A New Testing Capability for Seismic Resistance Assessment of Structures Damaged Due to a Fire

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SUMMARY:
This paper provides results of a hybrid structural test that simulates residual lateral load resistance of a structure after fire damage. The simulation was carried out at the National Research Council of Canada (NRC) using the NRC's furnace facilities. The hybrid test included two substructures; a structural element/assembly specimen, here a column specimen, and a model component, here the remaining of the structure. Using this method, first, fire damage was imposed to a structure, a 6-storey reinforced concrete building, and six days later, after the entire structure had cooled down to the ambient temperature, the building was subjected to a lateral load, which was determined based on a design seismic load. The test results showed a reduction of both residual lateral stiffness and residual lateral load capacity of the structure after the fire damage. This paper will present the hybrid test, its application and the results for the 6-storey building.

Keywords: Fire damage, seismic resistance, reinforced concrete, column, hybrid test, fire resistance, performance-based evaluation

1. INSTRUCTIONS

A review on the US statistics reports and studies show that the average annual fire occurrence in moderate and high-rise buildings exceeds 10,000 incidents in the United States (Hall, 2001). In Canada, more than 50,000 fire incidents were reported annually CCFMFC (2002). Approximately 3500 of these fire incidents occurred in apartments, hotels and dormitories, which could be considered as moderate to high-rise structures. In most of these fire incidents, building structures have experienced minor to major damage due to fire exposure, such as degradation of material properties due to elevated temperatures and damage to structural elements due to thermal expansion.

After a building fire incident, an inspection is required for the assessment of the structural damage and evaluation of the building residual capacity (CIB W14, 1990). One of the main objectives for a post-fire assessment should include effects of the fire damage on the structural performance, such as on the lateral load/seismic resistance of the structures. Studies have been carried out on seismic resistance of fire-damaged concrete columns using numerical modelling (Mostafaei et al. 2009). However, few experimental methods have been developed and implemented to assess the residual seismic capacity of structures after fire.

This paper provides a brief description of a test to explore residual lateral resistance of a RC structure after a fire using a hybrid method.

2. THE PROTOTYPE STRUCTURE

A six-storey reinforced concrete building prototype was designed based on the Canadian Seismic Code and Concrete Design Standard for this study. Figure 1 shows the overall 3D structural frame configuration of the building. Details of the design were described by Mostafaei (2011).
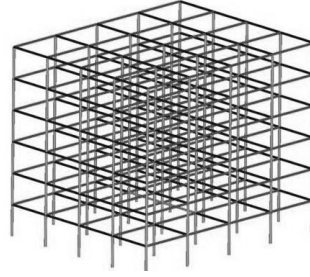
**Figure 1.** The Reinforced Concrete Building Structure prototype.

For this study a fire scenario was considered on the first floor of the building. Figure 2 shows both the floor plan of the building and the elevation of the main frame as well as the location of the fire compartment for the fire test. The floor loads are considered to be carried only by the main frames. The frames perpendicular to the main frames are considered secondary frames.
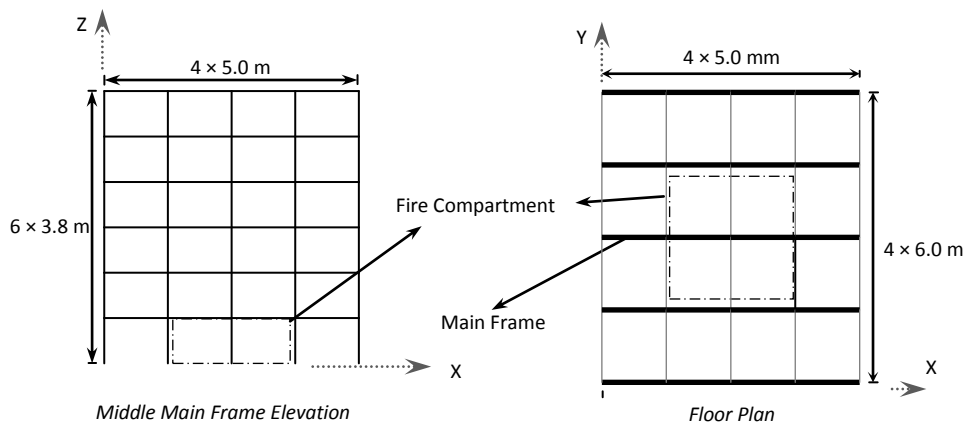
Figure 2. Elevation and floor plan of the building with location of the fire compartment.

2.1. Column Elements

The cross section and details of the reinforcements for columns are shown in Figure 3. The concrete had a compressive strength of 96 MPa and was made of siliceous aggregates with a mix of steel fibre, 42 kg per cubic meter.
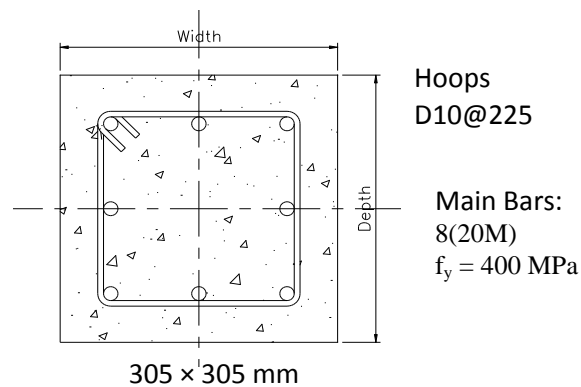

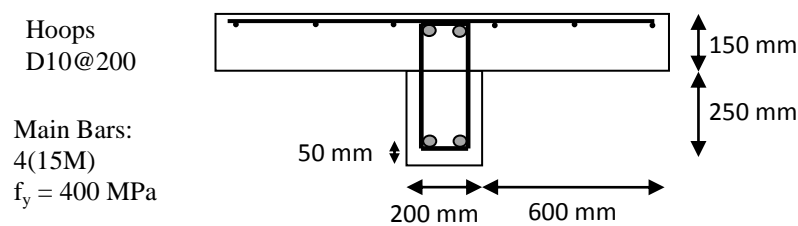
Figure 3. Cross Section of Columns at all Levels.

2.2. Beam Elements

The same concrete properties were considered for beams as that for the columns. Figure 4 shows the cross section for the beams of the main frame with material properties of the steel. Figure 5 illustrates the cross section for beams in the secondary frames. In order to include contribution of the floor slabs

in the building response, all beams were designed as T beams. For simplicity, end beams were modelled with the same cross sections as that of the mid beams.


Hoops
D10@200

Main Bars:
4(25M)
$f_y = 400$ MPa

150 mm

500 mm

50 mm

300 mm

500 mm
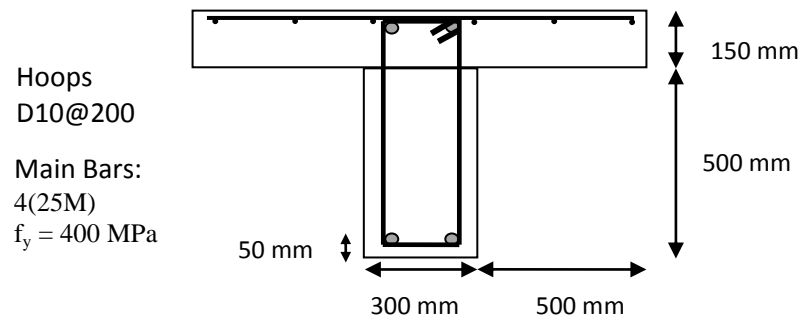
Figure 4. Cross Section of Beams for the Main Frames.

Hoops
D10@200

Main Bars:
4(15M)
$f_y = 400$ MPa

50 mm

150 mm

250 mm

200 mm

600 mm

Figure 5. Cross Section of Beams for the Secondary Frames.

3. FIRE LOAD

For the fire test of the building, a hybrid fire testing (HFT) was employed. Figure 6 illustrates the HFT dividing the 6-storey building prototype into two substructures: 1) columns specimen and 2) model component. The column specimen, which was the centre column on the first floor of the building, in the centre of the fire compartment, was tested in the furnace. The model component, the rest of the building structure, including beams and the floor in the fire compartment, was simulated using the SAFIR software. The average temperatures in the fire compartment, both in the furnace and in the simulation during the test, were controlled based on the CAN/ULC-S101 standard temperature-time curve. Details of the test method and complete results were reported by Mostafaei et al. (2011).

Model Component

Interaction

Whole structure

Test Specimen
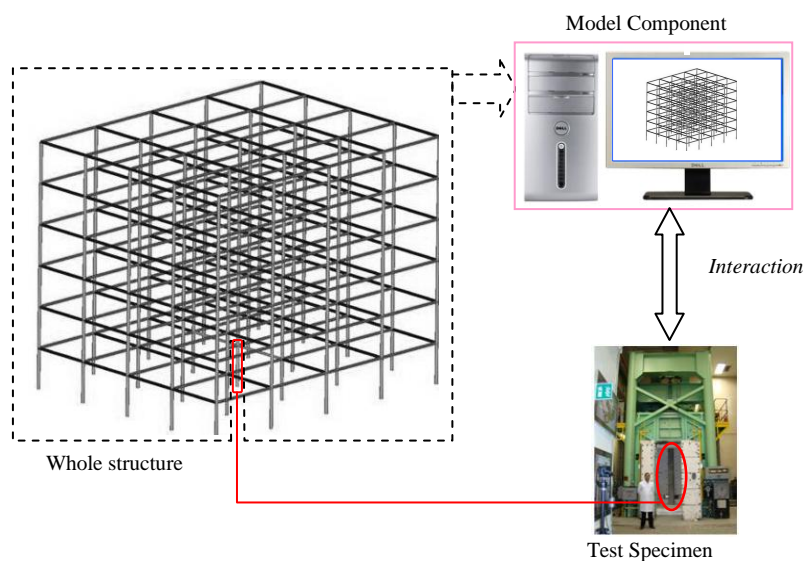
Figure 6. The HFT method for the 6-Storey Reinforced Concrete Building.

4. LATERAL LOAD

After the fire exposure, the building was evaluated to assess its residual lateral load capacity. Here, lateral load was assumed to be due to a seismic load. The seismic weight of the building was determined considering a commercial building located in Ottawa, based on the Canadian National Building Code.

As for the results, total seismic weight of the roof was $W_6 = 2544 kN$ and that for the 1st to 5th floors were, $W_1 = W_2 = W_3 = W_4 = W_5 = 4104$ kN, respectively. Lateral load distribution of the building was determined by Equation (4.1).

$$F_x = (V - F_t) \frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i} \tag{4.1}$$

where, w is the weight of each floor; h is height of each floor from the foundation level of the building. V is the total lateral seismic load and F_t is the portion of V to be applied at the roof level. In this study, since the natural period of the building is larger than 0.7s, based on the employed seismic code, $F_t = 0$.

The vertical load applied on the main mid frames at roof level was 33 kN/m and at other levels 53 kN/m. The end frames were subjected to half of the above loads accordingly.

5. TEST FACILITY

The tests were carried out using the NRC's full-scale column furnace facility in Ottawa. The furnace is capable of applying axial loads up to 9790 kN (2200 kips), lateral loads up to 110 kN (25 kips) in North-South direction, lateral loads up to 310 kN (70 kips) in East-West direction with/without fire loads. Further details are provided by Lie (1980).

During the test, the axial load is controlled by servo-controllers and measured with pressure transducers with ~4.0 kN accuracy at lower load levels and relatively better at higher loads. Lateral loads are controlled and measured with load cells.

Lateral and axial displacements are measured using transducers with an accuracy of ~0.002 mm.
The furnace chamber has a floor area of 2642 x 2642 mm and height of 3048 mm. The axial loading and lateral loading could be applied to the column specimen during the fire or before and after, without the fire. Figure 7 shows the column furnace chamber.


Figure 7. NRC's Column Furnace Facility.

6. STRUCTURAL PERFORMANCE RESULTS

6.1. Structural Response in Fire

First, structural response evaluation to fire was performed to assess the level of fire damage on the building prototype. This was carried out using the hybrid fire testing, an outline of which is depicted in Figure 6. Figure 8 illustrates the response of axial load and axial deformation of the column during the fire test. The axial deformation is also shown for the cooling phase for up to six days after the fire test. Note that due to the symmetric loading, fire and structural conditions, no lateral deformation interaction was employed between the test specimen and the model component. Hence the results here are shown only for the axial load and deformation interactions.

a. Axial Displacement Response

b. Axial Load Response
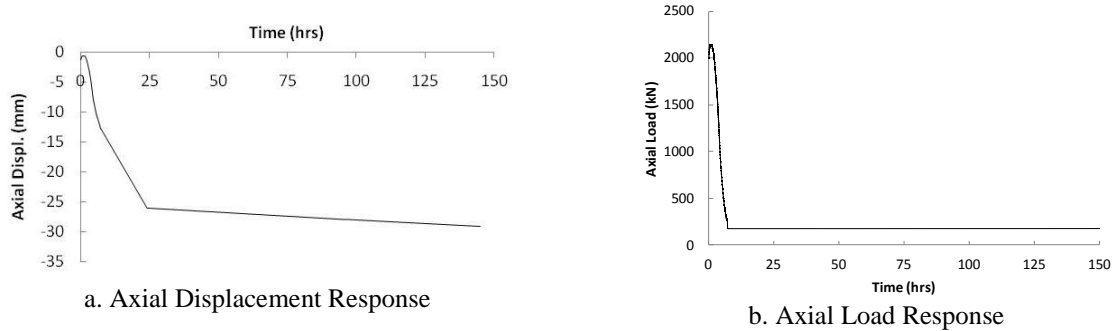
Figure 8. Axial Load and Axial Displacement Response of the Column (Test Specimen).

The results shown in Figure 8b indicate a considerable reduction in the axial load of the column to almost a zero load, during the fire test. In other words, the column did not carry any further load, since it became significantly shorter than its initial length and lost its interaction with the frame. In other words, the frame carried the entire load of the column after the fire. In fact, the axial load continued to drop however, for the sake of testing, a minimum axial load was applied on the column specimen and the test continued for 6 more days. Note that during this period, the minimum axial load was applied only once a day when measuring the corresponding axial displacement. For the remaining time, the hydraulic jack for the axial load was turned off.

6.2. Lateral Structural Response

In order to evaluate residual lateral load and deformation capacity, the structure prototype was subjected to both axial load and lateral deformation. Figure 8b shows that the axial load of the column after the fire test was reduced to almost zero. However, for the purpose of a comparison, considering a worst-case scenario, the axial load of the column was increased to its initial load applied before the fire test. One may assume that due to the creep in concrete, the beams eventually regain their contact with the column, resulting in recovery of the initial column's axial load, which is 2000kN.

The same hybrid test method, as applied for the fire test, was employed for the lateral response simulation. However, in the later case, lateral deformation interaction was also employed, in the loading direction, between the column specimen and the model component, in addition to the axial load interaction (Mostafaei 2011).

To achieve post peak response, a displacement controlled lateral loading was desirable. A separate analysis was carried out for the whole building, before the test, under the lateral loads, obtained by Equation 4.1, to estimate displacement distributions among different levels of the building. Then, the same relation was employed to control lateral displacement of the building during the test. Lateral displacements were increased until lateral displacement ratio of the column specimen reached about 1.1%, which was the current lateral displacement limit for the facility.
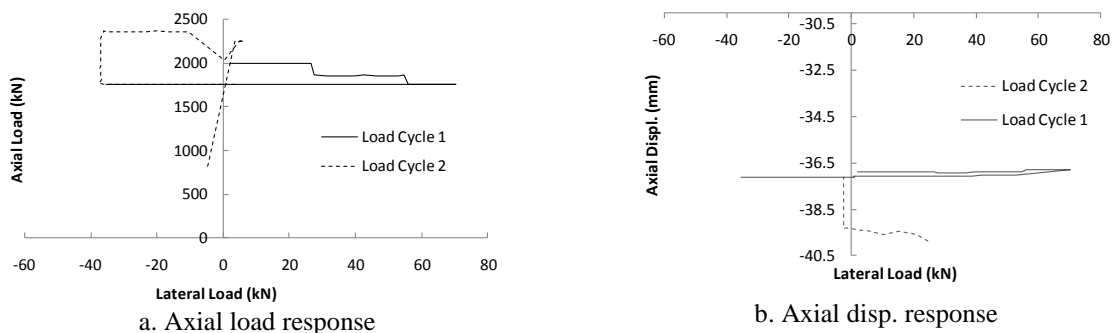
6.2.1. Response of the column specimen

Since the column did not fail at 1.1% displacement ratio, in order to achieve the column failure, the axial load was increased for the next cycle to 2350kN. The column failed at this axial load and under a lateral load of 6kN or lateral displacement of 6mm. Since this is a very small lateral displacement, it could be concluded that the column failed, in fact, under the only applied axial load. The result shows that the column lost more than 70% of its axial load capacity due to the fire damage. Furthermore, this may indicate that the lateral displacement ratio of 1.1% and the lateral load of 70kN, achieved in the previous cycle, could be considered as the column's lateral deformation and load capacity.

For the purpose of comparison, an analysis was carried out for the column specimen to evaluate its lateral load response before any fire damage, using a simple method called ASFI, (Mostafaei and Hum 2010). Figure 9 illustrates the lateral response of the column specimen before the fire, estimated by ASFI method and that after the fire, obtained from the test. The results indicate a 50% reduction in the column's maximum lateral deformation capacity and about the same reduction in its lateral load capacity after fire damage. For verification of the ASFI results obtained for the column specimen in this study see Mostafaei (2011).
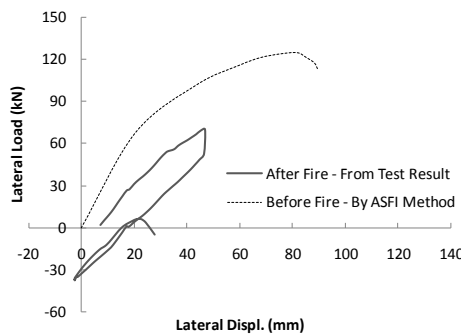
Figure 9. Lateral Load Response of the Column Specimen after Fire Test until failure; before and after Fire.

Figure 10(a) illustrates the axial and lateral load response of the column for two cycles; cycle 1 started with an axial load of 2000kN and cycle 2 started with an axial load of 2350kN. The figure shows that the axial load of the column was decreased from 2000kN at about a zero lateral load to 1750kN at 69kN lateral load in three steps, which indicates the effects from the interaction of the column specimen with the rest of the building, the model component. That could be due to relatively lower stiffness of the column specimen compared to the other columns of the frame, due to the fire damage. Increasing the lateral load resulted in lifting up the other columns of the building, due to their axial-flexure mechanism. However, the column specimen, damaged in fire, had much less stiffness and could not follow the lift up; therefore its axial load reduced as the lateral displacement rose.

a. Axial load response

b. Axial disp. response

Figure 10. Axial Load/Displacement vs. Lateral Load Response of the Fire-damaged Column Specimen.

Figure 10(b) shows the same results but for axial displacement. It indicates that at the column failure, the column has experimented 39 mm axial deformation, from the time before fire, column shortening, which is about 1% of its height. That is about 400% more than axial deformation failure of a column with no fire damage, which is about 0.25%. However, on the day of the lateral load test, when the

axial load was increased from zero to 2350kN, the failure load, the corresponding relative axial deformation was 12 mm. That is about 0.3% strain of concrete compressive capacity, which seems to be reasonable and very close to that of the concrete with no fire damage.

Figure 11 shows the column specimen before and after the hybrid fire testing. Figure 12 illustrates the same column specimen, 6 days later, before and after the lateral load test.

Figure 11. Column Specimen before, on the Left, and just after the Fire Test, on the Right.
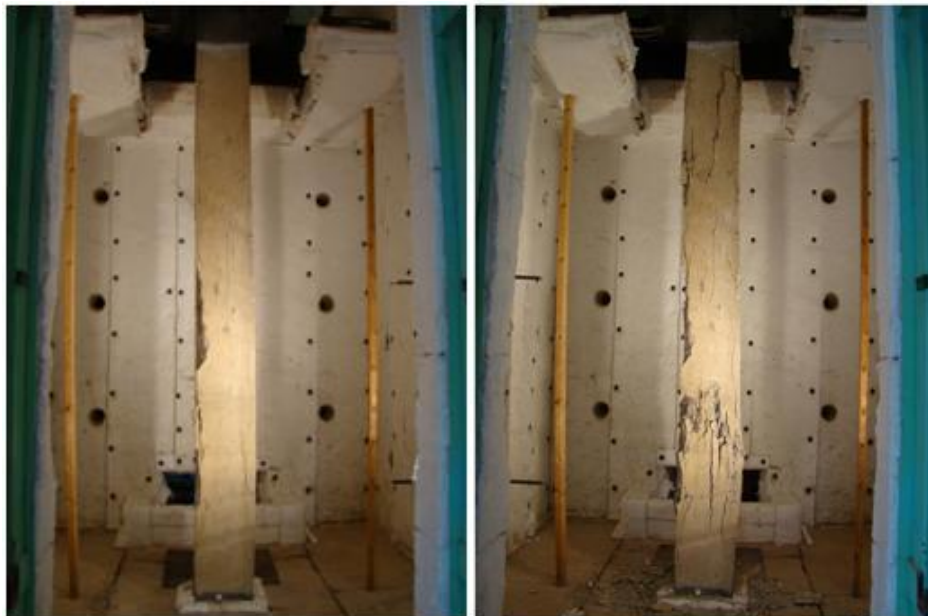

Figure 12. Column Specimen Six Days after the Fire Test, on the Left, before the Lateral Load Test, and on the Right, after the Lateral Load Test.

6.2.2. Response of the model component
The numerical simulation of the model component was carried out using a computer software called SAFIR (2005), during the hybrid test. Figure 10 shows that the axial load was changed three times during the load cycle. This means the analysis was carried out at least three times during this cycle. For a smoother response, more analyses can be implemented during each load cycle.

Although, all the performance components for the building structure were calculated during the test, this study focuses more on the application of the hybrid seismic test of fire-damaged buildings. Therefore, only overall results of the structural performance of the building were provided in this paper. Figure 13 illustrates overall lateral deformation of the building at the time when the lateral displacement at the top of the column reached its maximum value. The building experienced 217 mm lateral displacement at the roof level, which is about 1.1% drift ratio.
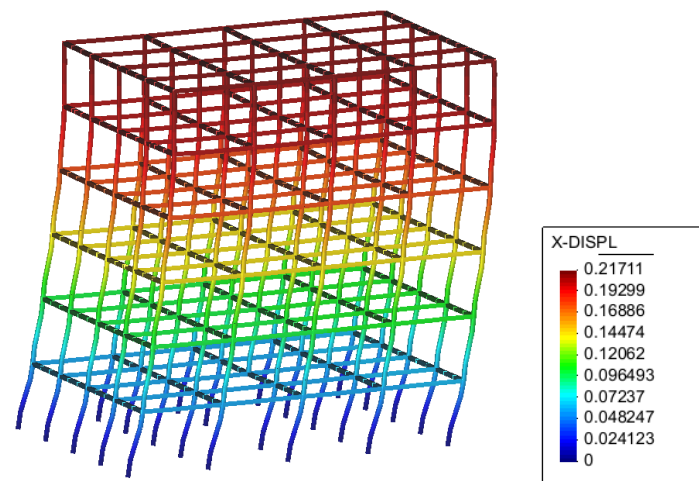
Figure 13. Lateral Displacement Response of the 6-Storey Building corresponding to the Time at which the Column Specimen Experienced Its Maximum Lateral Displacement.

7. CONCLUSIONS

A hybrid testing was implemented successfully to evaluate residual lateral resistance of a fire-damaged 6-storey reinforced concrete building. The following conclusions can be made based on the results of this study.

1. Fire damage could cause considerable reduction in lateral load and deformation capacity of concrete columns.

2. Even after large permanent axial deformation during and after the fire, the column specimen failed in compression at about 0.3% average axial strain, which is about the same strain for the concrete failure in compression without fire damage.

3. Application of the structural response evaluation method employed in this paper may provide an estimate on the residual lateral load capacity of structures after fire damage.

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