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# Stability of Steel Structures at Elevated Temperatures: A Hybrid Fire Testing Approach

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**Abstract**
The response of structural systems to fire loads is typically assessed through performing 'standard' tests under constant mechanical boundary conditions. Such tests are usually performed on the individual elements. Full scale tests showed differences in their behavior compared to the behavior of the individual members tested in standard conditions. Full-scale tests of structural systems remain impractical due to physical, economical, and time constraints. The goal remains to have the capability to predict the behavior of a full-scale test though experimentally testing individual structural members. A promising approach to that problem is "Hybrid Fire Testing (HFT)" where a subset of the structural system (Physical Substructure PS), is physically tested, while the remaining structure (Numerical Substructure NS), is simultaneously numerically analyzed. PS represent the parts of the structure with higher behavioral uncertainty, while the NS represent the parts which can be numerically modelled with high confidence. During the test, the mechanical boundary conditions on the PS and NS are continuously updated. Certain challenges are unique to HFT due to the continuous fire exposure which induces continuous thermal expansions. This paper describes a recent virtual HFT study performed on a ten-story multi-span steel frame assembly, with the focus on the structural stability and interface equilibrium and compatibility between the substructures. In the early stage of this work, the HFT study is done in a virtual environment, where the PS is also modeled numerically as a proof of concept. As part of the study, a traveling fire analysis was performed on the building and results highlighted the importance of considering the performance of the structure as a whole assembly, and showed that individual member standard testing could be unsafe. The paper also describes some of the challenges that are unique to the HFT, and ways to overcome them.

## 1. Introduction
The structural engineering industry is rapidly changing. Structures with unique functionalities and scales are being designed, and new materials and technologies are constantly emerging. This is leading the industry into transitioning towards performance-based design methodologies, which require structural engineers to assess the performance of such structures, especially under

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extreme loading conditions. Such structures and materials require testing under these extreme loads in order to validate and calibrate their design modeling approaches.

Fire tests are performed to understand the behavior of structures exposed to fire conditions. Generally, the fire tests are performed on individual elements, with no restraint, neglecting the restraining effect of the rest of the structure, e.g. Tondini et al. (2013). Entire structures have been tested as presented by Lennon (2003), Newman et al. (2000), Armer et al. (1994), but the high cost of such tests limits the ability to conduct them. Full-scale tests of structural systems remain impractical due to physical, economical, and time constraints. However, the goal remains to have the capability to predict the behavior of a full-scale structure though experimentally testing individual structural members. Maintaining the practicality of testing only parts of the structure, but at the same time considering the overall global behavior of the structure as a whole system, Hybrid Fire Testing (HFT) methodology appears to be a promising approach.

Hybrid testing in general is an approach based on a sub-structuring procedure, where a subset of the structural system (Physical Substructure (PS)), is physically tested, while the remaining structure (Numerical Substructure (NS)), is simultaneously numerically analyzed. PS represents the parts of the structure with higher behavioral uncertainty, while the NS represents the parts which can be numerically modelled with high confidence. During the test, the mechanical boundary conditions on the PS and NS are continuously updated.

Hybrid testing can potentially be applied under different loading conditions such as wind, blast, impact, waves, fire, traffic and seismic events. Although it can potentially be applied for such a variety of loading conditions, it has only been widely implemented in the seismic field since early 1970s. The implementation of the method developed from the seismic field to the fire field remains a challenge and only a few HFT have ever been performed.

Hybrid testing under fire conditions in particular has proven to be necessary for many reasons. Globally, the thermal expansion of the structural elements exposed to fire induces major forces that affect the structural behavior during the fire test. The forces are dependent on the boundary conditions adopted in the test. Locally, specific phenomenon occurs for some materials when exposed to fire such as the spalling of the concrete. This phenomenon is very challenging to be numerically modeled. Therefore, fire tests in the realistic conditions are required to better understand the overall structural behavior, and HFT has the potential to provide insight into behavior of structures exposed to fire, under their real boundary conditions.

Usually, standard fire exposure (ASTM E119-16a (2016)) is adopted when studying structural behavior under fire loads. More research is currently focused on studying the behavior of structures exposed to natural fires (Sauca et al. 2015). Zhang et al. (2015, 2013a, 2013b) investigated the behavior of a steel beam under natural fire exposure. These studies showed that under the effect of temperature gradient, the behavior of steel members when exposed to localized fires may be very different from when exposed to the standard fire. Moreover, the failure temperature of steel members exposed to localized fires may be much lower than when exposed to the standard fire. Rackauskaite et al. (2017a, 2017b) presented the differences in the behavior of a multi-story steel frame exposed to travelling fires versus the traditional design fires (standard fires). Results indicated that both travelling fires and Eurocode fires need to be considered to ensure a safe building design. Different structural responses for the same structure might be triggered depending on the type of the considered fire exposure. In case of uniform fires, higher axial forces were observed compared with the case of traveling fires, where larger displacements at different times and locations were observed. The study also showed that the highest stresses developed in the fire floors adjacent to the unheated floors while in the intermediate fire floors, the stresses were significantly lower. Peak compressive and tensile axial forces developed in the fire floors adjacent to the cooler floors, and in the cooler floors, respectively. This shows that considering natural (realistic) fire scenarios when analyzing the behavior of structures is necessary for safe design. The study also highlighted the effect of the unheated part of the structure on the performance of the remaining structure exposed to fire.

Therefore, in order to predict the real behavior of the tested structural member under specific conditions, the effect of the remaining structure should be considered, which in turn could be exposed to fire. Thus, hybrid fire testing is a promising approach to reach this objective, i.e. perform tests on individual structural members while simultaneously considering the effect of the surrounding members. However, despite the large amount of existing information related to hybrid testing in other fields, the application in the fire field is not straightforward, and has its unique challenges. This is due to the main characteristics of HFT which require the tests to be performed in real time. The effect of the fire on the materials is immediate and continuous and the thermal expansion requires modification of the interface boundary conditions in real time, as discussed in the following section.

## 2. Background and specific HFT challenges

The main concept of hybrid simulation is to combine the advantages of individual member testing and full-scale testing, meaning that an individual member(s) or sub-structure will be tested while accounting for the remaining structure's contribution. The surrounding can be modelled aside, via nonlinear numerical software or a predetermined matrix, if elastic behavior of the surrounding is expected (see Sauca 2017) for a detailed discussion). The tested substructure is referred to as the physical substructure (PS), while the surrounding (remaining) structure is referred to as the numerical substructure (NS). The approach ensures equilibrium and compatibility between the PS and the NS over the duration of the test. At each time step, data (displacements or forces) are measured at the substructure interfaces. Due to the fire exposure of the PS, equilibrium and compatibility at the interface are generally no longer satisfied at the end of the time step. To restore the equilibrium and compatibility at the interface, new data are computed (forces or displacements) and are imposed to the substructures, based on the measured data from the previous time step. At frequent intervals (time step $\Delta t$), the displacements or the forces at the interface are measured from the PS and computed for the NS. There may be an additional delay of time, $\Delta t_P$, required for the calculation of the NS reaction and for application of the reaction to the PS. The procedure is either called Force Control Procedure (FCP) or Displacement Control Procedure (DCP), when reaction forces or displacements are sent back to the PS.

A specific challenge of the HFT is the continuous fire exposure of the PS (and the NS in some cases). The consequence of the continuous fire exposure is the continuous change of the registered displacements and reaction forces at the boundaries (interface) of the PS and the NS. Therefore, to ensure equilibrium and compatibility between the two substructures, HFT must be performed in real-time. This is unlike the seismic field (where hybrid testing evolved), in which slow tests, fast tests, real time tests, and smart shaking table tests, are all possible, only requiring a specific algorithm to solve the dynamic equation for each case.

Very few HFT are reported in the literature. Kiel et al. (1989) performed the first HFT, but the publications are not publicly available. The first reported HFT was performed by Korzen et al. (1999), where a column specimen was experimentally tested as part of a simulated building environment. The mode of action between both parts was exemplified on a single degree-of-freedom (DoF) basis, i.e. the axial column force is measured and adjusted continuously to the model force, which is represented through a – not necessarily constant – stiffness, in displacement control. Another hybrid fire test was performed by Robert et al. (2009, 2010). In the latter, the PS consisted of a concrete slab whereas the NS was a surrounding one floor concrete building. One axial DoF and two rotational DoFs were controlled. A force-controlled procedure was employed. The behavior of the NS was modelled by a constant predetermined matrix, which had been calculated before the test. Mostafaei (2013a, 2013b) presented the hybrid fire test of the first-floor central column part of a 3D concrete frame. The column was tested in a furnace (PS) while the surrounding was numerically modelled in the non-linear finite element software SAFIR® (Franssen 2005, Franssen and Gernay 2017). For each time step, the interaction between the PS and NS was manually enforced, and axial force in the column was controlled. Part of the NS was also exposed to fire. Whyte et al. (2016) presented a HFT performed on a small-scale steel coupon. The objective was to extend the mechanical hybrid simulation of OpenFresco (2016) and OpenSees (2016) (software commonly used for seismic hybrid simulation) by introducing the temperature DoF and loads. The NS was modeled in OpenSees while OpenFresco was the framework used for the experimental setup and control. Tondini et al. (2016) presented a static solver for HFT, a method based on the finite element tearing and interconnecting (FETI) algorithm. The validation of the method was done in a numerical environment considering a moment-resisting steel frame. Schulthess et al. (2016) present a hybrid fire test performed on a small-scale specimen, i.e. steel coupon specimen, tested in a universal testing machine inside an electric furnace. The NS was analyzed in ABAQUS (2011). Therefore, the only hybrid fire tests performed on full size members (of which the authors are aware) remain the tests reported by Korzen (1999), Robert (2009, 2010) and Mostafaei (2013a, 2013b) and the same methodology was used to perform each of those three tests. In this paper, the method used in those tests will be referred to as the "first generation method".

Stability issues have been observed in the former methodology, i.e. first generation method, as presented by Sauca et al. (2016a) meaning that the methodology was only stable for certain ratios between the NS stiffness and PS stiffness. A new method has been proposed by Sauca et al. (2016b, 2017a, 2017b) and Sauca (2017) which is stable independently from the stiffness ratio between the NS and PS. The stability of the method was numerically proven and validated.

## 3. A novel HFT approach

In the HFT presented by Korzen (1999), Robert (2009, 2010) and Mostafaei (2013a, 2013b), data (displacements or forces) were measured from the PS and sent to the NS at every time step $\Delta t$. The reactions (forces or displacements) of the NS at the interface were then calculated considering only the characteristics of the NS (i.e. disregarding the characteristics of the PS). This calculation can be done using a numerical model or a predetermined matrix for the NS

Sauca (2017). Then, the reactions are sent back to the PS and applied at the interface (boundary) to restore equilibrium and compatibility. There may be an additional delay of time $\Delta t_p$ requested for the calculation of the NS reactions and for the application of the reactions to the PS. When reaction forces or displacements are sent back to the PS, the procedure is either called FCP or DCP, respectively. Sauca et al. (2016a, 2017b) showed that the later methodology was not always stable. The stiffness ratio between the NS and PS dictated the type of procedure to be used and stability was ensured only for some values of this stiffness ratio. The instability was caused because the stiffness of the PS was not measured during the test and thus was not considered in the calculation of every time step's boundary conditions.

Thus, in the novel method (Sauca et al. 2016b, 2017a, 2017b), the stiffness of the PS is considered in the calculation process and thereby the stability of the solution is ensured regardless of the stiffness ratio between the NS and PS.

A step by step summary of the method for the DCP case is presented below (see Sauca et al. (2017b) for a detailed description).

a. The interface (boundary) forces and displacements are determined before the start of the test from the analysis of the entire structure.
b. The PS is placed in the heating condition (direct fire or furnace), and loaded with the exterior loads and interface displacements, while the NS is numerically modeled aside.
c. Heating of the PS starts. The interface displacements of the PS are blocked for the duration of a time step (DCP) and the reaction forces are measured.
d. Meanwhile, the corresponding displacements are blocked at the interface of the NS and the reaction forces of the NS are computed.
e. The measured reaction forces of the PS are compared with the computed reaction forces of the NS. Generally, the equilibrium is not ensured due to the fire effect. The same displacements are applied at the interface of the PS and NS, thus the compatibility is enforced.
f. To restore the equilibrium, a correction of displacement vector $d\mathbf{u}$ is calculated and applied at the interface of the PS and NS. The calculation includes the vector of out-of-balance forces $d\mathbf{F}$ (from step *e*). In this step, the stiffnesses of both the PS and the NS are accounted for, according to Eq. (1). This is the main difference from the first generation method (in which only the stiffness of the NS was considered) and the most important contribution that allows ensuring stability of the method.

$$d\mathbf{u}\ (t_n) = (\mathbf{K}_N+\mathbf{K}_P)^{-1}\ \mathrm{d}\mathbf{F}\ (t_n) \tag{1}$$

g. The new calculated displacements are imposed on both the PS and NS. Some time is required to perform the above computations and to adjust the new displacements in the actuators representing the structural interface.
h. The new imposed displacements will generate new reaction forces in the PS and NS.

Steps *e-g* are repeated until the end of the fire test. As shown in Eq. (1), the stiffness of the PS is used for the correction of displacements. As this stiffness is generally unknown during the HFT, a constant value can be considered and several iterations would normally be needed at each time step to converge to the correct solution. In a fire test, the evolution of temperatures in the PS cannot be held constant during the period requested to perform the iterations at every time step. During the time needed to perform the calculations and for the testing equipment to apply the corrections of displacements, the temperatures are still changing, which in turn changes the stiffness of the PS, the restraint forces, etc. Hence, the convergence process tends to achieve an equilibrium that is constantly changing. As a result, it is not relevant to distinguish between iterations and time steps. Instead, the test can be performed by continuously applying Eq. (1), with a cycling frequency that is as high as possible, which requires computing techniques and testing equipment that has a short response time. Note that the compatibility is continuously achieved, as the same displacements are imposed both on the PS and on the NS at their interface. The purpose of the methodology is thus to constantly adapt these displacements to satisfy equilibrium between the substructures throughout the entire test duration.

## 4. Analysis

This section describes a virtual HFT study performed on a ten-story multi-span steel frame assembly, with the focus on the structural stability and interface equilibrium and compatibility between the substructures. The HFT study is done in a virtual environment, where the PS is also modeled numerically as a proof of concept. As presented in the previous sections, the HFT found in the literature were performed using the standard fire exposure. This study focuses on applicability of the HFT to real fire scenario exposure cases. Therefore, in the first part of the study, a traveling fire analysis was performed on the ten-story building.

### *4.1. Fire-structural Analysis*

#### *4.1.1 Prototype building*

A 2D analysis was performed on a prototype structure that is a 10-story moment resisting steel frame (Fig. 1), which has been designed in accordance to the American Society of Civil Engineers (ASCE 7-02) standard by Sadek et al. (2010). The floor is 45.5 m x 30.5 m (5 bays in the longitudinal and transversal directions). The ground floor columns are 5.3 m high while the rest of the columns are 4.2 m high. The lightweight concrete floor slab is supported by the steel beams. Fig. 1 shows the details of the building. It is noted that this building was studied under natural fire conditions by Rackauskaite et al. (2017a). The composite action between the beams and concrete floor is neglected in the 2D analysis. It was shown in the literature that the tensile membrane action has a beneficial effect to the structural response during fire (Bisby et al., 2013), which if neglected will result in slightly more conservative results.

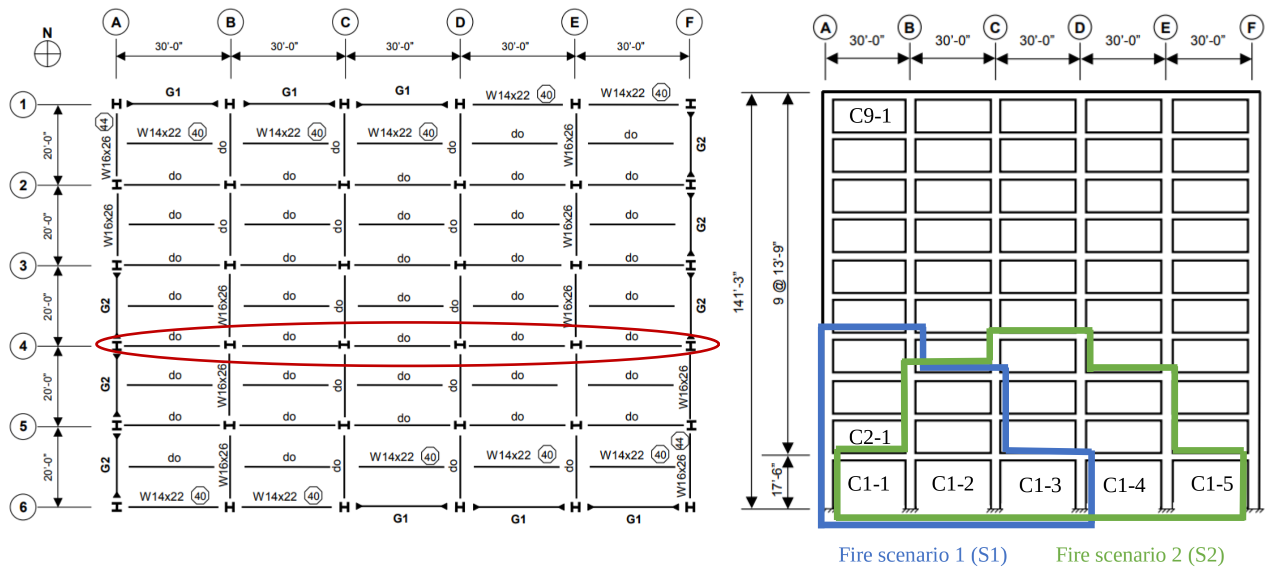



Figure 1. Plan layout and elevation of the steel frame building (Sadek et al., 2010)

Beam sections of the 2D frame (highlighted in Fig. 1) were W14x22 for all the floors. Column sections of the 2D frame are W18x119, W18x97, and W18x55, for floors 0 through 3, floors 4 through 6, and floors 7 through 9, respectively. ASTM A992 structural steel with yield strength ($F_y$) of 344.8 MPA is used for all the columns and beams. The dead load on the floor beams and roof beams was 3.64 kN/m$^2$, and 2.68 kN/m$^2$, respectively. Their corresponding live load was 4.79 N/m$^2$ and 0.96 kN/m$^2$. The considered load combination is consistent with equation A-4-1 of Appendix 4 presented in the American National Standard ANSI/AISC 360-16. The compartments are labeled as Ci-j, where "i" refers to the floor number and "j" to the compartment number.

### *4.1.2 Fire scenarios*

Two fire scenarios were considered in this case study. Figure 2 shows: (a) the heat release rate curve for each compartment fire; (b) simulated fire curves for fire scenario 1; (c) CFAST model for fire scenario 1; (d) CFAST model for fire scenario 2. CFAST (Peacock (2017)) is a two-zone fire model capable of predicting the environment in a multi-compartment structure subjected to a fire. 1D heat transfer FE model is used to determine fire spreading.

Fire scenario 1 initiates in a corner compartment and Fire scenario 2 starts in a middle compartment. For each compartment, one-zone model (which assumes uniform gas properties in a fire compartment) was used to simulate the fire environment. Fire spread from one compartment to an adjacent compartment occurs when the temperature of the unexposed surface of the connection wall, ceiling, or floor reaches a critical temperature of 139 $^{o}$C (as per ASTM E119-16a (2016)). Fire in a compartment ignites immediately when temperature of the inner surface of any of the compartment boundaries (walls, ceiling, and floor) reaches 139 $^{o}$C.

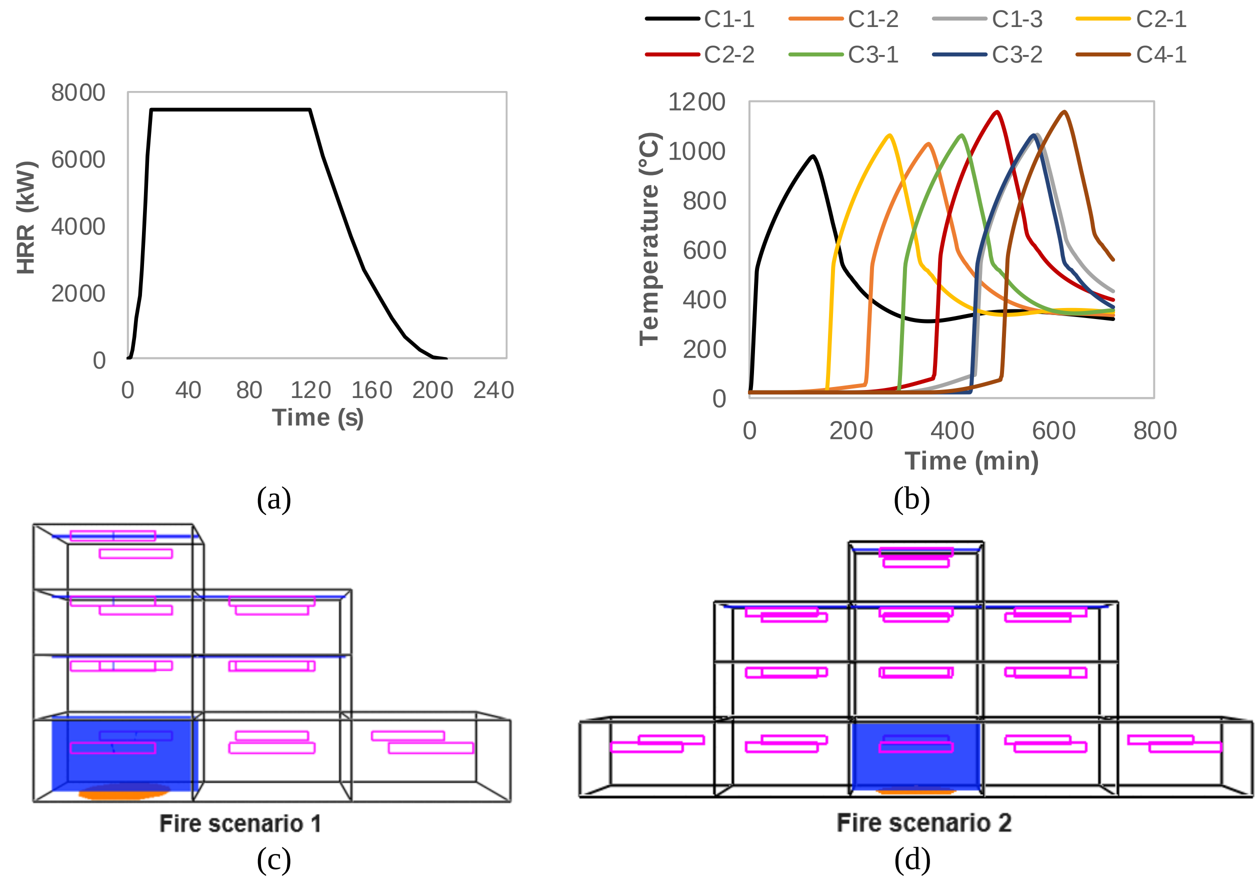



Figure 2. Fire scenarios considered in this study

### *4.1.3 SAFIR model*

Analysis of the building exposed to fire scenarios 1 and 2 was carried in SAFIR®. To determine the evolution of temperature in the structural members, the thermal analysis was performed prior the structural analysis and then used in the latter.

*Thermal analysis of the structural members*

Table 601 in the International Building Code (IBC (2015)) specifies a fire resistance rating of 3 hours for the Type IA primary structural frame analyzed here. The fire rating of the floor construction and associated secondary members is 2 hours as presented in the same table. Steel insulation properties are considered as for the high-density perlite (Buchanan, (2001)) with the following characteristics: thermal conductivity of 0.12 W/mK, density of 550 kg/m$^3$ and specific heat of 1200 J/kgK. The thickness of the insulation (for columns and beams) was determined in the FEM so the temperature of the steel members does not exceed the accepted critical temperature for steel of 538°C (as per ASTM E119-16a (2016)).

The thermal analysis of beams and columns was performed in SAFIR®, with the following characteristics of steel: heat transfer coefficient of 35 W/m$^2$K, density of 7850 kg/m$^3$ and radiative emissivity of 0.7. The temperature evolution of beams and columns in different compartments is presented in Figure 3 for fire scenario 1 and in Figure 4 for fire scenario 2.

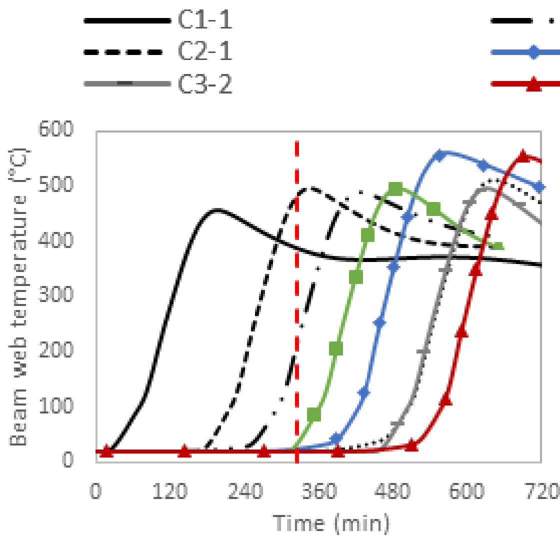


Figure 3. Steel beam and column web temperatures for fire scenario 1

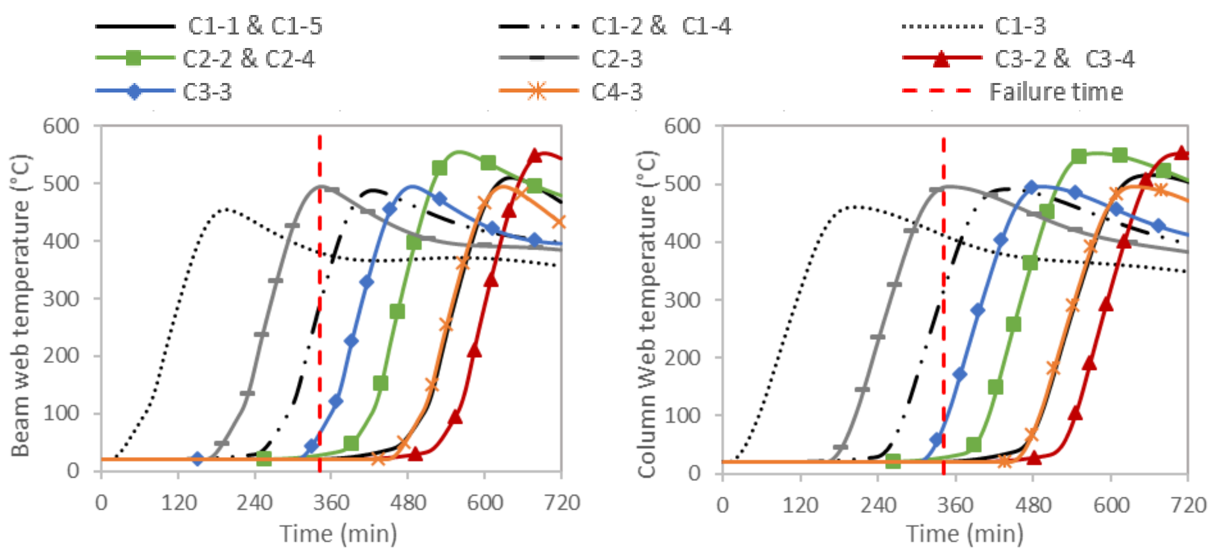
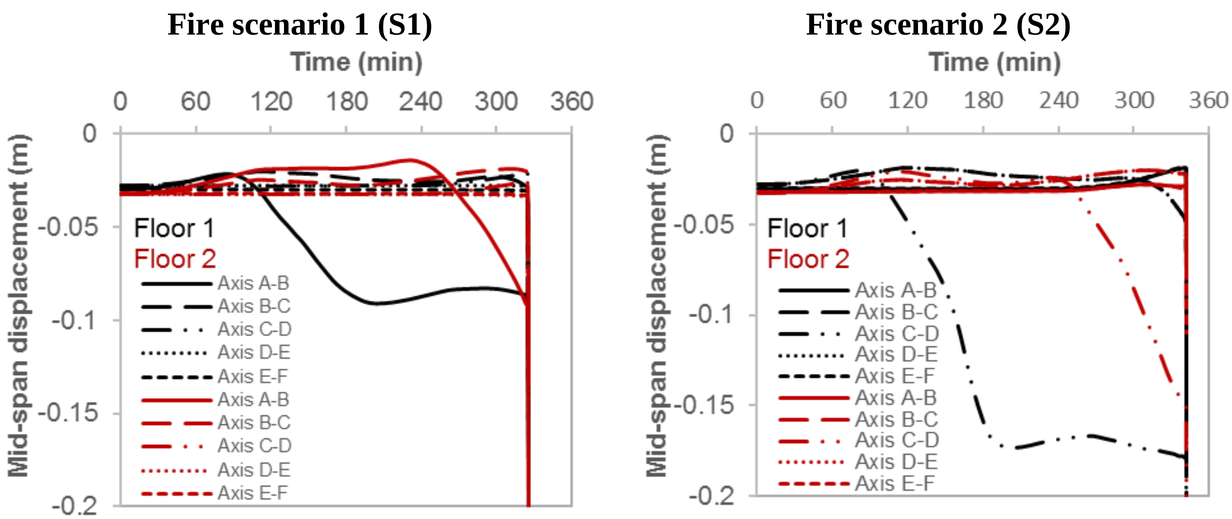


Figure 4. Steel beam and column web temperatures for fire scenario 2

*Structural analysis and the global response*

The structural model was built with the following assumptions: the supports for the ground floor columns are assumed to be fixed while the beams and columns are assumed rigidly connected. The steel beams and columns are model using beam elements, and the beam element length was around 0.25 m. The Eurocode (CEN. EN 1991-1-2:2005) temperature dependent material properties are used for columns and beams.

Figure 5 shows the mid-span displacement, the axial force, and the mid-span bending moment versus time, developed within the beams on floor 1 and floor 2. The failure (defined as the formation of plastic hinges at the beam ends, thus leading to global instability of the building) occurred at about 360 min for both cases, S1 and S2. At the time of failure, temperature in the compartment where the fire initiated was on the descending branch (cooling phase) and the fire had already spread to the compartment above and to the compartments adjacent to the compartment where the fire initiated. Therefore, the figures presented in this section focus on the members of floor 1 and floor 2. The fire initiated in the corner compartment (Floor 1) in the case of fire scenario 1 (S1) and in the central compartment (Floor 1) in the case of the fire scenario 2 (S2). The mid-span displacements were higher in the case of S2 than in the case of S1. The axial restraint for the beam in the central compartment was higher than in the case of the beam in the corner compartment. This is due to the restrain to thermal expansion which is provided by multiple columns, whereas, for the fire scenario S1, the restrain to thermal expansion is provided by only one column on one of the sides. The axial force developed in the beams close to the fire compartments oscillated between compression and tension in the case of S1. In the case of S2, the Floor 1 beams exposed to fire were under compression during the entire duration of the analysis while the Floor 2 beams were starting to be compressed at about 240 min (before the beam temperature of compartment C7 reached the maximum temperature). Larger bending moments developed in the central beam (in the case of S2) than in the corner beam (in the case of S2). The mid-span bending moment developed in the corner beam (in the case of S1) reached the peak value of about 100 kNm, while the central beam (in the case of S2) reached the peak value of about 140 kNm.



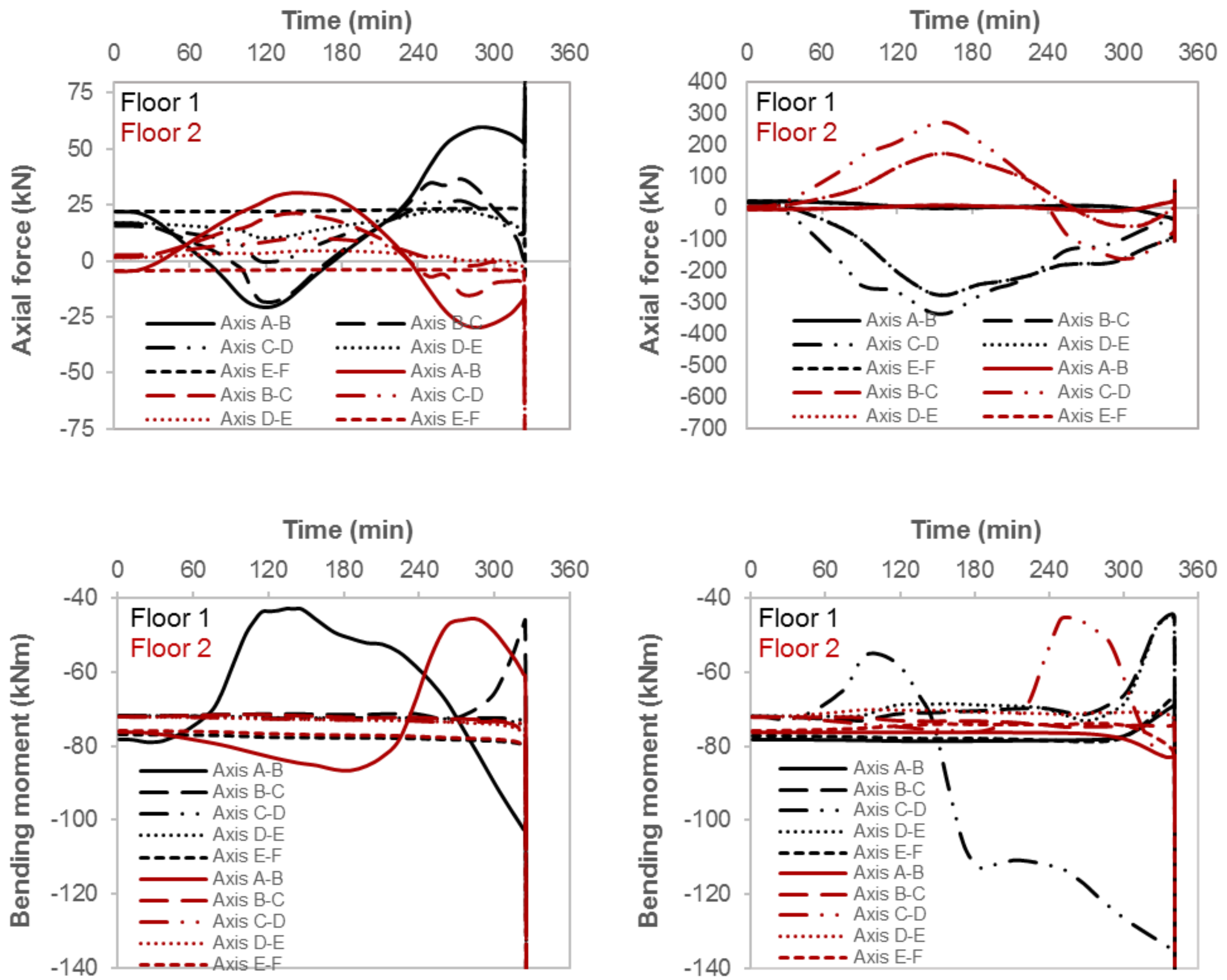



Figure 5. Developments of mid-span displacements, axial force, mid-span bending moment within beam of the floor 1 and 2 for fire scenario 1 (S1) and 2 (S2)

Figure 6 presents the axial displacement developed within columns of floors 1 and 2 for the fire scenarios S1 and S2. Larger axial displacements developed in the exterior columns, where the forces were lower than in the case of the central columns.

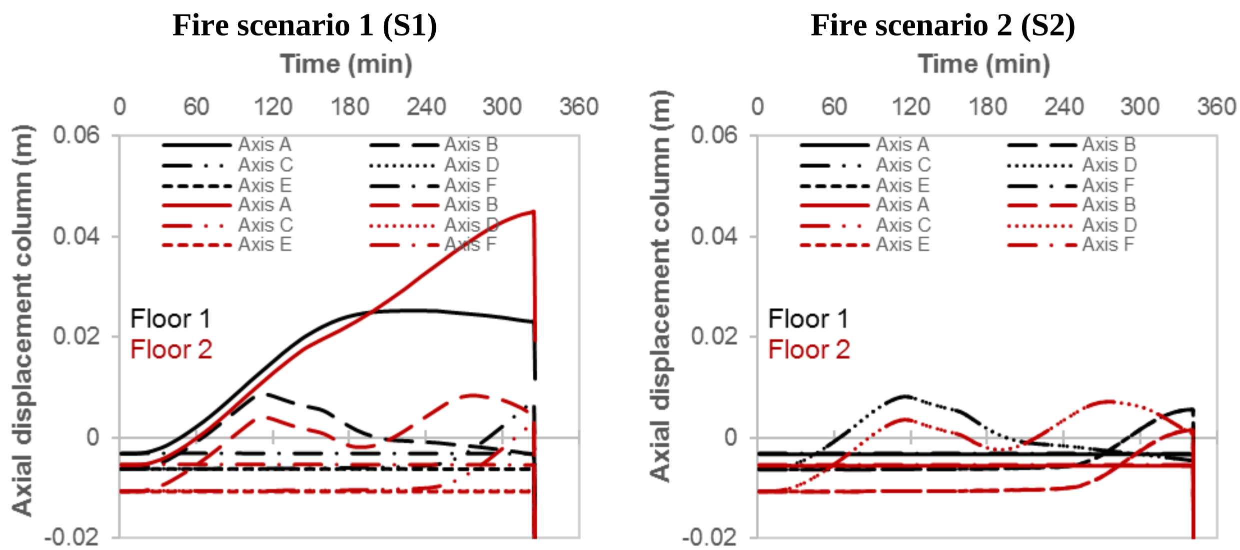



Figure 6. Developments of axial displacements within columns of the floor 1 and 2 for fire scenario 1 (S1) and 2 (S2)

From the global thermal analysis of the 10 story building, it is determined that the beam where the fire initiated (first floor corner beam (S1) and first floor central beam (S2)) is most critical, and therefore will be the focus of the structural analysis described in this section.

Numerical analysis was performed to compare and observe the behavior of the beam when exposed to fire scenarios S1, and S2, in different possible testing configurations. The global behavior of the building was presented in the previous sections and a full scale test would reproduce similar results. Since, as mentioned above, full scale testing is not a cost-effective approach, standard tests are often performed on individual structural members (or sub-structural assemblies). In standard testing, the beam is tested as an individual structural member under constant mechanical boundary conditions. Here, various boundary conditions were considered for the individual beam testing, i.e. f-f (fixed-fixed ends), f-th (fixed-fixed ends with free thermal expansions), f-h (fixed-hinged ends), f-r (fixed-rolling supports), s-s (simply supported beam), h-h (hinged-hinged ends).

Figure 7 presents the mid-span displacements of the beam situated in the compartment where the fire initiated, in different testing configurations. The fire initiates in the first floor corner compartment in the case of fire scenario S1 and in the first floor central compartment for the fire scenario S2. The beams in these two cases are exposed to the same temperature.

Fig. 7 shows that when considering the standard testing approach, regardless of the end conditions, no failure is observed. However, if a full-scale testing were to be performed instead, the failure would occur in the both cases, when exposed to S1 or S2.

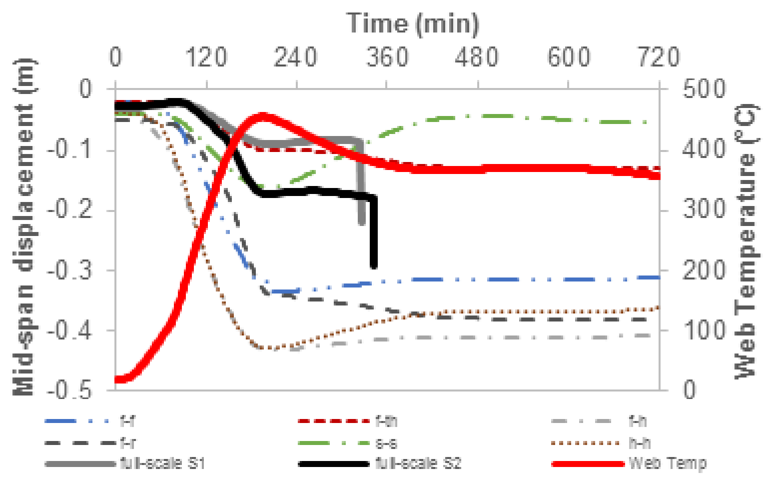



Figure 7. The mid-span displacement of the beams (compartment where the fire initiated) in different testing configurations and web temperature when exposed to natural fire scenarios S1 and S2

This is a clear example that standard fire testing (with constant mechanical boundary conditions) is not always a safe approach when natural fire scenarios are to be considered. Moreover, the standard fire testing cannot capture the effects of the fire spreading from one compartment to the others. Even if the temperature of the steel elements did not exceed the critical temperature of 550°C, the failure still occurred by the time the fire spread to the adjacent compartments. The unheated surrounding plays an important role in the behavior of the structure since it restrains the heated parts of the structure to thermally expand, thus inducing critical stresses in some structural members. The real behavior can only be reproduced if a full-scale test is performed, which, as discussed, is not often feasible. Nevertheless, the same global behavior can be reproduced using HFT. This is possible since the surrounding structure is modeled aside (on a computer) and the interaction with the tested member (which is usually an individual structural member or sub-assembly) is considered during the entire fire test. Thus, HFT represents the global behavior of the building (which proved to be critical compared with the individual testing) and at the same time can consider the fire spread in the adjacent compartments.

### *4.2. Virtual Hybrid Fire Testing*

The goal in this section is to investigate the global structural response of the 10 story building when exposed to the fire scenarios 1 through a Virtual Hybrid Fire Test (VHFT). VHFT refers to the case when the tested PS is also modeled in a computer (tested virtually) instead of being physically tested in a lab. Therefore, in a VHFT, the physical substructure PS (tested substructure) is represented by using a finite element model and aside, the numerical substructure NS (surrounding structure) is also modeled in a finite element software. The VHFT will help validate the available methodologies prior to the real tests. Also, some parameters need to be selected prior conducting real HFT, and when theoretical formulations are not available, VHFT helps determine the proper values of these parameters. No attempt was made to capture the connection behavior or localized instability of the beams and columns in this study.

#### *Framework of the VHFT*

In the VHFT, the PS and NS were separately represented in SAFIR. During the analysis, the two substructures were communicating every time step, thus simulating a real hybrid fire test. The exchange of information (i.e. interface displacements and reaction forces) between the PS and

NS is done every time step and in order to increase the efficiency, some modifications were done in the nonlinear finite element SAFIR code to allow this exchange. The framework for communication for this VHFT was developed in Matlab (2016) and National Instrument's LabView (Elliott et al. (2007)) was used to establish data exchange between SAFIR and Matlab (Figure 8).

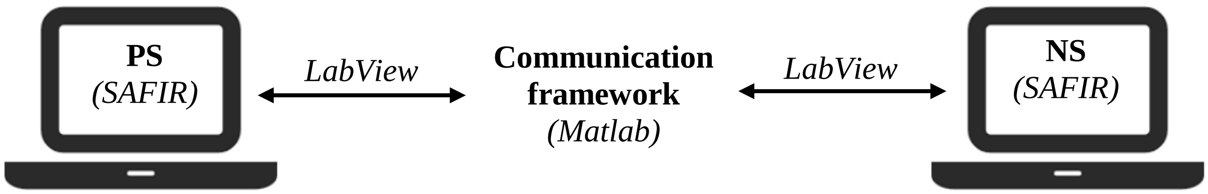



Figure 8. Communication in VHFT

*Results of the VHFT*

Fire scenario S1 was analyzed by means of VHFT. The analyzed structure was divided to the PS and NS presented in Figure 9. The beam of the initial fire compartment represents the PS while the surrounding structure represents the NS. Since a 2D analysis was performed, 6 DoF were controlled at the interface of the PS and NS. The displacements and forces of the PS and NS for these DoFs need to be compatible and in equilibrium for the entire duration of the VHFT. The VHFT must reproduce; the global behavior of the analyzed structure, in addition to the interface compatibility and equilibrium.

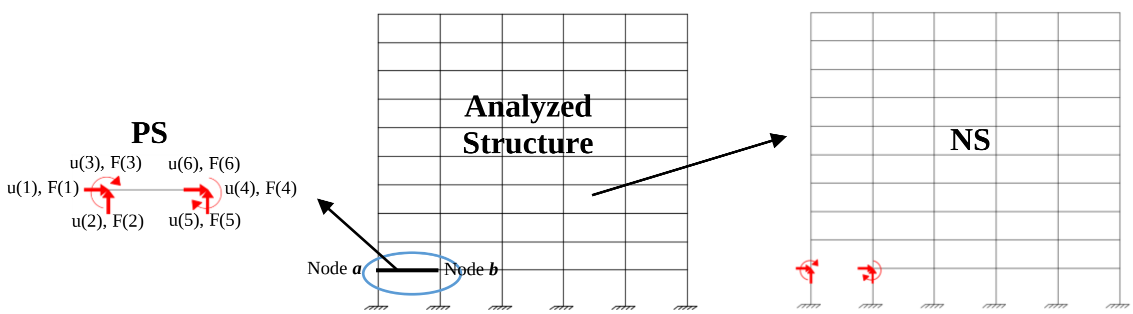
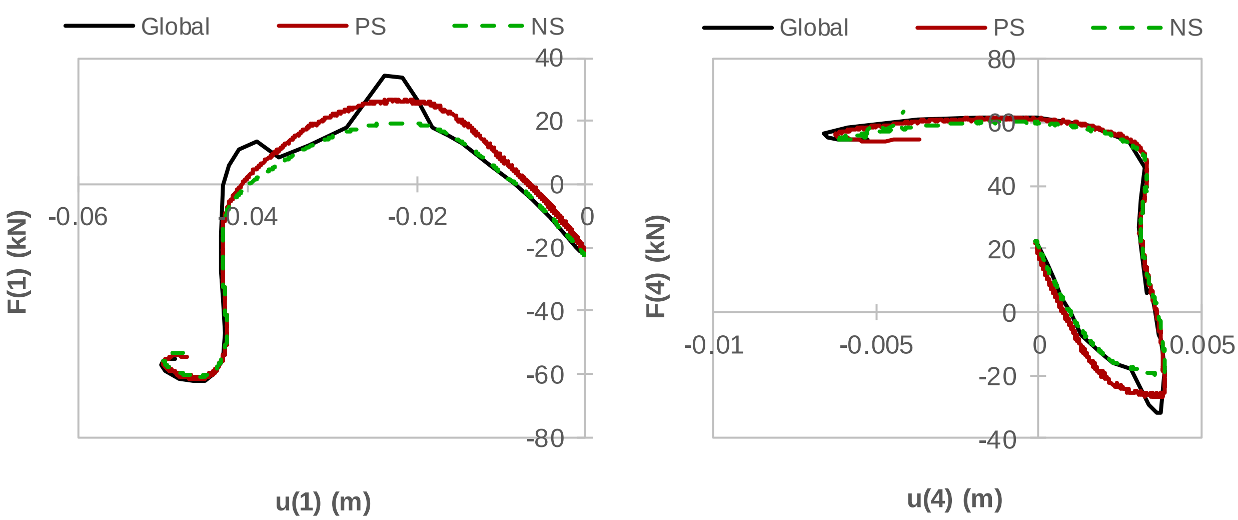



Figure 9. The PS and NS in the case of the VHFT

Figure 10 presents the evolution of forces and displacements for the interface DoFs at nodes *a* and *b*. Their notation is shown on Fig. 9 as follows: *1* – axial DoF of node *a*, *2* – vertical DoF of node *a*, *3* – rotational DoF of node *a*, *4* – axial DoF of node *b*, *5* – vertical DoF of node *b*, and *6* – rotational DoF of node *b*. Each sub-plot illustrates the interface solutions of the PS and NS resulting from the VHFT versus the global solution from the global analysis of the structure. The compatibility and equilibrium are ensured if the solution of PS equals the solution of the NS in the case of the VHFT. Their results should in turn match the global behavior of the analyzed structure.

To perform the VHFT, parameters such as the stiffnesses of the PS and NS, as well as the time step must be defined prior the analysis. The exact values of the stiffnesses of the PS and NS are continually changing during the VHFT, and hard to specifically predict at each time step. Their values at ambient temperature were used and kept constant in this initial phase of the study. Since the stiffnesses were kept constant, iterations were performed to converge to the global solution. Moreover, since the fire effect is continuous (continually changing the boundary (interface) conditions), this iteration process needs to be done continuously. In this initial analysis, it was assumed that 10 *s* are required to perform each time step's calculations and to apply the updated boundary conditions to the PS and the NS. Comparing the interface forces and displacements between the PS and the NS, initial results show that although equilibrium is not fully satisfied, axial and rotational (DoF 1, 3, 4, and 6) results show a good match. However vertical (Dof 2 and 5) had significant differences (up to 30 %). Also, the axial and rotational (DoF 1, 3, 4, and 6) results followed the global behavior very closely.

It is noted that these are preliminary results presented as a demonstration of the framework and as a proof of concept. An optimization process is needed (currently underway) to optimize the parameters used in the solution (such as the time step and the stiffnesses). Initial results from the optimization analysis already show that using a smaller time step dramatically improves the solution's convergence. However, note that the smaller the time step is, the faster of a reaction time of the HFT actuators needs to be. It also requires using higher computation power. Thus, allowing the new boundary conditions to be applied faster on the PS, and the computation of each solution step to be done faster. Since, the actuators and computers limit how small of a time step can be used, it is beneficial to also optimize the updating process of the PS and NS stiffnesses values used in the calculations for each time step. Finally, after optimizing the parameters, an uncertainty analysis is required to quantify the precision and accuracy of the solution.



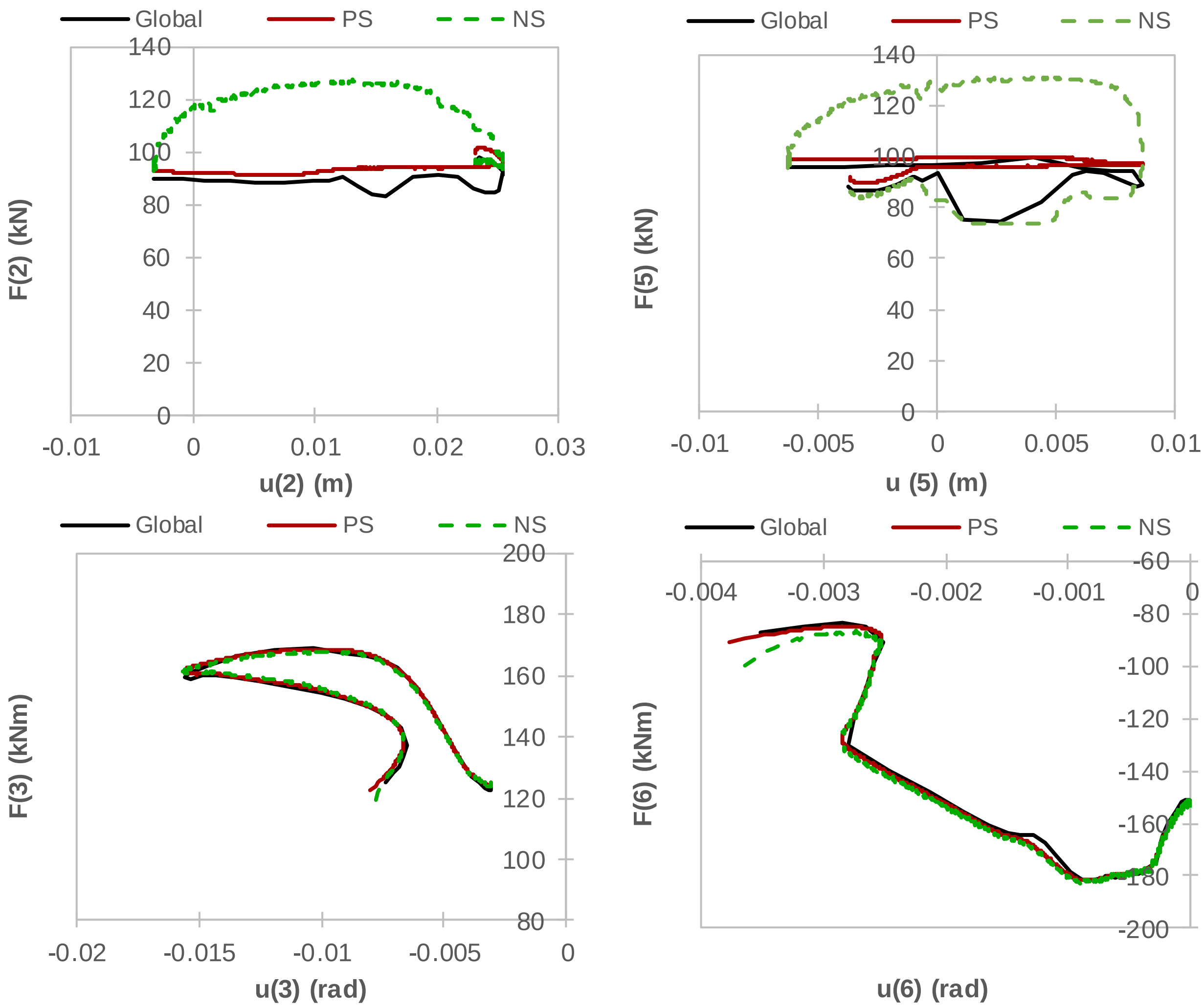



Figure 10. Interface forces versus displacements for the six interface DoF

## 5. Summary

This paper presented the results of a heat analysis performed on a 10 story steel building when exposed to spreading natural fire scenarios. First, the global behavior of the structure was presented for two fire scenarios. In one fire scenario (S1), the fire initiated in the first-floor corner compartment and spread to the adjacent compartments. In the second fire scenario (S2), the fire initiated in the first-floor central compartment and then spreads in the adjacent compartments. The results showed the impact of the unheated compartments on the global behavior of the structure, thus when fire initiates in the central compartment (S2), the thermal expansion is restrained by the unheated surrounding, leading to the increase of mid-span displacements, axial forces and mid-span bending moments. When the fire initiates in the corner compartment (S1), the thermal expansion is restrained in one direction but less restrained in the other direction, therefore the mid-span displacements, axial forces, and bending moments are not as significant as in the S2 case. The temperature of the structural members does not reach the critical temperature of 550°C, nevertheless, the failure occurs due to the fire spread in the compartments. This analysis highlighted the importance of considering the effect of surrounding members (the structure as a whole system) when performing tests on individual structural elements. For the analyzed case, when the effect of surrounding members is neglected, and individual "standard" member tests are to be performed, no failure is observed, whereas failure of those members was to occur in the global behavior. Afterwards, a virtual hybrid fire test was performed on the 10-story building for the case S1. The PS and NS were modeled in SAFIR and the framework for communication was developed in Matlab. The VHFT showed that the global solution can be reproduced if proper parameters are selected, such as the stiffnesses of the PS, NS and the time step of the analysis. The VHFT is useful in preparing the real HFT to properly select parameters (e.g. the time step of the analysis) when analytical formulations are not available so the interface equilibrium and compatibility are satisfied and the global behavior of the structure is satisfied.

## Disclaimer

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