Effects of Localised or Non-Uniform Heating on Reinforced Concrete Columns

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Title: **Effects of Localised or Non-Uniform Heating on Reinforced Concrete Columns**

Authors (names are for example only): Jamie Maclean
Vadims Goremikins
Luke Bisby
Tim Stratford

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Dr. Maria E. Moreyra Garlock
Associate Professor
Princeton University

E-mail: mgarlock@princeton.edu

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(FIRST PAGE OF ARTICLE)

ABSTRACT

This paper describes a research project being conducted to shed light on the response of concrete columns to non-uniform heating, and on the ability to credibly model the response of realistic concrete structures during non-standard fires. A series of 48 one-third scale reinforced concrete (RC) columns are being tested under sustained eccentric axial loading, and exposed to a localized radiant heat source to observe their response, to provide validation data for full-frame structural fire modelling, and to determine the "damage" caused by non-standard fire exposures. All 48 columns are being loaded eccentrically and exposed to a constant incident heat flux at their mid height by means of a propane-fired radiant panel. This paper describes the project, including; the objectives and expected outputs of the project, the experimental test series which is underway, and initial *a priori* modelling results obtained using the commercially available Finite Element Software ATENA, which will be compared against the results of the experiments once these data are available.

INTRODUCTION AND MOTIVATION

In addition to life-safety concerns during building fires, uncontrolled fires within buildings have the potential to cause extensive structural damage. Current design guidance for structures in fire, however, focuses almost exclusively on the life safety of the occupants within buildings. This is generally achieved by specifying a defined fire resistance period during which structural integrity must be maintained and fire spread must be prevented, to ensure that the building's egress routes are not compromised until all occupants have escaped from the building and fire-fighting operations have been completed. Designers are not typically required to explicitly consider the residual, post-fire effects of fires on structures – nor the associated direct and indirect repair costs associated with fire induced damage to the building itself. Only particularly enlightened consultants or clients will demand that fire damage to the building be explicitly considered during design.

Historically concrete buildings have performed relatively well in fires, and concrete structures can often be repaired after a fire event [1]. It is, however, challenging to predict the response of concrete structures during heating, and even more challenging to quantify their residual structural performance after cooling. Little validated computational modelling work is available in the literature, particularly for non-standard fire exposures. This paucity of information makes holistic, performance-based, full frame design and analysis of concrete structures difficult, since the total damage (and thus true costs) associated with uncontrolled fires cannot currently be quantified.

To better understand how concrete structures may react in fires, both non-standard fire tests on concrete structural elements and validated numerical modelling techniques are essential. In recent years a number of studies have presented attempts to computationally model the behavior of concrete structures at high temperature, as well as during the cooling phase and residually [2-4]. Such studies have required the development of material and numerical models to predict the response of concrete under load, both during and after elevated temperature [3-4]. These models may offer engineers additional analysis capability during design, however further validation work is needed before they can be confidently applied on a widespread basis as part of the structural fire design process.

This paper presents a project aiming to better understanding the thermal, and more importantly structural, response of concrete columns subjected to loading and non-standard, non-uniform fire conditions. Both numerical and experimental studies are being undertaken, with the objective of increasing the pool of data available from tests on concrete structures under non-standard arrangements. Such data are essential for comparisons against computational model predictions. A total of 48 square one-third scale reinforced concrete (RC) columns have been constructed. These are being exposed to localized heating at their mid-height using a constant incident radiant heat flux from propane-fired radiant panels, and subjected to varying degrees of sustained axial loading eccentricity. The data gathered during the transient thermal experiments includes: the internal temperatures through the depth of the columns using a total of 18 thermocouples; vertical and lateral deflections; and strains using digital image correlation.

The parameters being investigated in the test series include: the magnitude of loading; the degree of eccentricity of loading; the number of sides heated (1 or 2); the severity of the incident heat flux; and the concrete compressive strength. In addition to the experimental test procedure, this paper gives results of *a priori* finite element (FE) modeling undertaken in accordance with thermal and mechanical property relationships suggested by Eurocode 2 [5]. All FE models have been developed before the tests are carried out to ensure that the authors' modelling ability is genuinely interrogated without temptation to vary model input parameters to match the experimental results (as would occur in an engineering design office).

The vast majority of the available research investigating the performance of concrete columns subjected to elevated temperatures has concentrated on the performance of columns when subjected to uniform heating over the full (or almost full) length of the column and over all four sides simultaneously [3]. In addition, the columns are almost always exposed to the ISO 834 standard fire curve (or similar) within a fire testing furnace [2-3]. Furnace testing, although useful for comparative testing of different columns and materials, is not very useful to observe or quantify the precise thermal exposure and structural response, which makes use of such tests problematic for validation of structural fire analysis software. To address this shortcoming, radiant panels are being used in the current study to induce a constant incident heat flux to the columns; this can be directly controlled, measured, and used as a well-defined thermal boundary condition for computational models being developed and validated on the basis of the experiments.

It is well documented that concrete is sensitive to elevated temperatures [1-7]. In real fires, when a concrete column is subjected to heating on one side only, thermal gradients will be induced within. This will in turn result in the development of thermal deformations and thermal stresses within the column, resulting in a section with differing strength and stiffness over its cross-section. The effect of these factors is being investigated by exposing the columns to different severities of heat fluxes on one face only. Standard heating, e.g. ISO 834 [6], is not being considered, since the goal is to understand response and validate modelling capability, rather than simply to provide simplistic fire resistance ratings, which do not help to evaluate the time history of structural fire response or the structural damage that might be expected after a fire. It is hoped that this will help inform post-fire analysis of RC columns for use by engineers and insurers to provide property protection and quantified probabilistic structural fire engineering design in the future.

TEST SPECIMENS AND PROCEDURES

Columns and Instrumentation

Figure 1 gives section and elevation schematics of the design of both the columns and the testing frame to be used. Forty-eight geometrically identical reinforced concrete columns have been cast in two sets of twenty-four. All of the columns are 150mm x 150mm in cross-section and 1400mm long, longitudinally reinforced with four 10mm diameter deformed reinforcing bars and 5mm diameter deformed steel ties spaced at 140mm on center. The design of the columns follows Eurocode 2 [5]. One set of concrete columns is lower strength (32MPa at 28 days) and second set was cast using higher strength concrete (52MPa at 28 days). It is noteworthy that both mixes include 2kg/m^3 of polypropylene (PP) fibers for spalling mitigation during the tests.

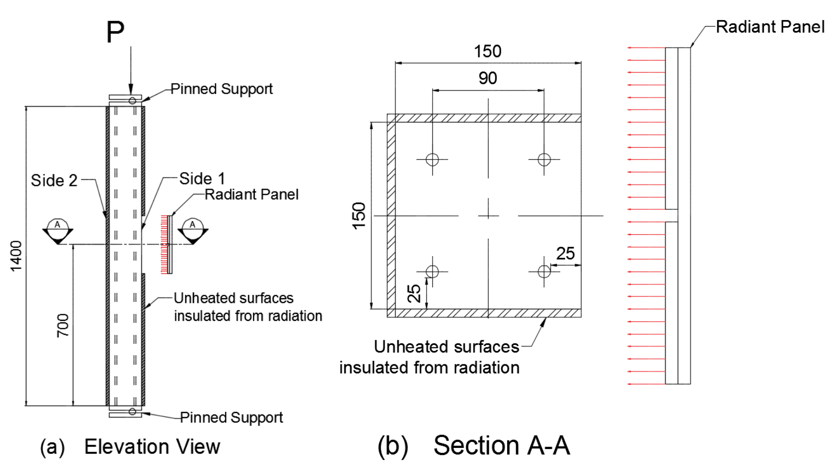

Figure 1 – Detailed of reinforced concrete column design, heating regime, and testing configuration

The test setup has been specifically designed such that the thermal exposures experienced by the columns can be accurately controlled, compared, and modelled. A total of 18 thermocouples have been cast into each concrete column at mid-height, 900mm (the top of the radiant panel) and 950mm (50mm above the top of the radiant panel) from the base of the column respectively. The thermocouples are being used to quantify the thermal exposure on the surface of the columns and to validate the thermal models being developed in conjunction with the tests. The samples have been cured unsealed and open to the atmosphere at 50% relative humidity and 23^{o}C, for a period of 6 months.

Experimental Procedure

The test setup is illustrated in Figure 1. Tests have four distinct stages:

1. Columns are placed in the testing frame and loaded to the desired sustained load. This load is maintained constant during steps 2 and 3.
2. Columns are exposed to a localized incident radiant heat flux over a defined area at their mid heights. The incident heat flux is applied instantaneously and maintained constant for 30 or 60 minutes.
3. After 30 or 60 minutes of heating, provided the column has not failed, the heating is removed and the column is allowed to cool to ambient temperature, with its mechanical response during cooling also monitored.
4. The sample is tested residually to failure to determine its residual response.

This test procedure allows the entire sequence of elevated temperature heating, cooling, and potentially post-heating residual response, to be interrogated and modelled. The parameters being investigated include:

1. the strength of the concrete (32MPa or 52MPa);
2. the number of sides heated (front face only, or both front and back faces);
3. the severity of the thermal exposure (60kW/m^2 or 80kW/m^2);
4. the eccentricity of the sustained axial load (5mm or 25mm); and
5. the magnitude of the load during heating (20% or 60% ambient capacity).

Table 1 shows the full testing matrix for the current study, along with the associated parameters being varied.

A PRIORI FINITE ELEMENT MODELLING

Model Description

An *a priori* FE model of the column tests was developed using the commercial FE code Atena 5 Science with the GID 12 preprocessor. Concrete was modelled using 3D brick elements applying a nonlinear fracture-plastic material model. The concrete model includes post-cracking and post-crushing softening phases. Steel bars and ties (see Figure 2a) were modelled using 1D bar elements. The steel base plates and rollers (Figure 2b), which were used during testing to provide well-defined pinned-pinned column end conditions, were included in the FE models to capture the precise load application used in the tests, rather than the load application that might occur in a real building. The material models used account for changes in the mechanical and thermal properties of the steel and concrete due to increases of temperature; this is done in accordance with the Structural Eurocodes [6].

TABLE I. REINFORCED CONCRETE COLUMN TEST MATRIX

No.	Conc. Strength f_c (MPa)	Side Heated (see Fig. 1)	Heat Flux (kW/m^2)	Duration (min)	Eccentricity (mm)	Load (% amb. cap.)
1	32	1	80	30	5mm	20%
2	32	1	80	30	5mm	60%
3	32	1	80	30	25mm	60%
4	32	1	80	60	5mm	20%
5	32	1	80	60	5mm	60%
6	32	1	80	60	25mm	60%
7	32	1	60	60	5mm	20%
8	32	1	60	60	5mm	60%
9	32	1	60	60	25mm	60%
10	32	2	80	60	25mm	60%
11	32	2	60	60	5mm	60%
12	32	2	60	60	25mm	60%
13	32	1&2	80	30	5mm	60%
14	32	1&2	80	30	25mm	60%
15	32	1&2	80	60	5mm	60%
16	32	1&2	80	60	25mm	60%
17	32	1&2	60	60	5mm	60%
18	32	1&2	60	60	25mm	60%
19	52	1	80	30	5mm	20%
20	52	1	80	30	5mm	60%
21	52	1	80	30	25mm	60%
22	52	1	80	60	5mm	20%
23	52	1	80	60	5mm	60%
24	52	1	80	60	25mm	60%
25	52	1	60	60	5mm	20%
26	52	1	60	60	5mm	60%
27	52	1	60	60	25mm	60%
28	52	2	80	60	25mm	60%
29	52	2	60	60	5mm	60%
30	52	2	60	60	25mm	60%
31	52	1&2	80	30	5mm	60%
32	52	1&2	80	30	25mm	60%
33	52	1&2	80	60	5mm	60%
34	52	1&2	80	60	25mm	60%
35	52	1&2	60	60	5mm	60%
36	52	1&2	60	60	25mm	60%
37	32	N/A	N/A	N/A	5mm	100%
38	32	N/A	N/A	N/A	5mm	100%
39	32	N/A	N/A	N/A	25mm	100%
40	32	N/A	N/A	N/A	25mm	100%
41	52	N/A	N/A	N/A	5mm	100%
42	52	N/A	N/A	N/A	5mm	100%
43	52	N/A	N/A	N/A	25mm	100%
44	52	N/A	N/A	N/A	25mm	100%
45	52	TBC	TBC	TBC	TBC	TBC
46	52	TBC	TBC	TBC	TBC	TBC
47	52	TBC	TBC	TBC	TBC	TBC
48	52	TBC	TBC	TBC	TBC	TBC

(a) (b)

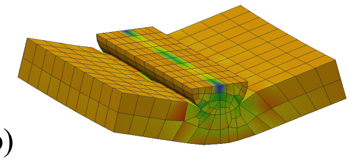

Figure 2 – Schematics showing (a) steel bars and ties and (b) steel base plate and roller end conditions applied during FE modelling (exaggerated deformations)

The analyses were carried in two steps. In the first step the three-dimensional thermal fields were calculated within the columns. Since it is not possible to apply an incident surface heat flux in Atena, as will be applied by the radiant panels during the experiments, the incident heat flux was converted to a convection and radiation boundary condition. In the second step the mechanical analyses were performed with the thermal fields taken as inputs. The columns were loaded during the analyses first by applying the sustained mechanical loading; new thermal fields were then applied in each subsequent time step, with the mechanical loads held constant.

Modelling Results

A range of the testing scenarios listed in Table 1 has been modeled computationally. However, it should be noted that concrete strengths of 40MPa and 80MPa were used since these were the specified concrete strengths specified during casting of the test columns (these were not achieved and the models now require refinement for comparison against the tests, when performed). Figure 2 shows the predicted ambient response of the columns when loaded monotonically to failure. Using this response as a baseline, the response of the columns has been compared when subjected to a localized heat heating and sustained eccentric axial load.

The predicted capacity of the columns is roughly 1400kN and 1000kN for the C80 concrete for 5mm and 25mm eccentric loads, respectively, whereas for C40 concrete it is 1100kN and 700kN, respectively. As expected, lower concrete strength results in predicted decreases in axial load-bearing capacity, and additional loading eccentricity results in considerable increases in predicted lateral deflections.

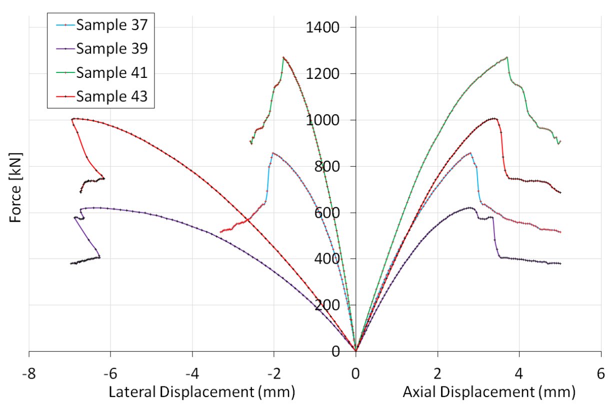

Figure 2 – Selected predicted RC column responses at ambient temperature (see Table 1)

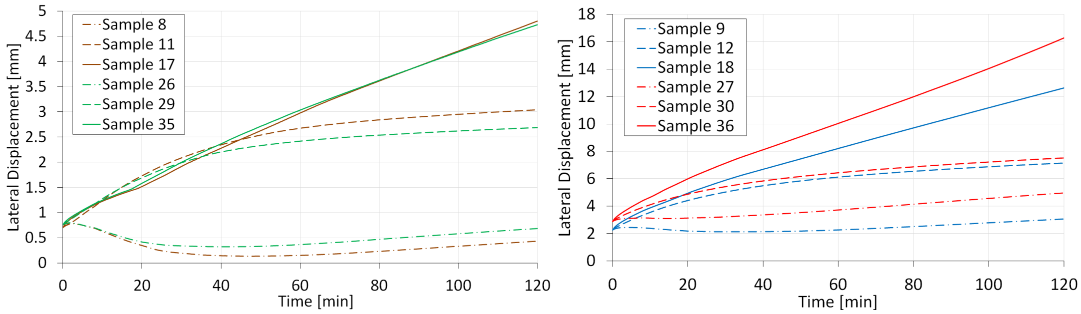

Figure 3 – Selected predicted lateral displacements of columns with time of exposure to 60kW/m^2 of localized incident heat flux (see Figure 1 and Table 1); effect of sides heated; (a) 5 mm eccentricity; (b) 25 mm eccentricity

Figure 3 shows the predicted response of selected columns listed in Table 1, subjected to a localized incident heat flux of 60kW/m^2 for 120 minutes. All columns shown were loaded to 60% of their ambient capacity. Figure 3 shows that, when the 'tension' (i.e. least compressed) face of the eccentrically loaded columns is subjected to heating, horizontal deflections in the column are predicted to increase continuously during heating. Obviously this can be attributed to thermal expansion of the concrete on the tension face (thermal bowing), which increases lateral deflections and exacerbates secondary moments. If, however, the 'compression' face of the eccentrically loaded column is heated, the horizontal deflections in the column initially decrease for both 5mm and 25mm eccentricities; this is later followed by increasing deflections with further heating, as a consequence of deterioration of mechanical properties of concrete on the more highly compressed face. This confirms that thermal expansion of the concrete plays a key role in the response of asymmetrically heated RC columns, particularly in the early stages of heating. In the later stages of heating, however, the reduced mechanical properties of the concrete dominate the response.

Columns heated on both faces initially experience similar deflections to columns heated on the tension face only; however they experience greater deflections in the later stages of heating, again once reduced mechanical properties begin to dominate the response. Higher eccentricities are predicted to lead to an overall response which is more dominated by material property reductions, particularly under two-sided heating.

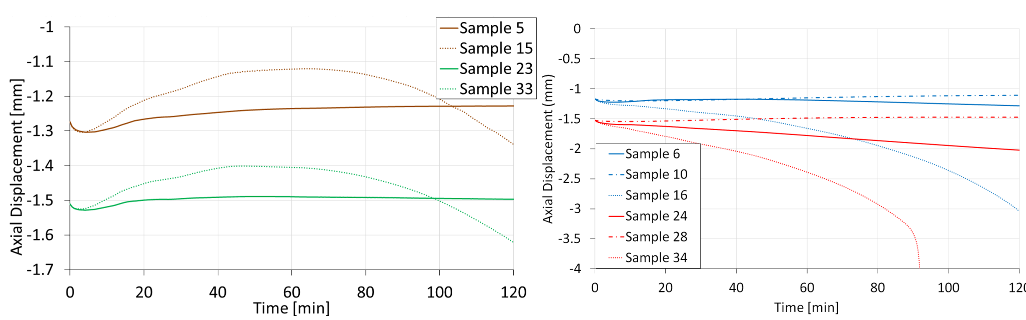

Figure 4 – Predicted axial displacements of selected columns subjected to an 80kW/m^2 localized incident heat flux, heated on one or two faces (see Table 1); (a) 5 mm eccentricity; (b) 25 mm eccentricity

Figure 4 shows the predicted vertical displacements of the columns subject to an 80kW/m^2 heat flux loaded to 60%. It can be seen that, as the concrete temperatures increase, the vertical displacements increase until failure. However, where the load eccentricity is small (i.e. 5mm), similar to the horizontal deflections, the vertical displacements initially decrease as a result of thermal expansion. This suggests that the lower the load eccentricity, the larger relative influence thermal expansion will have on the structural response of the columns (and consequently, the overall structural frame in a real concrete building). The testing that is currently underway (see Table 1) will provide a unique opportunity to compare the above (and other) computational predictions against ad-hoc non-standard fire testing concrete columns.

CONCLUSIONS

Based on the *a priori* modelling completed for the RC columns described in Table 1 of this paper, the response of the concrete columns is seen to depend on the thermal gradient through the depth of the column and on the specific loading scenario applied. This in turn places a higher degree of importance on the location and time history of the fire itself. Testing and analysis of RC structural elements under standard fire testing procedures is therefore not necessarily conservative for assessing either the element response or the full structural response to heating. Additional research work, currently underway, is required to support analysis and design of concrete columns under non-standard heating regimes, and to validate models' ability to predict "real" structural behavior in fire. Based upon the simulations completed, the following conclusions can be drawn on the response of RC columns to localized heating:

1. As the thermal wave passes through concrete columns exposed to elevated temperatures non-uniformly, the structural response is predicted to depend considerably on the thermal gradients and, in turn, the fire severity and its and location.
2. When the most compressed column face is exposed to elevated temperatures, thermal expansion of the concrete opposes the thermal bowing of the column, and results in decreased horizontal displacements during the initial heating stages. The opposite is predicted if the least compressed face is heated.
3. The lower the first order moment (i.e. load eccentricity) applied to a column, the more influence thermal expansion has on the response in the initial stages of heating.

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