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Hybrid Fire Testing for Assessing Performance of Structures in Fire – Methodology

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Abstract. A new performance-based assessment approach for structures in fire, referred as Hybrid Fire Testing (HFT) method, is presented in this article. The HFT was developed based on a sub-structuring method, by dividing the whole structure into two substructures, one being tested in a furnace and one being simulated in a computer. This represents a form of “hardware-in-the-loop” simulation.

Using HFT, the performance of the whole building can be evaluated at a very reasonable cost, significantly less than the cost of the direct whole building test. More reliable results than the prescriptive method can also be achieved with comparable and even more comprehensive results than that of a direct full-scale test.

A 6-storey reinforced concrete building was designed, as a prototype for application of the new hybrid fire testing approach. Two fire scenario examples were considered; a 6-storey building with a fire compartment on the first floor, in the center of the building and a 6-storey building with a fire compartment on the third floor. The two substructures for these two HFT scenarios were; one the column in the fire compartment and one the rest of the building.

This paper includes description of the hybrid testing methodology, details of the 6-storey building prototype and the methodology verification.

Using the HFT approach, various scenarios could be explored to couple modelling and testing globally. This may also provide the possibility of running one test in a testing facility, e.g.

NRC's, and running the analysis remotely at a different location. This would make furnace facilities more accessible to the research communities around the globe.

Keywords: hybrid test, fire resistance, performance-based design, performance-based design, hybrid test, full-scale test, test and analysis/simulation, hybrid fire testing, HFT.

Introduction

Hybrid fire testing is a new method for fire performance assessment of structures (Mostafaei 2010 and Mostafaei 2011). There are two general applications for the HFT; it can be employed to assess performance of the whole building for a fire scenario, or to evaluate fire resistance of a single element but by including its interaction with the rest of the building. To evaluate performance of the whole building in fire one may test the whole building physically in fire, using a direct full-scale test. However, such a method is expensive to apply, since the entire building needs to be constructed and tested. In addition, there is almost no flexibility in the configuration and mechanical property of the building. For any new building system, a whole new building specimen needs to be constructed for the direct full-scale test. On the other hand, in case of the traditional single element or prescriptive test method, to evaluate the performance of individual building elements, no interaction is included with the structural system of the whole building. In other words, building elements, such as beams, floors, walls and columns, are tested separately in fire.

The new hybrid fire testing (HFT), described in this report would simulate fire performance of the whole building with much lower cost but with more comprehensive results than the direct full-scale test method. On the other hand for a single element test purpose, HFT provides more reliable results than the prescriptive testing. Furthermore, HFT provides more flexibility than the direct full-scale testing. Various building structural configurations and properties could be tested by building only the structural elements that are tested in furnace.

The HFT described in this report includes load and deformation interactions between the test and analysis. Both the furnace test specimen and the rest of the structure are exposed to an appropriate design fire. In other words, the temperature curve should be known before the test. In case of a real fire test, interactions must include temperature component in addition to the load and deformation. That is to measure temperatures during the test and impose the rest of structures to the same temperatures in the analysis.

The HFT, in this article, was presented for buildings. Future studies may confirm that the same methodology could be applied to structural systems, such as bridges and to other structural elements, including beams, floors and walls, with different fire compartmentation scenarios. The same methodology could be employed for a structure divided into three or more substructures, when two elements or more required to be tested in different furnaces. For the purpose of the HFT in this report, the column in the fire compartment was exposed to fire in the furnace and the rest of the building was simulated by the SAFIR software. The beams and floor in the fire compartment were also exposed to fire but only in the analysis/simulation.

Hybrid Fire Testing Methodology

Basic concept

The hybrid method is based on a sub-structuring analysis concept, (Kaveh, Bahreininejad and Mostafaei 1999). This is to divide a structure into two, or more sub-structures by including force and deformation interactions amongst the sub-structures. In the case of the HFT employed in this study, there are only two substructures, the column specimen and the rest of the building. The main interaction components between these two substructures are 12 structural components at each end, three displacements and three load components in the directions of the three main axes, X, Y and Z, and three moment and three rotation

components about the three main axes. In other words, at all times, the above twelve components must be identical at each end for the column and for the frame. Another interaction is temperature; temperatures distributions on the column, which should be the same as that of the rest of the building, in the fire compartment, during the test. To do so, an iteration process is employed. First, an analysis is carried out for the whole building to determine the initial values for loads and deformations of the column specimen. Then the iteration process starts by running the column test under the initial loads/deformations. The column response, deformations/loads, will be measured and recorded as inputs for the analysis of the rest of the building. The remaining structure is now analyzed and the results will determine new loads/deformations for the column specimen. This process can be repeated for a reasonable time step in order to determine performance of the whole building for the duration of the fire. The time step must be selected based on the accuracy required for the test results.

Equilibrium and Compatibility Conditions

Figure 1 shows interaction components between a column specimen and the frame for a simple 2D system, which are F_{xf} , Δ_{xf} , F_{yf} , Δ_{yf} , M_f , and θ_f , for the frame and F_{xc} , Δ_{xc} , F_{yc} , Δ_{yc} , M_c , and θ_c , for the column correspondingly. In the case of the 3D frame, these components are also required in the direction of and about the third axis, Z.

To satisfy equilibrium conditions, all the forces in the two directions of X and Y and the moment in Figure 1 must be identical, respectively.

$$F_{xf} = F_{xc} \quad (1-1)$$

$$F_{yf} = F_{yc} \quad (1-2)$$

$$M_f = M_c \quad (1-2)$$

For compatibility conditions, all the displacements in the two directions of X and Y and the rotation in Figure 1 must be identical respectively.

$$\Delta_{xf} = \Delta_{xc} \quad (2-1)$$

$$\Delta_{yf} = \Delta_{yc} \quad (2-2)$$

$$\theta_f = \theta_c \quad (2-3)$$

The equilibrium and compatibility conditions need to be satisfied at each time step during the test. To do so, an iteration process will be carried out during the test as described later in this report.

HFT general method

The column specimen in Figure 1 was located in the first floor. For a general column specimen location such as the column specimen shown in Figure 2, equilibrium and compatibility conditions could be also satisfied.

Equilibrium conditions require:

$$F_{xc} = -F_{xf1} = F_{xf2} \quad (3-1)$$

$$F_{yc} = -F_{yf1} = F_{yf2} \quad (3-2)$$

$$M_{c1} = M_{f1} , M_{c2} = M_{f2} \quad (3-3)$$

Compatibility conditions yield to:

$$\Delta_{xc} = \Delta_{xf2} - \Delta_{xf1} \quad (4-1)$$

$$\Delta_{yc} = \Delta_{yf2} - \Delta_{yf1} \quad (4-2)$$

$$\theta_{c1} = \theta_{f1} , \theta_{c2} = \theta_{f2} \quad (4-3)$$

HFT – general method application challenges

The general HFT method requires a column furnace facility with capability of applying axial and lateral loading as well as imposing rotations to the column's ends. The NRC's column furnace has been designed for such a capability. However, for the purpose of simplicity in the experimentation process, an alternative approach, referred as the equivalent method, was developed. By means of the equivalent method, the HFT can be implemented using a column furnace by only applying lateral and axial load at one end of the column. The equivalent method is described in the next section.

HFT equivalent method

The HFT equivalent approach is similar to the general method, except that for the column specimen in the furnace the column end rotations will be compensated by adding an equivalent lateral displacement. Figure 3 illustrates the HFT equivalent method.

In other words, while rotations at both ends of the column specimen are zero, the column is subjected to the axial load obtained from the frame reaction and a lateral displacement that is the sum of the lateral deformation of the frame and the equivalent displacement obtained from the applied rotation as described here.

Columns are all considered vertical elements with no applied load along their length except at the two ends.

Hence, relationships between forces and displacements of such a column specimen can be determined by Equation (5-1).

$$\begin{Bmatrix} F_{yf1} \\ F_{xf1} \\ M_{f1} \\ F_{yf2} \\ F_{xf2} \\ M_{f2} \end{Bmatrix} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ & 12\frac{EI}{L^3} & 6\frac{EI}{L^2} & 0 & -12\frac{EI}{L^3} & 6\frac{EI}{L^2} \\ & & 4\frac{EI}{L} & 0 & -6\frac{EI}{L^2} & 2\frac{EI}{L} \\ & & & \frac{EA}{L} & 0 & 0 \\ SYM & & & & 12\frac{EI}{L^3} & -6\frac{EI}{L^2} \\ & & & & & 4\frac{EI}{L} \end{bmatrix} \begin{Bmatrix} \Delta_{yf1} \\ \Delta_{xf1} \\ \theta_{f1} \\ \Delta_{yf2} \\ \Delta_{xf2} \\ \theta_{f2} \end{Bmatrix} \quad (5-1)$$

From Equation (5-1), relations between lateral load and moment can be derived as:

$$\begin{Bmatrix} F_{xf1} \\ M_{f1} \\ F_{xf2} \\ M_{f2} \end{Bmatrix} = \frac{EI}{L^3} \begin{bmatrix} 12 & 6L & -12 & 6L \\ 6L & 4L^2 & -6L & 2L^2 \\ -12 & -6L & 12 & -6L \\ 6L & 2L^2 & -6L & 4L^2 \end{bmatrix} \begin{Bmatrix} \Delta_{xf1} \\ \theta_{f1} \\ \Delta_{xf2} \\ \theta_{f2} \end{Bmatrix} \quad (5-2)$$

Column A in Figure 4 has general displacement and rotation conditions. Based on Equation (5-1) axial loads can be treated separately and are not included here. For Column A, based on the relation provided by Equation (5-2), lateral loads and moments are obtained.

$$M_{f1} = (6\Delta_{xf1} + 4\theta_{f1}L - 6\Delta_{xf2} + 2\theta_{f2}L) EI / L^2 \quad (6-1)$$

$$M_{f2} = (6\Delta_{xf1} + 2\theta_{f1}L - 6\Delta_{xf2} + 4\theta_{f2}L) EI / L^2 \quad (6-2)$$

$$F_{xf1} = -F_{xf2} = 6EI[2(\Delta_{xf1} - \Delta_{xf2}) + \theta_{f1}L + \theta_{f2}L] / L^3 \quad (6-3)$$

Column B has the same conditions as the column specimen in Figure 3 with zero end rotations and zero bottom-end's lateral displacement. Lateral loads and moments for Column B are given below:

$$M = 6EI \Delta_{xc} / L^2 \quad (7-1)$$

$$F_{xc} = 12 EI \Delta_{xc} / L^3 \quad (7-2)$$

To have equivalent load conditions for Column A and B, lateral loads for both columns must be the same. This results to the following compatibility condition:

$$\text{If } F_{xc} = F_{xf2} \text{ then } \Delta_{xc} = -[(\theta_{f1} + \theta_{f2}) L / 2 + (\Delta_{xf1} - \Delta_{xf2})] \quad (8)$$

In other words, in order to compensate the employed rotation to the column ends, additional lateral displacement of $(\theta_{f1} + \theta_{f2}) L / 2$ must be added to the column's initial lateral displacement of $(\Delta_{xf1} - \Delta_{xf2})$.

Equivalent average moment for column specimen is:

$$M = -(M_{f1} + M_{f2}) / 2 \quad (9)$$

A useful moment factor used later in this report is R which is defined as:

From (6),

$$R = M_{f1} / M_{f2} = \{(3\Delta_{xf1} - 3\Delta_{xf2} + 2\theta_{f1}L + \theta_{f2}L) / (3\Delta_{xf1} - 3\Delta_{xf2} + \theta_{f1}L + 2\theta_{f2}L)\} \quad (10)$$

Based on the above applied equilibrium conditions, lateral load and displacements are the same for both Column A and Column B. As well, total rotations at the inflection points of Columns A and B and the sum of the end moments are the same. Although rotations and moments at the individual ends are different, this may have a very slight effect on the response simulation of the frame. Since the total rotation can be determined, rotation at one

end of the column on the frame can be obtained by deducting the rotation of the other end from the frame analysis, from the total rotation. The only consideration would be at the ultimate stage. In other words, as in the case of Column A moment at one end is larger than the average moment in Column B, faster failure may occur at that end of Column A than that of the Column B. However, it would not affect the overall failure of the column. This is because the failure would start from one end of Column A but after reaching the plastic moment at this end, the moment will be constant until the other end reaches its plastic moment capacity. Since both ends have the same plastic moment capacities then both will have similar moments at the failure stage.

$$M_u = -(M_{f1u} + M_{f2u})/2 = -M_{f1u}/2 = -M_{f2u}/2 \quad (11)$$

Therefore, moments for both Columns A and B are in equilibrium conditions at the failure stage, except the small difference during the plastic hinging transition. On the other hand, since shear load due to the moment rotation, in fire, would be a fraction of the total shear force of column, such difference is even smaller and can be ignored.

If the column fails in shear or shear-flexural, since both Columns A and B have the same shear stress along the columns, this would result again in a similar failure load. Total axial deformation of the column is the same for columns A and B due to similar total rotations at the inflection point, as they follow the same curvature, hence, axial shear flexure interaction effects would also be included in the response (Mostafaei and Kabeyasawa, 2007).

Above equilibrium conditions and method is verified for a 6-storey building prototype later in this report. The above approach can be applied based on the same concept for a bi-axial lateral loading. Figure 5 shows results of an analysis for a reinforced concrete column with a cross section of 305mm by 305mm and length of 3.8m with fixed bottom end. The top of the column was once subjected to rotation for both horizontal axes and the shear reactions of the column were obtained. Then based on the above equivalent technique, the top of the column

was subjected to lateral loads with the same magnitude in the two directions, and the shear responses were determined. As illustrated in Figure 5, a good agreement was achieved for both methods, when the initial system was subjected only to the rotations and when the equivalent system was subjected to the lateral loads. The figure shows the column's shear force in each direction.

HFT Steps – equilibrium general method

Here are the steps for the implementation of the HFT equilibrium general method.

Step 1 Run analysis/simulation for the entire structure, including the column specimen, at the initial stage or ambient temperature and obtains M_{fi} , θ_{fi} , F_{xfi} , Δ_{xfi} , F_{yfi} , and Δ_{yfi} , where, $i = 1, 2$, see Figure 3. Then determine the column specimen load and deformation components, F_{xc0} , Δ_{xc0} , F_{yc0} , and Δ_{yc0} at the ambient temperature.

$$F_{xc0} = -F_{xf1} = F_{xf2} \quad (12-1)$$

$$F_{yc0} = -F_{yf1} = F_{yf2} \quad (12-2)$$

$$\Delta_{yc0} = \Delta_{yf2} - \Delta_{yf1} \quad (12-3)$$

$$\Delta_{xc0} = -[(\theta_{f1} + \theta_{f2}) L/2 + (\Delta_{xf1} - \Delta_{xf2})] \quad (12-4)$$

Above equation is a sign sensitive. Follow the signs in Figure 4 for the positive and negative directions.

Step 2 For the column specimen in the furnace, apply the initial column's axial load, F_{yc0} , obtained from step 1. Then impose the column to the initial lateral deformation, Δ_{xc0} . The test is now ready to start.

Step 3 Start the fire in the furnace for the column specimen.

Step 4 Read Δ_{yct} and F_{xct} , of the column specimen at time = t . Include P- Δ effects on lateral load due to eccentricity of the axial load for the additional lateral deformation due to the

rotation Equation (13-2). Then, run the analysis/simulation for the frame in Figure 3 while it is subjected to:

$$F_{yf1} = F_{cy} \quad (13-1)$$

$$F_{xf2} = F_{xct} + [F_{yct} (\theta_{f1} + \theta_{f2}) L/2]/L \quad (13-2)$$

$$F_{xf1} = F_{xf2} \quad (13-3)$$

$$\Delta_{yf2} = \Delta_{yct} + \Delta_{yf1} \quad (13-4)$$

$$\text{From (9) and (10) } M_{f1} = -2M/(1+R) \text{ and } M_{f2} = M_{f1}/R \quad (13-5)$$

The results of the analysis/simulation will include Δ_{xf1} , Δ_{yf1} , θ_{f1} , Δ_{xf2} , θ_{f2} , F_{yf2} at time = t.

Step 5 For the column specimen in the furnace, employ the obtained column's axial load, $F_{yc} = F_{yf2}$, and lateral deformation, Δ_{xc} using Equation (8).

Step 6 Repeat steps 4 to 5 for each time increment, Δt , for the entire period of the test including the cooling phase. Δt is depending on the level of the acceptable error.

A SIX-Storey Reinforced Concrete Building for HFT

A 6-storey reinforced concrete building was designed based on the Canadian Building Code, for design loads, and Concrete Design Standard for a hybrid fire test. The objective of the test was commissioning the HFT and verifying application of the HFT method in practice.

Building Configuration

The overall configuration of the building structure is shown in Figure 6. Figure 7 illustrates both the floor plan of the building and the elevation of the middle main frame as well as the location of the fire compartment scenario 1, which will be examined later.

The main frames of the building are in the direction with the shorter spans (5.0m), as shown in Figure 7. The frames perpendicular to the main frames are considered secondary frames.

The floor loads are considered to be carried only by the main frames.

Design Criteria

Applied Load

The building is assumed as a commercial building located in Ottawa, Canada. The dead load, live load and the snow load were determined according to the 2010 National Building Code of Canada (NBCC) for different floors. Then the factored loads were calculated based on the CAN/ULC-S101 standard for fire resistance tests. The resulting factored uniform distributed load to be applied at the building roof level during the fire test, was determined as 7 (kPa). The factored uniform distributed load to be applied on the other floor levels was obtained at 11 (kPa). With the span of 6.0 m, the main mid frames at the roof level were subjected to 42 kN/m and the main mid frames at other levels to 66kN/m. The end frames were subjected to half of the above loads accordingly, since they carry loads of only half span. Previously, a reinforced concrete column was tested using the traditional prescriptive method under a constant load of 2000kN. For the sake of comparison, the applied load was adjusted in order to achieve the same level of axial load in the centre column of the first floor. 3D structural analyses were carried out for the building, until the above axial load was achieved. The new applied load obtained for the main mid frames at the roof level was 43.7 kN/m and that for the main mid frames at the other levels was 68.5kN/m. The end frames were subjected to half of the above loads accordingly. The new loads were larger than the initial calculated loads based on CAN/ULC-S101 standard; therefore the applied loads were more than the minimum required load.

Column sections

As mentioned previously, a reinforced concrete column was tested using the traditional prescriptive method under a constant load. For the sake of comparison, the second column specimen, with identical properties, was chosen for this test. The column specimens were 3.8m long with a cross section, the details shown in Figure 8. Concrete compressive strength

measured 96 MPa based on three cylinder compression tests, carried out before the fire test. The concrete was made of siliceous aggregates with a mix of still fibre, 42kg per cubic meter.

Beam sections

The same concrete properties were considered for beams as that for columns. Figure 9 shows the cross section for the beams of the main frames with material properties for concrete and steel. Figure 10 illustrates the cross section for beams in the secondary frames. In order to include stiffness of the floor slabs in the analysis, all beams were designed as T beams. For simplicity, end beams were modeled with the same cross sections as that of the mid beams.

Assumptions

Mechanical properties of the concrete for all sections were identical with 96MPa compressive strength concrete. All the reinforcement steel had 400MPa yielding stress. The numerical analyses were carried out using SAFIR software. Beams and columns were simulated using fibre models and therefore shear responses of the elements were ignored. All the connections were considered moment resisting connections.

The fire compartment was considered to be at the centre of the building, therefore, lateral deformations due to thermal expansion could be ignored. This assumption is verified later in this report. A future study will test an end/corner column with lateral load deformation interactions included. The column specimen was fixed at the top end for rotation. The interaction components between the column specimen and the frame were the column end's axial load and axial deformation.

Verification – Fire Scenario 1

Figure 7 shows the 6-storey building exposed to fire scenario 1. Few assumptions can be made for this fire scenario 1. The main assumption is that, due to the symmetric conditions, rotations at the top end of the column specimens are considered zero. In order to verify this assumption, three 3D numerical analyses were carried out for the 6-storey reinforced concrete

building prototype, shown in Figure 6, using the SAFIR structural analysis software. For the purpose of comparisons, CAN/ULC-S101 fire was selected as the design fire, since the same fire was employed previously for fire resistance of structural assemblies using traditional testing approach.

1) Simulated analysis for the whole building, while all the floor elements and the column in the fire compartment were exposed to the CAN/ULC-S101 fire. The results included axial load of the column in the fire compartment and its top-end vertical deformation at different time steps. Lateral load and lateral deformation at the top end of the column specimen were obtained negligible, which would be due to the symmetric conditions. Hence, lateral components were not included in this fire scenario.

2) Simulated analysis of the building, but without the column specimen while it was subjected to the column's top-end vertical deformation, obtained in analysis (1), determined the corresponding reaction which is F_{yf} . The frame was imposed to zero rotations at the node, where the column was separated. Figure 11 illustrates comparison of the axial load of the column when being part of the whole building and F_{yf} was obtained from this analysis. Figure 12 shows both the obtained top-end vertical deformation of the column from analysis (1) and the employed Δ_{yf} for this analysis. The difference between the two curves is 0.075mm during the entire test which is due to the implemented load adjustment. The adjustment of the initial load was made as described previously in the HFT steps.

3) Simulated analysis of the single column specimen when it was subjected to the axial load of the column obtained from analysis 1 and determined vertical deformation of the column specimen, Δ_{yc} . Zero rotations were imposed at the top of the column. This simulates the same boundary conditions as that of the column in the NRC's column furnace. Figure 13 shows axial load obtained from analysis (1) and the vertical load applied in this analysis on the column specimen F_{yc} . Figure 14 illustrates the comparison of the axial deformation at the top

end of the column, when being part of the whole building, analysis (1), and the vertical displacement of the column specimen, Δ_{yc} , that was obtained from this analysis.

Lateral deformations at the top-end of the column, when it was part of the whole building, obtained from the analysis (1), were negligible. Therefore, no lateral displacement was applied for analyses (2) and (3).

Analyses (2) and (3) are simulating the HFT method. Therefore, since the F_{yf} obtained in analysis (2) is matching the column axial load obtained from analysis (1), Figure 11, and Δ_{yc} obtained in analysis (3) is matching that of analysis (1) for the whole building, In Figure 14, the verification is achieved and the assumptions of zero rotations are confirmed.

Verification – Fire Scenario 2

Further numerical analyses were carried out for a more general fire compartment scenario, using the HFT equivalent method to validate the applicability of the proposed process. At this time, an end column on the third floor of the building was chosen as the column specimen.

Figure 15 shows the building with the location of the column specimen.

The fire compartment scenario 2 is located on the third floor as shown in Figure 16. All the beams and the column in the fire compartment were exposed to the CAN/ULC-S101 standard fire.

Three analyses were carried out:

1) Simulated analysis for the whole building, while the structural elements in the fire compartment were exposed to the CAN/ULC-S101 fire and obtained axial load, shear force, end-moments, top-end and bottom-end displacements and rotations of the column in the fire compartment, referred to as the column specimen. Considering the location of the fire compartment, only displacements in the directions of X and Z axes, Δ_{xf1} , Δ_{xf2} , Δ_{zf1} , Δ_{zf2} , and rotations about Y axis, θ_{yf1} , θ_{yf2} , and their corresponding forces, F_{xf1} , F_{xf2} , F_{zf1} , F_{zf2} , and

moments, M_{yf1} , M_{yf2} , at the top and bottom ends of the column were obtained. In other words, only components related to the main frame are considered and the problem can be treated in two-dimension. F_{zf1} , and F_{zf2} , are in fact axial loads of the column $F_{zf1} = F_{zf2}$. F_{xf1} and F_{xf2} are shear forces of the column. Shear forces at both ends could have different values due to the P- Δ effect. Average value of F_{xf1} and F_{xf2} were considered as the average shear force in the column specimen.

Note: For this analysis, remaining components of the deformations, Δ_{yf1} , Δ_{yf2} , θ_{xf1} , θ_{xf2} , θ_{zf1} , θ_{zf2} , and their corresponding loads and moments were found negligible. In the case of a corner column specimen, Δ_{yf1} , Δ_{yf2} , θ_{xf1} , θ_{xf2} , need to be included in the analysis. However, the same process described here could be employed. Effects of torsions in the column were considered negligible. This would be a reasonable assumption for conventional columns, where, due to relatively long length, compared to its cross sections dimensions, torque could be ignored.

2) Simulated analysis of the single column specimen when it was subjected to axial load, $F_{zc} = F_{zf2}$, lateral deformation, $\Delta_{xc} = -[(\theta_{yf1} + \theta_{yf2}) L/2 + (\Delta_{xf1} - \Delta_{xf2})]$ based on the results in analysis (1) and obtained $F_{xf2} = F_{xct} + [F_{yct} (\theta_{yf1} + \theta_{yf2}) L/2]/L$, Δ_{zc} and end-moments M_{f1} and M_{f2} . It is important to use the right signs, positive or negative, for the variables in these equations to obtain the proper results. Figures 17, 18 and 19 show the results of axial deformation, shear force and moment obtained from this analysis compared with that from the whole building analysis, analysis (1), which indicates a good agreement. Moments were calculated using Equation (13).

3) Simulated analysis of the building but without the column specimen, Figure 15, while it was subjected to F_{xf2} and $F_{xf1} = -F_{xf2}$, $F_{zf2} = F_{zc}$, and $\Delta_{zf2} = \Delta_{zc}$, obtained from analysis (2). No other force, moment, deformation or rotation is imposed on the frame. One may subject the frame also to M_{yf1} and M_{yf2} both calculated by Equation (13) from analysis (2), however, for a large frame, where stiffness of the column compared to that of the whole building is very

small, this could be ignored. For the purpose of comparison, analyses were carried out here with and without including the moment interactions between the column specimen and rest of the building.

Figures 20-23 show vertical and lateral displacements and rotations at the top and bottom ends of the column and its axial load obtained from the analysis of the whole building, analysis (1), and from the analysis of the building without the column specimen, analysis (3), which all show good agreements. These figures illustrate the results with no moment interaction between the column specimen and the rest of the building. In order to assess how effective it is to include moment interactions, an analysis was carried out for the rest of the building when M_{yf1} and M_{yf2} , calculated by Equation (13), were applied to the top and bottom ends on the frame. Figure 24 illustrates the results of this analysis. It indicates a better correlation compared to that in Figure 22. However, the difference is not very significant. Note that the equivalent HFT can be applied with or without moment interactions. The advantage of not having moment interaction included in the analysis is less variable involved and less errors. Moments are sign sensitive and therefore more effort would be needed to make sure the top and bottom column ends are subjected to the correct moments and directions.

Implementation of the HFT

The HFT process, described in this paper, was successfully employed for a 6 –storey building with a fire compartment in the first floor at the NRC-IRC's furnace facility in Ottawa. The results of the test will be published in a separate article.

Conclusion

A Hybrid Fire Testing method was developed and presented for evaluating performance of structures in fire. HFT was employed, numerically, to a 6-storey reinforced concrete building prototype for two fire compartment scenarios, a middle fire compartment in the first floor and a side fire compartment in the third floor. The HFT was validated successfully using the results of a 3D analysis of the whole building, analysis of the column, referred as the column specimen, when separated from the building and analysis of the rest of the building, without the column. The numerical results indicated that the HFT equivalent method could provide reliable results as that of the full structure simulation. The advantage of the equivalent method was that the column furnace did not require having a facility to apply support rotations.

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FIGURE 1. Simplified HFT Method for a Middle Column.

FIGURE 2. HFT General Method

FIGURE 3. HFT Equivalent Method

FIGURE 4. Equivalent Column Ends Conditions.

FIGURE 5. Shear Response, F , of a Reinforced Concrete Column, Calculated Using the Direct Method and the Equivalent Approach.

FIGURE 6. The 6-Story Prototype Reinforced Concrete Building Structure.

FIGURE 7. The Elevation and Floor Plan of the 6-Storey Reinforced Concrete Building – Fire Scenario 1

FIGURE 8. Cross Section of Columns at all Levels.

FIGURE 9. Cross Section of Beams in the Main Frames.

FIGURE 10. Cross Section of Beams in the Secondary Frames.

FIGURE 11. Axial Load of the Column when Being Part of the Whole Building and F_{yf} Obtained from the Analysis of the Building without the Column.

FIGURE 12. Top End Vertical Deformation of the Column when Being Part of the Whole Building and Δ_{yf} imposed to the Building without the Column.

FIGURE 13. Axial Load in the Column Specimen for Both when it is Separated and When it is Part of the Whole Building.

FIGURE 14. Top End Vertical Deformation of the Column Specimen for Both when it is Separated and When it is Part of the Whole Building.

FIGURE 15. The 6-Storey Prototype Building with an End Column Specimen on the 3rd Floor.

FIGURE 16. The Elevation and Floor Plan of the 6-Storey Reinforced Concrete Building – Fire Scenario 2

FIGURE 17. Top End Vertical Deformation of the 3rd Floor's Column Specimen for Both when it is Separated and When it is Part of the Whole Building.

FIGURE 18. Average Shear Force of the 3rd Floor's Column Specimen for Both when it is Separated and When it is Part of the Whole Building.

FIGURE 19. End-Moments of the 3rd Floor's Column Specimen Obtained from Analysis of the Whole Building and Calculated using Equation (13).

FIGURE 20. Top and Bottom Ends Vertical Deformation of the Column when Being Part of the Whole Building and for the Building without the Column.

FIGURE 21. Top and Bottom Ends Lateral Displacement of the Column in X direction when Being Part of the Whole Building and for the Building without the Column Specimen.

FIGURE 22. Top and Bottom Ends Rotations of the Column about Y Axis when Being Part of the Whole Building and for the Building without the Column Specimen – With no Moment Interaction.

FIGURE 23. Axial Load of the Column when Being Part of the Whole Building and for the Building without the Column Specimen.

FIGURE 24. Top and Bottom Ends Rotations of the Column about Y Axis when Being Part of the Whole Building and for the Building without the Column Specimen – With Moment Interaction.