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IMPACT OF A LOCAL FIRE ON THE RESIDUAL SEISMIC DISPLACEMENT CAPACITY OF THE ENTIRE BUILDING

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Abstract

Most common building fires cause small damage to the structure, usually limited to the fire compartment of origin. These buildings could be reused after inspection by engineers and with a small or local repair. We investigate how the damage from a local fire (in terms of residual deformation) to the fire-exposed elements of a reinforced concrete building could impact the seismic resistance of the unexposed elements, and present a method to evaluate seismic damage and residual displacement capacity of unexposed elements. The structural interaction of the entire building is considered for a local fire limited to a fire-resistance-rated compartment.

Available test data show that when a concrete column is exposed to fire, the column experiences large shortening deformation in addition to loss of load capacity. Because of the interaction with the rest of the building, the load of the column redistributes to other nearby columns and causes permanent deformation of other structural elements. Test results of a full-scale concrete building also showed that thermal expansion of a fire-exposed floor can cause damage to unexposed columns that are connected to the fire-exposed floor. Damage to the unexposed beams due to the column shortening, and to unexposed columns due to the floor thermal expansions, will result in reduction of the seismic resistance of the entire building.

We used available test data, from the National Research Council of Canada (NRC) partly conducted by the author and the Building Research Establishment (BRE) of the UK, to assess damage and acceptance criteria of the ASCE 41-13 standard to determine seismic displacement capacity of the unexposed elements. The method is expected to help engineers for post-fire structural inspection, particularly in areas of high earthquake hazard, to more realistically evaluate the impact of fire damage on the residual seismic resistance of buildings.

Keywords: Seismic damage, seismic capacity; fire damage, thermal expansion; performance-based evaluation.

1. Introduction

In the United States, average annual fire occurrence in medium- and high-rise buildings exceeds 10,000 incidents [1]. In most of these fire incidents, the structures experienced minor to major damage, such as degradation of material properties due to elevated temperatures and damage to structural elements induced by thermal expansion and deformation. Methodologies are available to determine reparability of fire-damaged structures [2]. However, there is only a limited number of studies for assessing the residual seismic resistance of structures especially in areas with high seismicity considering the unexposed elements after fire-exposed elements suffer thermal deformation during a fire. Figure 1 shows damage to unexposed beams and columns induced by thermal deformation (column shortening and floor thermal expansion) of fire-exposed elements.

We present a method to evaluate post-fire seismic displacement capacity of reinforced concrete beams and columns of buildings for a local fire limited to a fire-resistance-rated compartment. Seismic performance acceptance criteria of the ASCE 41-13 standard [3] are used to evaluate seismic damage and displacement capacity of unexposed concrete beams and columns due to thermal deformation of fire-exposed elements (using test data by National Research Council Canada [4, 5] and Building Research Establishment of UK [6]). We use examples in this paper from [4, 5, 6]. As schematically shown in Fig. 1, the unexposed elements that are affected (damaged) by thermal deformations (Δ_x and Δ_y) are the upper beams connected to the columns along the same axis with the fire-exposed columns, and columns connected to the fire-exposed floors/beams. The focus of the proposed method is only on seismic evaluation of unexposed beam and column elements. Fire-exposed elements of the buildings can be evaluated using other avaiable methods [7, 8].

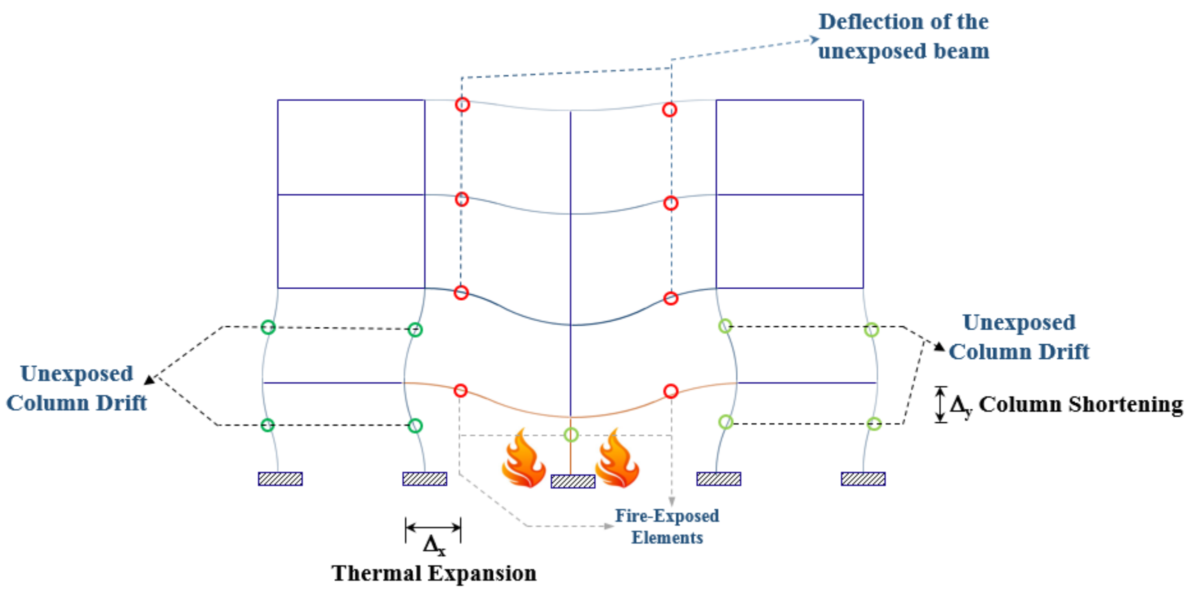

Fig. 1 – Damage to unexposed beams and columns due to column shortening and floor thermal expansion, respectively, during a fire

2. Column Shortening (Δ_y)

We use results of fire resistance tests on three columns conducted by NRCC [4, 5] that include measurements of column shortening (Δ_y). Figure 2 shows the test setup and a picture of the column furnace facility used to perform these three tests. Specification of the column specimens (NRC1, NRC2, and NRC3) and test results are provided in Table 1. The fire loads for these tests included a 1-hour, 2-hour, and 4-hour ASTM E119 standard fire exposure [9]. Table 1 provides also the residual column shortenings (Δ_y) measured after the tests for the column specimens. Column shortening is the total residual axial deformation of the fire-exposed columns measured after the entire column sections are brought back to ambient temperature.

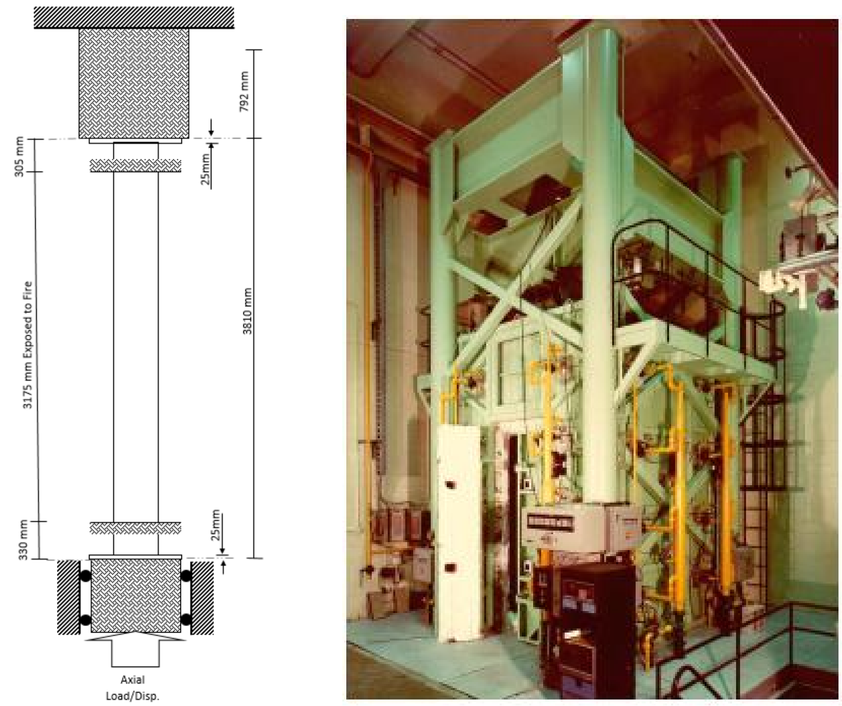

Fig. 2 – NRC Column furnace (on the right) and test set-up for column tests (from Table 1) [5]

Table 1 – Column Test Data [4, 5]

Specimen [Ref]	Fire Exposure Time (hr)	Axial Load during Fire Exposure (kN)	Concrete Strength (MPa)	Initial Concrete Relative Humidity	Main Bars Φ (mm) f_y (MPa)	Ties Φ (mm) @(mm) f_v (MPa)	Residual Axial Capacity (kN)	Total Column Shortening (Δ_y) after Cooling (mm)[2]
NRC1 [4]	1	992	39	87%	4 Φ 25 f_y=444	Φ10 @305 f_y=427	2671	6
NRC2 [4]	2	1022	42	83%	4 Φ 25 f_y=444	Φ10 @305 f_y=427	1987	25
NRC3 [5]	4	Varied[1] (1212-2130)	96	90%	8 Φ 20 f_y=400	Φ10 @225 f_y=400	2200	28*

Columns were tested with fixed-fixed ends condition and under ASTM E119 standard fire curve (ASTM E119, 2011). Columns have total length of 3810 mm, exposed length of 3175 mm and a square cross section (305 mm×305 mm). Clear concrete cover for NRC1 and NRC2 columns is 48 mm and for NRC3 column is 38 mm.

[1]This was a hybrid test where axial load was varied during the test accounting for the response of the remaining structure (obtained using a computer simulation). The applied load on the column at the test start was 2000 kN.

[2]When entire concrete section of the column reached the ambient temperature.

*For this test, Δ_y was measured 6 days after the fire test. Day 6 had 10% increase compared to day 1.

Figure 3 shows test results for the maximum temperature that each part of the concrete cross section of the column specimens achieved during the heating and cooling phase. We determine average maximum exposed column temperature for each column by averaging the temperatures shown in Fig. 3 for each section zone weighted by their associated area. Figure 4 shows the residual axial deformation ratio of the columns (0.002 for NRC1 column, 0.008 for NRC2 column, and 0.009 for NRC3 column in Table 1) versus the average column maximum exposed temperature. Residual axial deformation ratio of each column is the ratio of the column shortening (Δ_y) from Table 1 to the exposed length of the column (3175 mm) shown in Fig. 2.

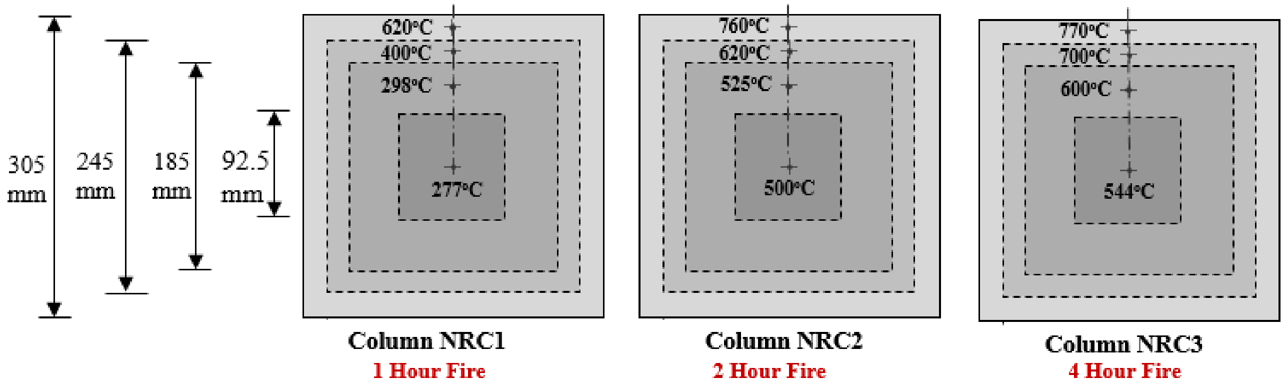

Fig. 3 – Maximum temperatures measured during the heating and cooling phase (the cooling phase was between 20 and 24 hours for these tests) for the column cross sections in Table 1, based on [4, 5]

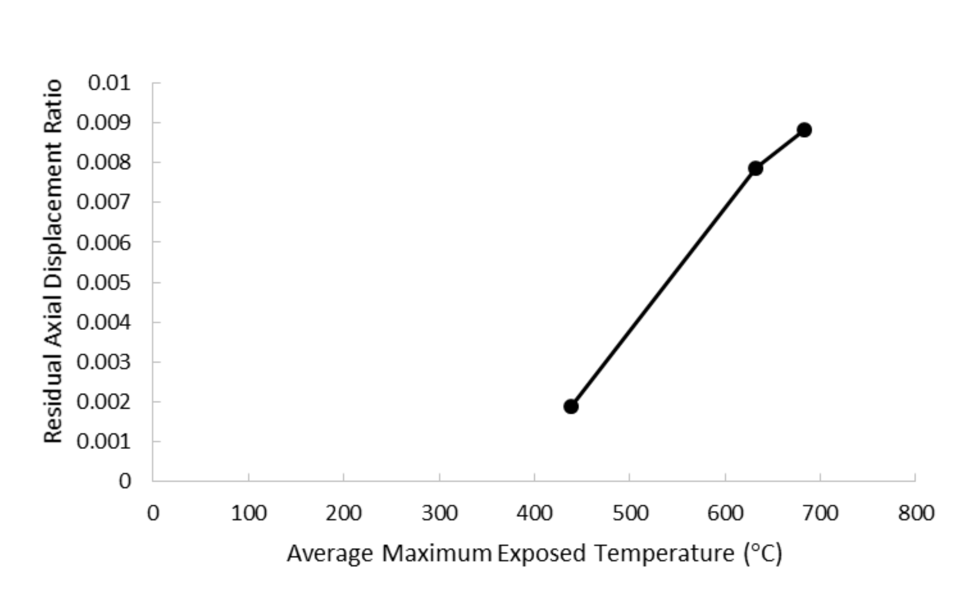

Fig. 4 – Column average maximum exposed temperature versus residual axial deformation ratio of columns (Table 1), based on [4, 5]

3. Floor Thermal Expansion (Δ_x)

We used test results from a full-scale 7-story concrete building [6] constructed at the Building Research Establishment (BRE) Laboratory in Cardington, UK, to estimate floor thermal expansion. The building has 3 by 4 bays (each 7.5 m) designed as a commercial office building based on Eurocode 2 [5, 10]. Figure 5 shows plan and elevation of the building specimen and the locations of fire compartments. Each floor slab is nominally 250 mm thick with normal weight concrete, 28 days strength of 61 MPa, and designed as a flat slab supported by 400 mm square internal columns and 400 mm by 250 mm external columns as shown in Fig. 5. Columns were made of concrete with 28 days strength of 103 MPa. The average concrete cover was 41 mm for columns and 23 mm for slabs. Bars of 12 mm and 16 mm diameter with various spacing (spacing is not reported by [6]) were used in floor slabs. The fire exposure for the test was equivalent to a 1-hour design fire [6]. Figure 5 shows residual horizontal thermal expansion of the 1st floor after the fire test.

The results indicate that the floor experienced lateral deformations between 20 mm and 67 mm. For a floor span (distance between two columns) of 7500 mm, thermal expansion ratios (lateral deformation divided by length of each floor span, 7500 mm) between 0.0027 and 0.009 are calculated for the floor. Note that floor thermal expansion could reach much larger values during a longer fire exposure. For example, during the US Military Personal Record Center, Overland, MO, fire of 1973 [11], some of the columns of the 6-story reinforced concrete building experienced residual lateral deformation of 600 mm leading to their complete loss of load capacity. However, this large thermal deformation was achieved after a very long fire exposure (20 hours to burn out). The results in Fig. 5 are more reasonably expected for a typical 1-hour fire exposure.

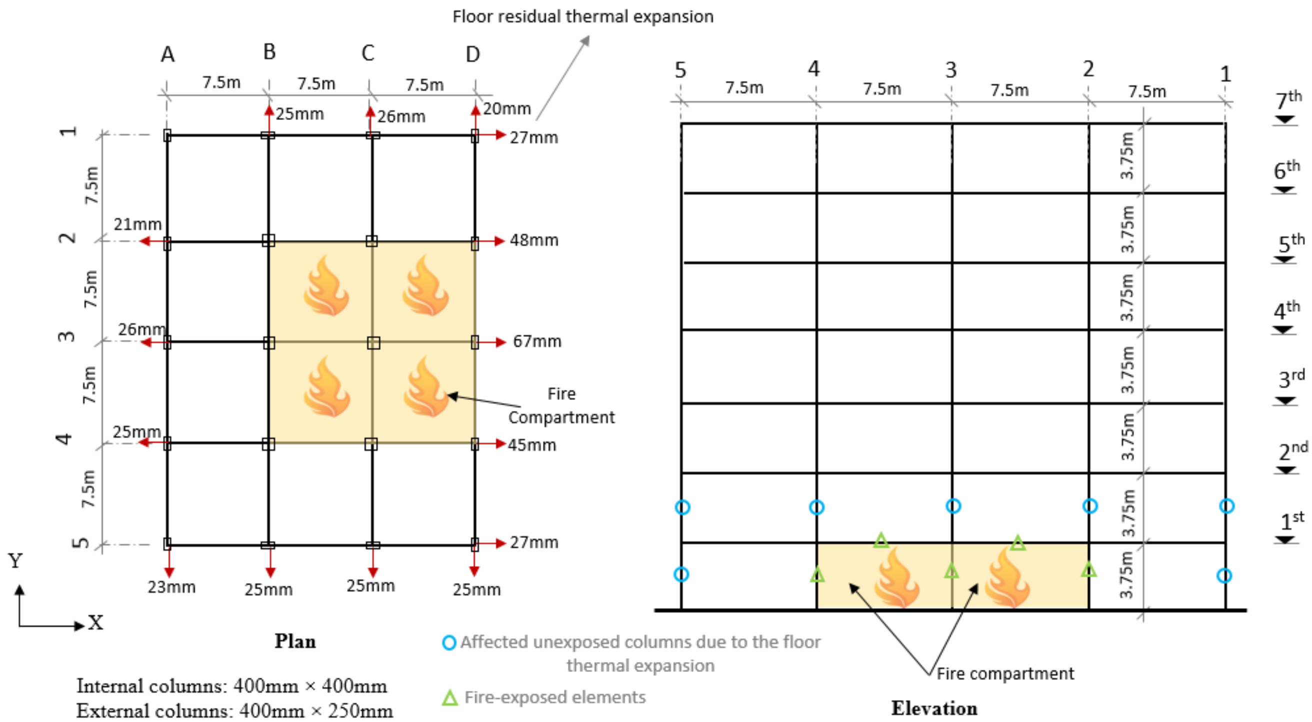

Fig. 5 – Plan (left) and elevation (right) of the building specimen and location of fire compartment, based on [6]. Floor residual thermal expansions are shown on the perimeter of the left panel. Column shortening data are not available for this test.

3. Damage and Seismic Resistance Evaluation

We used the seismic performance acceptance criteria of the ASCE 41-13 standard [3] to evaluate damage to the unexposed reinforced concrete beams and columns and determine their residual seismic displacement capacity.

3.1 Beams

Columns NRC1 and NRC2 in Table 1 were tested as single elements. Column NRC3 was tested using a hybrid method [6] and included design and analysis of a 6-story concrete building (Fig. 6) for the numerical part of the hybrid test. For the hybrid test, the column was tested physically in a furnace (shown in Fig. 2) and the rest of the frame was modeled using finite element analysis software (SAFIR [12]) with real-time structural interaction at the column top connection. For our example, we assume that the frame elements (columns and beams) used for NRC3 test (Fig. 6) can be also representative of the frame elements for NRC1 and NRC2 columns. Therefore, we evaluate damage to the unexposed beams in Fig. 6 due to column shortenings for each of the three column test results in Table 1.

We follow the steps below to evaluate damage and seismic resistance for the beams:

1- Obtain column shortening Δ_y from Table 1 and rotation for each beam R (beam damage) as explained in Table 2.

2- For each beam in Table 2 (also shown in Fig. 6), calculate parameters required in Chapter 10 of ASCE 41-13 [3] for determining acceptance criteria for reinforced concrete beams including shear force (V) and shear ratio $V/(b_w d\sqrt{f_c})$, hoop spacing (S), flexural depth (d), and shear resistance provided by hoops (V_s).

3- Identify performance acceptance criteria R_{ac} of each beam for Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels from ASCE 41-13 based on the parameters determined in step 2 (Table 2).

4- Calculate a new performance acceptance criterion $R_{ac\_new}= R_{ac}- R$ for each performance level (we subtracted the residual deformation from R_{ac} to obtain the remaining deformation for the element to reach the corresponding performance level). For instance, for the main beam in Table 2, R=0.0012 (due to a 1-hour fire) with original R_{ac}=0.005 for IO level. Therefore, the new IO level acceptance criterion is $R_{ac\_new}= R_{ac}- R= 0.0038$ (the element has only 0.0038 deformation capacity remaining to meet the IO level).

5- Calculate percent residual seismic displacement capacity, equal to $100*(R_{ac\_new})/ (R_{ac})$

The results for all beams are provided in Tables 2 and 3. We assumed conservatively that column shortenings are entirely transferred to beams above the fire compartments (Fig. 1) with no reduction. However, due to structural interaction and load sharing, deflections of the beams above fire compartments are usually less than the column shortenings. For beams in higher floors (e.g., roof level), such deflections could be even smaller. A full finite element analysis of the frame in cold conditions can be performed to estimate the beam deflections more efficiently by eliminating the exposed column and applying the column shortening as the boundary condition of the frame.

3.2 Columns

Similar to the approach we used for beams, damage R and new performance acceptance criterion R_{ac_new} can be determined for floor thermal expansion using the ASCE 41-13 acceptance criteria for columns. The following steps are employed to determine R and R_{ac_new} for columns in the Cardington test (Fig. 5) in Table 4:

1- Obtain floor thermal expansions Δ_x from Fig. 5 and rotation for each column R as explained in Table 4.

2- For each column, determine parameters required in Chapter 10 of ASCE 41-13 [3] for determining acceptance performance criteria for reinforced concrete columns, including the axial load P, shear ratio $V/(b_w d\sqrt{f_c})$, hoop reinforcement ratio $A_v/(b_w S)$, and ratio of plastic shear demand to shear capacity V_p/V_o.

3- Identify performance acceptance criterion R_{ac} according to ASC 41-13 specifications and based on the parameters determined in step 2 (Table 4).

4- Calculate the new performance acceptance criterion $R_{ac\_new}= R_{ac}- R$ for each performance level. For instance, for the corner column D1 in Table 4, R=0.0072 with original R_{ac}=0.01 for LS level. Therefore, the new LS level acceptance criterion is $R_{ac\_new}= R_{ac}- R= 0.0028$.

5- Calculate percent residual seismic displacement capacity, equal to $100*(R_{ac\_new})/ (R_{ac})$

Note that for the D1 column example, R exceeds R_{ac} at the IO level. This means that the columns cannot provide IO level performance due to the fire damage and therefore it requires repair. The results for all columns are provided in Tables 4 and 5. The results show that all the column examples in Table 4 cannot satisfy the performance acceptance criteria for IO level. For two columns (D3 and D2), R exceeds even the CP level. Comparing results in Table 3 and Table 5, reduction in residual seismic displacement capacity is relatively higher for columns than for beams. This indicates that columns are more affected by thermal deformations and require more attention during the post-fire inspection.

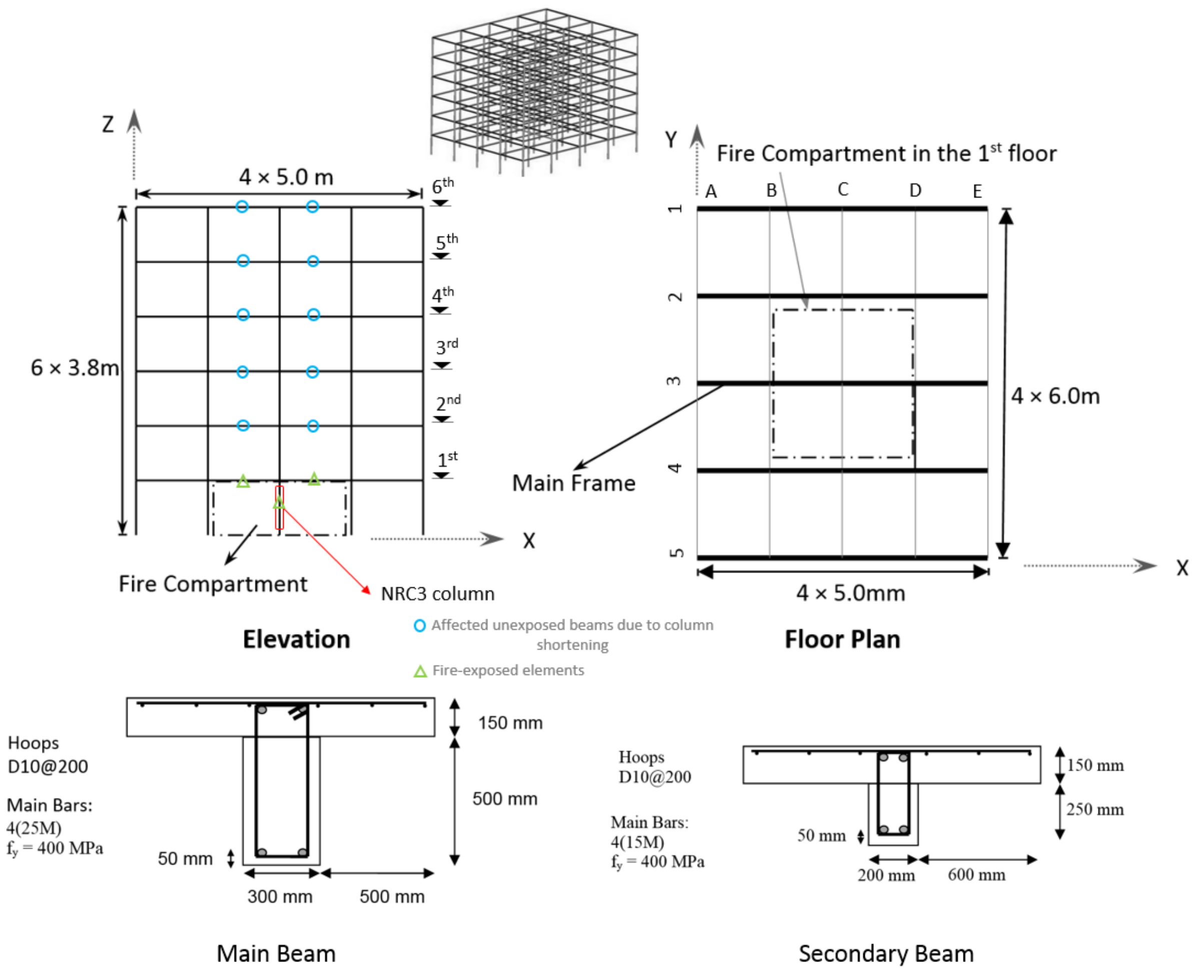

Fig. 6 – Elevation and floor plans (top panel) and cross sections and properties of the main and secondary beams (bottom panel) of NRC3 hybrid test. [5]

Table 2 – Beam parameters and ASCE 41-13 acceptance criteria [3]

Affected Beam	$\frac{V}{b_w d\sqrt{f'_c}}$	S (mm)	d/3 (mm)	V_s (kN)	0.75V (kN)	Test Data		ASCE 41 R_{ac} and Performance Level		
						T (hr)	R	IO	LS	CP
Main Beam	0.12	200	200	72	167	1	0.0012	0.005	0.02	0.03
						2	0.0050			
						4	0.0056			
Secondary Beam	0.24	200	116	42	169	1	0.0010			
						2	0.0042			
						4	0.0047			

T: Fire exposure time in hours for columns in Table 1.
R: Beam rotation= ratio of column shortening Δ_y from Table 1 to length of the beam L (L=5000 mm for main beams and L=6000 mm for secondary beams).
R_{ac}: Acceptance of beam rotation for Immediate Occupancy (IO), Life Safety (LS), and collapse prevention (CP) performance levels.
S: hoop spacing.
d: beam flexural depth.
V_s: shear force capacity provided by shear reinforcement.
V: maximum shear force in beam due to applied load. We used seismic weight provided by [7] and assume a seismic base share factor of 0.2 (a typical value; judgment-based) and apply a simple method (portal method for rough moment frame design calculations) to obtain shear force for the beams that provide maximum V.
For these beams, tension reinforcement ρ and compression reinforcement ρ′ are the same, therefore ρ-ρ′=0 (requited for selection of acceptance criteria in ASCE 41)

Table 3 – Acceptance criteria and residual seismic displacement capacity for unexposed beams in Table 2 [3]

Affected Beam	T (hr)	R	ASCE 41 R_{ac_New} and Performance Level			% of Residual Seismic Displacement Capacity*		
			IO	LS	CP	IO	LS	CP
Main Beam	1	0.0012	0.004	0.019	0.029	76%	94%	96%
	2	0.0050	0.000	0.015	0.025	0	75%	83%
	4	0.0056	0.000	0.014	0.024	0	72%	81%
Secondary Beam	1	0.0010	0.004	0.019	0.029	80%	95%	97%
	2	0.0042	0.001	0.016	0.026	17%	79%	86%
	4	0.0047	0.000	0.015	0.025	7%	77%	84%

Table 4 – Column sections and load properties and ASCE 41-13 performance acceptance criteria [3]. For D3, D2 and B2, the affected unexposed columns are located between 2nd and 3rd floor

Affected Column	Column	Direction	$P/(A_g f'_c)$	$A_v/(b_w S)$	$\frac{V}{b_w d\sqrt{f'_c}}$	Test Data		ASCE 41 R_{ac} and Performance Level		
						Δ_x (mm)	R	IO	LS	CP
Corner	D1	X	0.056	0.002	0.144	27	0.0072	0.005	0.01	0.012
		Y		0.003	0.127	20	0.0053			
	A5	Y				23	0.0061			
Edge	D3	X		0.002	0.144	67	0.0179			
	D2	X				48	0.0128			
	C1	Y				26	0.0069			
Middle	B2*	X		0.005	0.127	21	0.0056			
		Y				25	0.0067			

* For this column, test data did not provide direct measurement. We assume displacement measured at the edge columns: A2 for X and B1 for Y (assuming the same displacements transferred from B2 through the unexposed beams to A2 or B1).

P: axial load of column (kN), A_g: gross area of column section (mm^2), f'_c: concrete compression strength (MPa), A_v: area of shear reinforcement (mm^2), b_w: web width of the column (mm), S: shear reinforcement spacing (mm), d: flexural depth (mm), V: shear force on the section due to lateral load in kN (we assumed 20% of the vertical load). R_{ac}: Acceptance of column rotation (drift ratio) for Immediate Occupancy (IO), Life Safety (LS), and collapse prevention (CP) performance levels and R: column rotation = ratio of floor expansion Δ_x divided by length of the column L (L=3750mm).

For all the columns, we obtained $1>V_p/V_o> 0.6$ (with hook-and-bob shear reinforcement) to select R_{ac} from ASCE 41, where V_p is plastic shear demand on the column and V_o is shear strength of the column according to the ASCE 41 definitions and requirements.

Table 5 – Acceptance criteria and residual seismic displacement capacity for unexposed columns in Table 4 [3]

Beam	Column		R	ASCE 41 R_{ac_new} and Performance Level			% of Residual Seismic Displacement Capacity*		
				IO	LS	CP	IO	LS	CP
Corner	D1	X	0.0072	0	0.003	0.005	0	28%	40%
		Y	0.0053	0	0.005	0.007	0	47%	56%
	A5	Y	0.0061	0	0.004	0.006	0	39%	49%
Edge	D3	X	0.0179	0	0	0	0	0	0
	D2	X	0.0128	0	0	0	0	0	0
	C1	Y	0.0069	0	0.003	0.005	0	31%	43%
Middle	B2	X	0.0056	0	0.004	0.006	0	44%	53%
		Y	0.0067	0	0.003	0.005	0	33%	44%

3.3 Entire Building

If fire-exposed elements cannot be repaired or replaced, their residual seismic resistance capacity should be also incorporated in the estimation of the seismic resistance of the entire building; that analysis is beyond the scope of this study (but see [5], [7] and [8]).

Seismic displacement capacity of the entire building can be simply and conservatively estimated as the minimum seismic displacement capacity of the unexposed beams and columns (assuming that fire-exposed elements are repaired to their original design capacity). For example, for the Cardington building (Fig. 5 and Table 5), the unexposed columns D2 and D3 provided zero seismic displacement capacity for Collapse Prevention (CP) level. Therefore, zero seismic displacement capacity for CP is assigned to the Cardington building even after the fire-exposed elements are repaired or replaced.

Post-fire inspection should include a plan to retrofit the damaged unexposed elements and ensure that the entire building can provide the required seismic performance level after a fire.

4. Conclusions

An approach was introduced to evaluate seismic displacement capacity of unexposed beams and columns of reinforced concrete buildings after a fire. We conclude:

- NRC concrete column tests showed residual axial deformations (column shortening divided by the exposed column length) of 0.002, 0.008, and 0.009 for 1-hour, 2-hour, and 4-hour fire test, respectively.
- Cardington concrete building test data indicated floor thermal expansion ratios (floor thermal expansion divided by length of beam) between 0.0027 and 0.009 after a one-hour fire.
- Seismic damage evaluation results showed higher loss of seismic displacement capacity due to thermal deformation for columns than for beams. Columns require more attention during post-fire inspection.
- The Cardington building example showed that, even if the fire-exposed elements are repaired to their original design capacity, seismic retrofitting of the unexposed elements is required to ensure the entire building can provide its original seismic performance level.

5. Acknowledgements

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