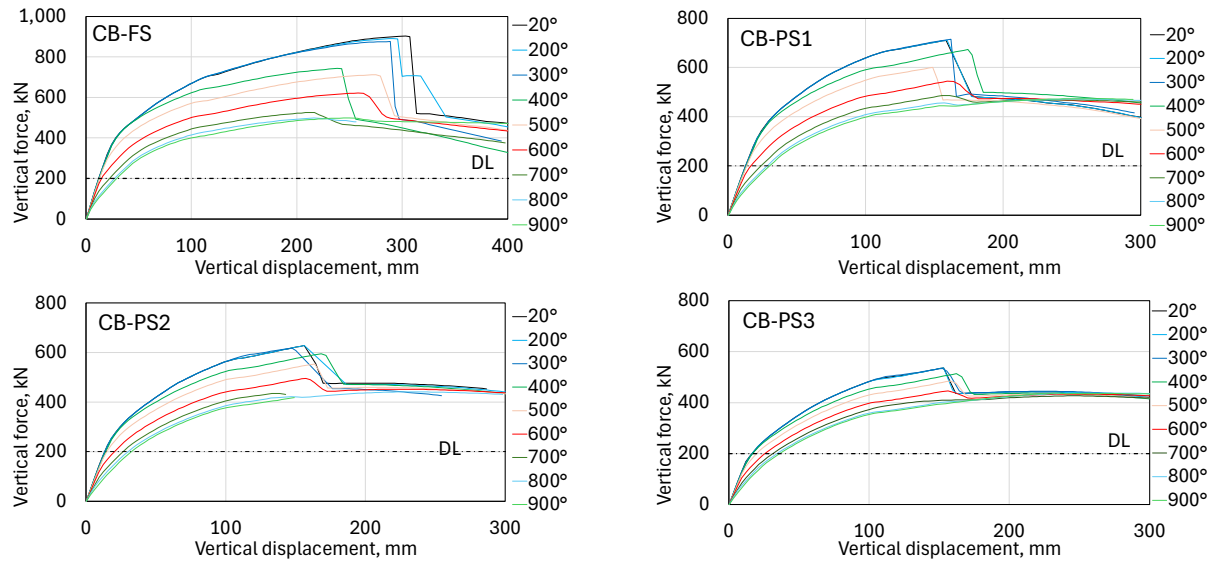


Figure 9 Force-displacement curves in push down tests for frames with composite beams

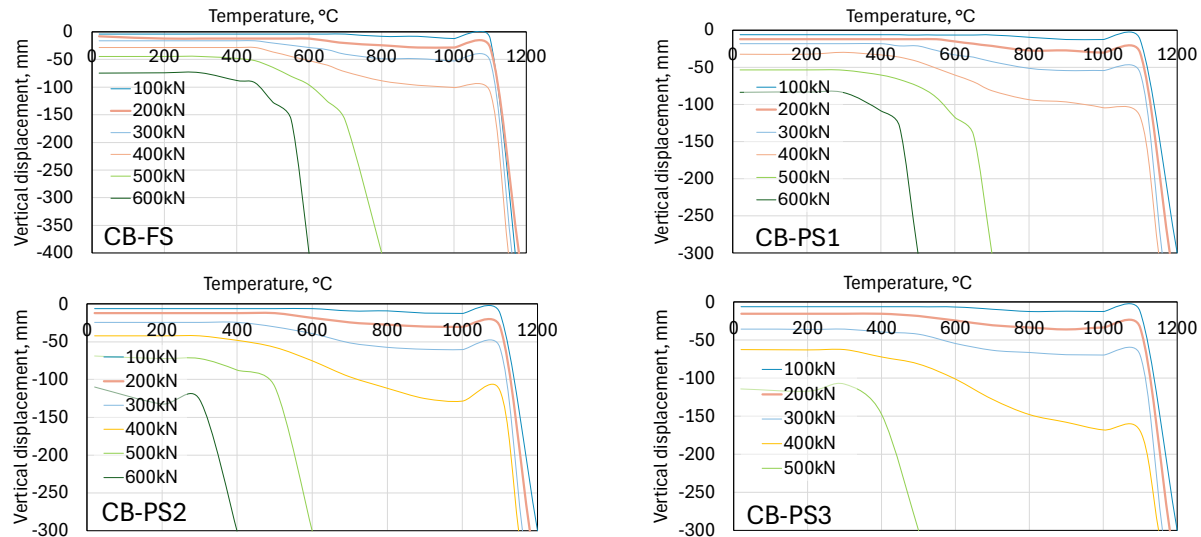


Figure 10 Displacement vs temperature curves for frames with composite beams

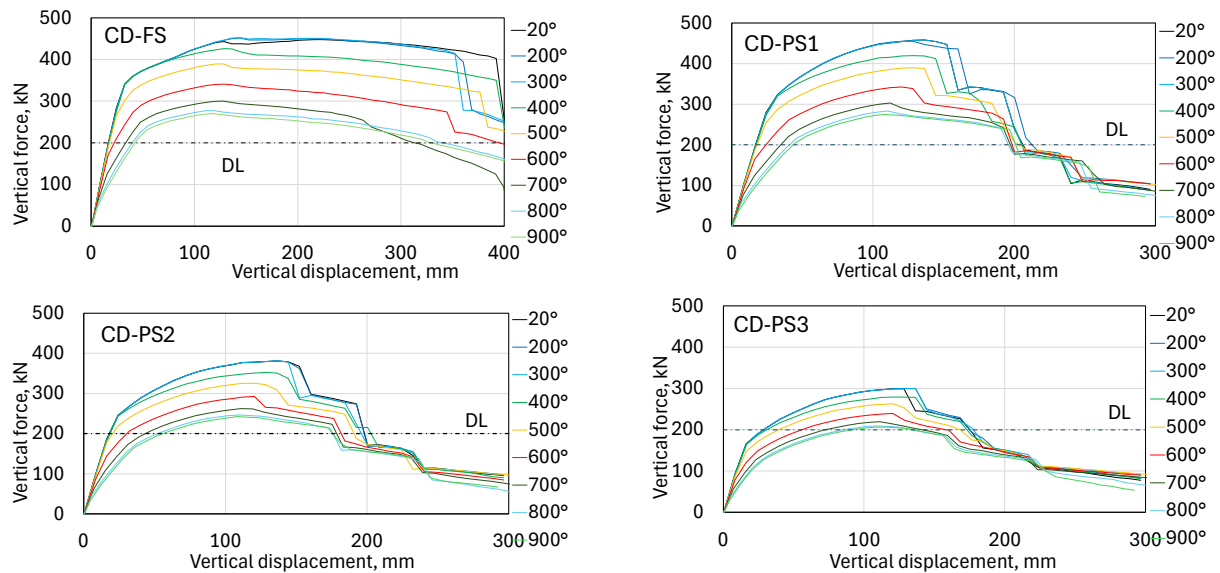


Figure 11 Force-displacement curves in push down tests for frames with composite deck

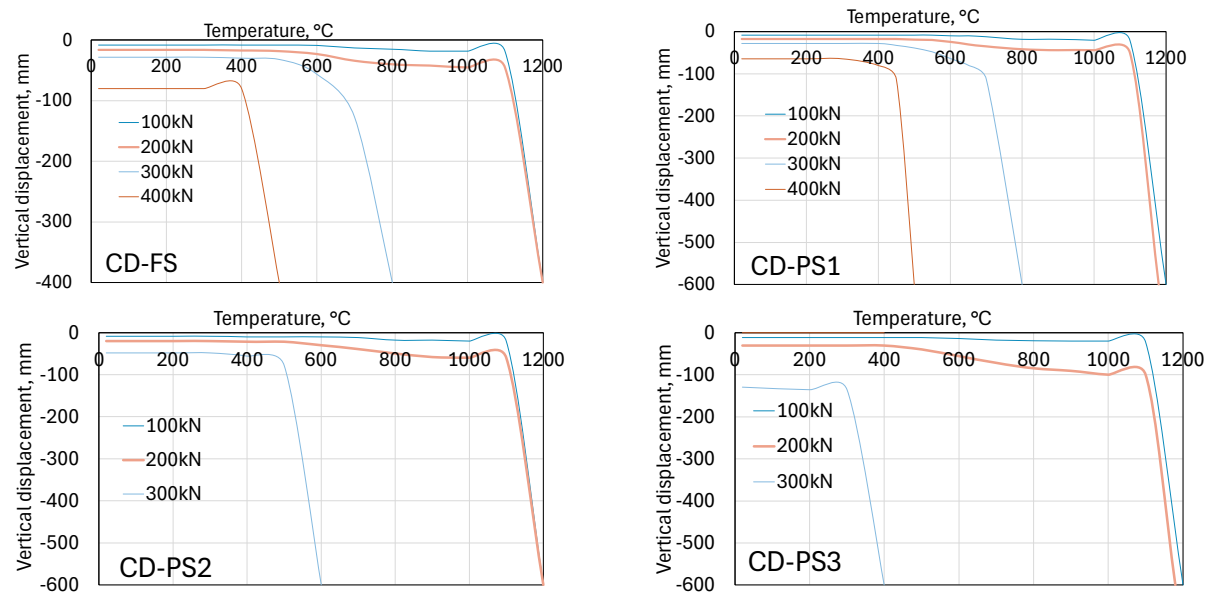


Figure 12 Displacement vs temperature curves for frames with composite deck

4 Conclusions

Progressive collapse resistance following a column loss generally indicates sufficient structural robustness. However, in the event of a localised fire, the degradation can propagate further than the column, affecting at least beam ends and beam-to-column connections. As a result, even the structure remains stable after the initial column removal, the damage may propagate and turn into a progressive collapse. Bare steel systems with ductile behaviour, two-way redundancy, and sufficient overstrength in the connections may provide alternative load paths and retain integrity in the event of a localised fire. Weaker connections dramatically reduce the critical temperature and inevitably the collapse. Composite steel-concrete beam systems can provide higher levels of robustness, even when connections are partial strength. However, special attention may be required for composite floor deck systems and partial strength joints, due to the possible premature loss of steel-concrete interaction.

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