POST-FIRE EARTHQUAKE RESISTANCE OF REINFORCED CONCRETE STRUCTURAL WALLS

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ABSTRACT: A potential but infrequently studied hazard is the sequential occurrence of earthquakes and fires. Fire hazards following an earthquake can be significant due to increased likelihood of fires igniting increased demands on firefighting resources, and potential obstacles to timely response. Increased ignitions and larger burn times can have significant structural impacts on reinforced concrete structures (RC) which are usually considered to have superior performance in a fire. The impact of this fire induced structural damage on the lateral load resistance of RC structures, particularly RC structural walls, is not well understood, but may be critical in the event of aftershocks and/or future earthquakes. Given the severity of the consequences of reduced lateral load resistance, it is important for engineers to better understand fire-earthquake hazards in RC walls. This paper presents preliminary results of a numerical study to investigate the impact of fire damage on the lateral load resistance of flexure-control RC structural walls. Results indicate that fire damage decreases the load-bearing capacity and the stiffness of RC walls under reversed-cyclic loads.

1. Introduction

Reinforced concrete (RC) structural walls are structural elements frequently used in buildings to provide lateral stiffness and strength against seismic actions. Their seismic behavior at room temperature has been widely explored. Due to the non-combustibility and low thermal conductivity of concrete, RC structural walls are also well-known for their good fire performance, often working as fire walls to supress the spread of a fire in a building. However, if concrete and reinforcing steel are damaged in a fire, the seismic resistance of RC walls may be decreased significantly.

Post-fire material tests (Nassif, 2006; Chang et al, 2006; Harada, 1966) have shown that the mechanical properties of concrete do not recover when cooled down, degrading even further in the first few months. The mechanical properties of reinforcing steel also degrade after exposure to elevated temperature above 600^{o}C (Neves et al., 1996; Kirby et al., 1986). Previous research about the post-fire performance of RC structural members (Cheng and Chang et al, 2009; Lie et al., 1986 and 1988; Mostafaei et al., 2009; Xiao et al., 2008; El-Hawary et al., 1997; El-Hawary et al., 1996; Franssen and Kodur, 2001) have identified the adverse effect of fire damage on the performance of RC structural members.

Compared to the post-fire performance of other RC structural members, research about the post-fire performance of RC structural walls, particularly under seismic loading, is very limited. Two test programs have focused on the post-fire seismic performance of RC walls (Xiao et al, 2004; Liu, 2010); however, these tests focused on RC walls with relatively low aspect ratios and small thicknesses. It remains unknown whether fire damage will significantly decrease the seismic performance of flexure-controlled RC walls with wall characteristic required by seismic actions.

This paper presents preliminary results of a numerical study to investigate the impact of fire damage on the lateral load resistance of flexure-controlled RC structural walls. Walls which have been tested under reversed-cyclic loads at room temperature and failed in flexural pattern were selected for the preliminary study. All of those walls were subject to heating-cooling cycle first and then subject to lateral reverse-cyclic loads in the numerical analysis. The load-bearing capacity, stiffness, drift capacity and ductility of walls before and after fire exposure were investigated and compared.

2. Methodology

2.1. Description of the simulation method

SAFIR (Franssen, 2011) was used for the heat transfer analysis of wall sections while OpenSees (Mazzoni et al., 2006) was used for the seismic analysis of walls under reversed-cyclic lateral loads. A wall section was divided into fibers. The heat transfer analysis determined the maximum temperature each fiber has experienced in the whole heating-cooling cycle. Matlab codes were written to modify the material properties of each fiber for the post-fire seismic analysis according to the maximum temperature. In the seismic analysis, the flexural and axial response of the walls was modeled using force-based beam column elements following the recommendations of Pugh (2012); shear deformation was simulated by linear shear model.

The concrete model was SILICON_ETC or CALCON_ETC (Gernay and Franssen, 2011) and the steel model was STEELEC2EN (Franseen, 2011) in SAFIR. These two material models consider the variation of material properties with temperature. The concrete model was Concrete01 and the steel model was Steel02 in OpenSees. Since these two material models do not consider the variation of material properties with temperature, the input material properties need to be modified for the residual concrete and steel after fire exposure. Models for the residual compressive strength, residual peak compressive strain, crushing strain and compressive fracture energy of concrete are based on the recommendations proposed by Chang et al. (2006). Models for the residual yield strength, young's modulus and ultimate strength of reinforcing steel are based on the research of Zhong et al. (2013). Those material models are functions of the maximum temperature the material has experienced during the whole heating-cooling cycle.

After the lateral load-drift response of walls under reversed-cyclic loads was obtained, the following response quantities were compared in this paper. 1) The lateral load capacity, V_{max}, is the maximum lateral load a wall experiences before its lateral loading capacity begins to decrease. 2) The stiffness up to yield, K_{sec}, is a secant stiffness, defined by the lateral load at yield divided by the displacement at yield. 3) Drift capacity, Δ_{fail}/H is the value of the failure displacement divided by the height of a wall. The failure displacement is the displacement at the first occurrence of one of following events i) the lateral load capacity decreases to 80% of the maximum lateral load capacity or ii) a wall reaches its maximum drift in the simulation. 4) Ductility, μ is the value of the failure displacement divided by the yield displacement.

2.2. Validation of the simulation method

This paper focuses on the post-fire seismic analysis of flexure-controlled RC structural walls. However, the post-fire seismic tests of flexure-controlled walls are unavailable for validation of simulation models. Fifteen walls with relative low aspect ratio were selected to validate the simulation method in Section 2.1 (Liu, 2010). Eleven walls were subject to the lateral reversed-cyclic loads after exposure to fire and four walls were subject to lateral reversed-cyclic loads only. All walls failed in shear or shear-flexure mode.

2-D models of wall sections defined in both SAFIR and OpenSees are shown in Figure 1. The thermal boundary condition of the walls is shown in Figure 1. Here, the thermal parameters (ε_r=0.2 and h_c =25 W/m^{2}K for the fire-exposed side and h_c=9W/m^{2}K for the fire-unexposed side) were selected in such a manner that the calculated and the measured temperatures in concrete agreed as much as possible (Bratina et al., 2005). The fire curves input into SAFIR were those measured in the fire tests (Liu, 2010).

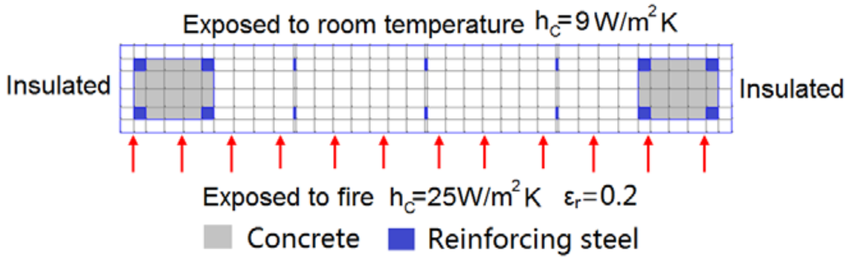
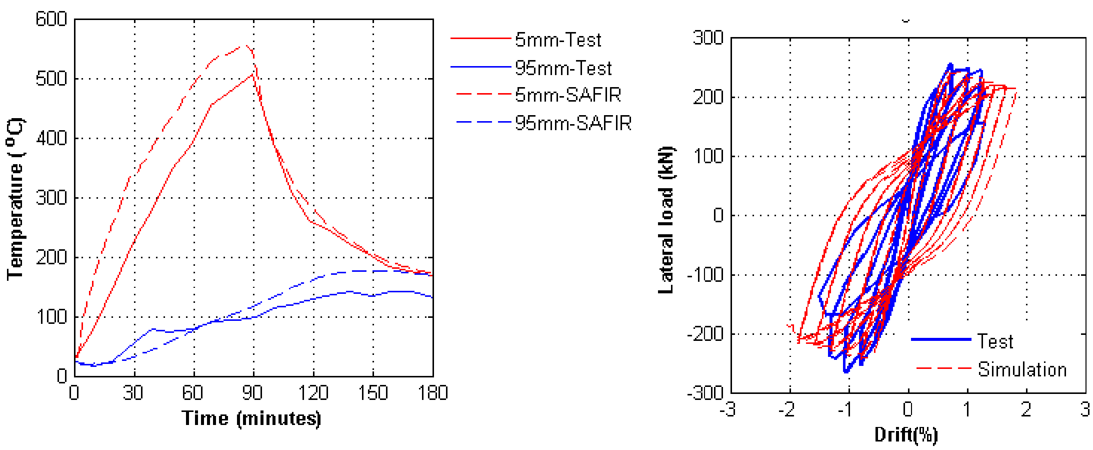


Fig. 1 – Thermal boundary conditions of walls in SAFIR

Take wall N4T9 in the Liu's tests (2010) for example. The temperature history of concrete at 5mm from the fire-exposed side and from the non-exposed side is shown in Fig. 2(a). It shows that the SAFIR model can well simulate the temperature history of concrete at 5mm from the fire-exposed side and from the non-exposed side. Fig. 2(b) shows the results of the OpenSees seismic analysis.

Table 2 compares the experimental and numerical results of the fifteen walls. The numerical method described in Section 2.1 can well predict the residual load-bearing capacity and stiffness of RC walls, by an average error of 7.9% and 9.4% respectively . Since the numerical analysis described in Section 2.1 can only simulate the flexure failure, it overestimate the drift capacity of thoses walls which were characterized by shear failure or shear-flexure failure in the tests. Despite this, these results are useful in demonstrating the capability of the modeling process to capture temperature distribution, initial stiffness, and strength.



(a) Heat transfer analysis (b) Seismic analysis

Fig. 2 – Comparison of experimental and numerical response of wall N4T9 under fire and reversed-cyclic loading (Liu, 2010)

Table 2 Compare experimental and numerical response of walls

	V_{max} Error(%)	K_{sec} Error(%)	$\Delta_{fail/H}$ Error(%)
Mean	7.9%	9.4%	25.5%
Std. Dev.	4.0%	6.7%	12.45%

3. Post-fire seismic analysis of slender RC structural walls

3.1. Description of walls analyzed

To evaluate the impact of fire damage on the seismic performance of flexure-controlled RC structural walls, the post-fire response of walls in Table 3 were investigated numerically, using the simulation procedure in Section 2.1. Those walls have been tested under reversed-cyclic loads at room temperature and all walls failed in flexure pattern.

Table 3 Flexure-controlled RC structural walls

Author	Specimens	$M_b/(V_b l_w)$	Failure Mode	Thickness (mm)
Dazio et al. (2009)	WSH1-WSH5	2.28	Flexure	150
Dazio et al. (2009)	WSH6	2.26	Flexure	150
Liu (2004)	W1	3.13	Flexure	200
Lowes et al. (2012)	PW1	2.84	Flexure	152
Lowes et al. (2012)	PW2-PW4	2	Flexure	152
Oh et al. (2002)	WR20, WR10, WR0	2	Flexure	200
Thomsen et al. (2004)	RW1, RW2	3.13	Flexure	102

3.2. Post-fire seismic analysis of walls

3.2.1 Heat transfer analysis

Since most of test data did not provide information about the type of concrete used, it was assumed that siliceous concrete was used in all tests. Compared to calcareous concrete, siliceous concrete is more vulnerable to fire exposure. According to wall thickness, concrete type and fire resistance rating in ACI 216, the approximated fire resistance of those walls under one-side fire are as following: nearly three hours for walls tested by Dazio et al. (2009) and Birely (2012), more than four hours for walls tested by Liu (2004) and Oh et al. (2002), and nearly 1.5 hours for walls tested by Thomsen et al. (2004). To avoid the failure of a wall under fire, the fire duration in the heat transfer analysis should not be longer than its fire resistance. Therefore, in the post-fire seismic analysis, walls tested by Dazio et al.(2009) and walls tested by Lowes et al. (2012) were subject to half-an-hour fire, one-hour fire, two-hour fire, and three-hour fire; walls tested by Liu (2004) and walls tested by Oh et al. (2002) were subject to half-an-hour fire, one-hour fire, two-hour fire, three-hour fire and four-hour fire; walls tested by Thomsen et al. (2004) were subject to half-an-hour fire, one-hour fire and one and half-an-hour fire. The fire duration here means the duration of the heating phase. ASTM E119 fire curve was used to heat walls in the heating phase and walls were cooled down at 0.5^{o}C/min in the cooling phase. The thermal boundary condition is shown in Figure 3. The thermal parameters are recommended by the SAFIR User Manual (Franssen, 2011).

Heat transfer analysis shows that when the cooling phase begins, the temperature of the fire-unexposed side will continue to increase. Take WR20 wall under half-an-hour fire for example. The temperature at the middle node on the fire-unexposed side (node A in Fig. 3) is 20^{o}C when the heating stops; while the maximum temperature this node has experienced during the whole heating-cooling cycle is 189.6^{o}C, shown in Fig.4. The distributions of maximum temperature in each fiber of the wall section has experienced under fire are shown in Fig. 5. The residual material properties are determined by the maximum temperature concrete and steel have reached during the full heating-cooling cycle rather than by the temperature at the end of heating.

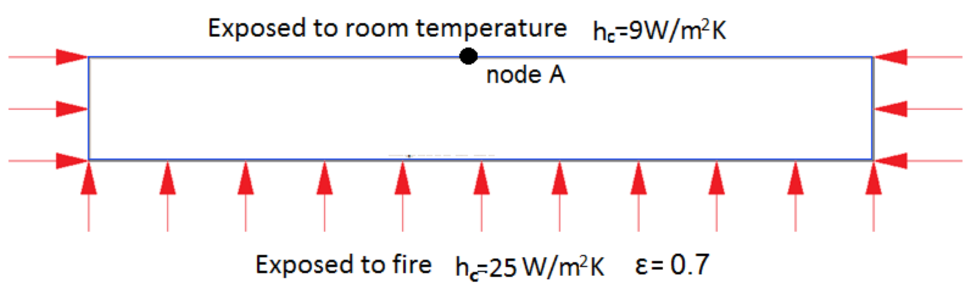


Fig. 3 – Thermal boundary condition

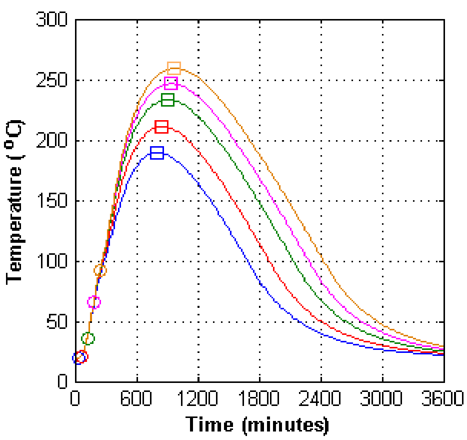


Fig. 4 –Temperature history of node A

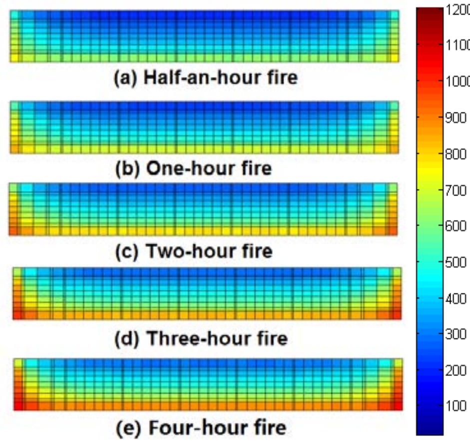


Fig. 5 – Maximum temperature of WR20 section

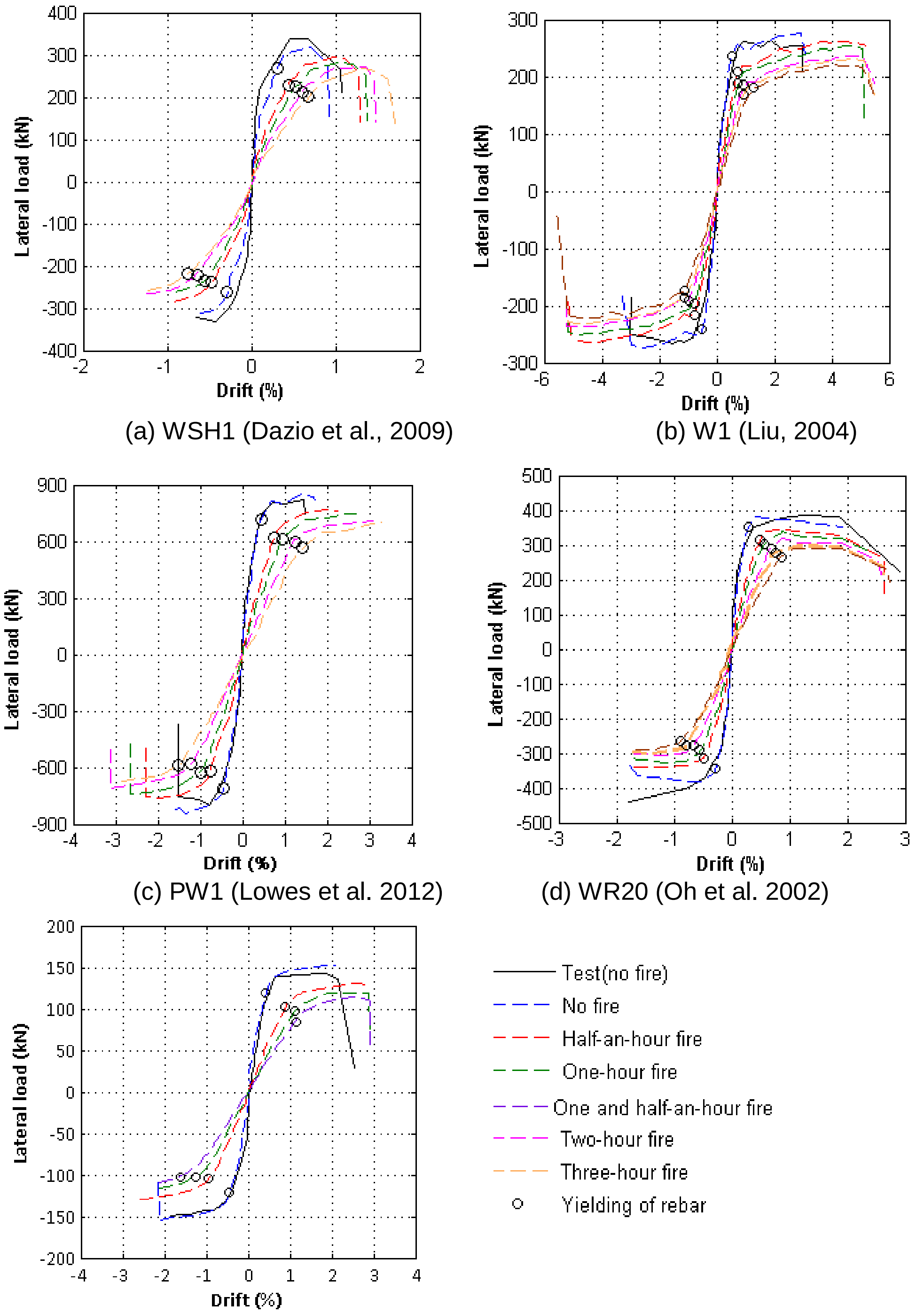
3.2.2 Seismic analysis

After the heat transfer analysis was conducted, the results were used to create OpenSees models with material properties reflecting those from the heat transfer analysis. Reverse cyclic loading was applied to the new models to compare the response to that of unheated walls. Fig. 6 summarizes the results for specimens WSH1, W1, PW1, WR20 and RW1 (Table 3). For clarity, only the backbone load-drift responses are shown for each analysis. The experimental data and the associated numerical model results are shown by solid black and dotted blue lines, respectively. All other lines show the analysis results for the cyclic response following fires of varied durations (with cooling afterwards).

Overall trends in the results presented in Fig. 6 indicate that increased duration of fire results in (1) deceased lateral load carrying capacity, (2) increased flexibility at lower level loads, and (3) increased drift associated with failure of the walls. To further evaluate these observations, key response characteristics are defined to characterize the backbone curve. These values include the lateral load capacity V_{max}, stiffness up to yield K_{sec}, drift capacity Δ_{fail}/H and ductility μ. Each quantity is then normalized by the corresponding value for analysis results with no fire demands (i.e. $V_{max,fire}/ V_{max,0}$, $K_{sec,fire}/K_{sec,0}$, $\Delta_{fail,fire}/\Delta_{fail,0}$ and $\mu_{fire}/\mu_{0.}$ The variation of those ratios with fire duration are shown in Fig. 7.

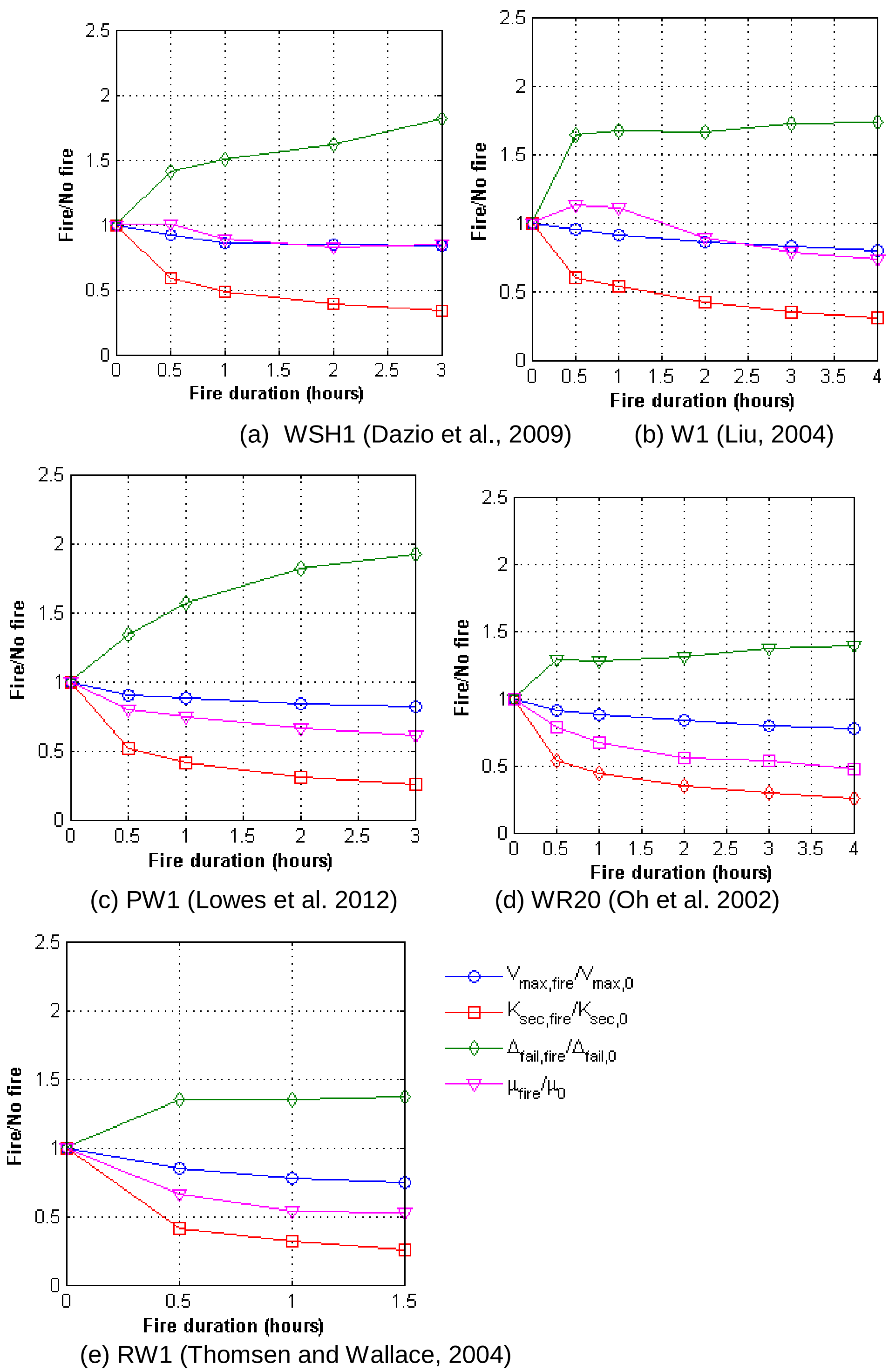
For all walls considered, the greatest impact of the fire exposure is on the stiffness up to yield. For only a 30 minute fire, the stiffness can be as low as approximately 50% of the stiffness for the same wall without fire exposure. Although there is increased reduction in stiffness at longer fire durations, the change is not as drastic as that between 0 and 30 minutes. Similar to the stiffness up to yield, any duration of fire will result in a decrease in lateral load carrying capacity. The amount of decrease in the lateral load carrying capacity is not as significant, with the reduced strength of a wall after three-hour fire exposure being approximately 80-85% of the strength of the same wall without fire exposure.

Fig. 6 shows a clear increase in the failure drift, a trend that is captured in the Fig. 7. This increase in failure drift is misleading due to the increased flexibility of the fire exposed walls. To account for this, the ductility associated with the failure drift was considered. The open circles in Fig. 6 indicate the point at which yielding occurs; this value is used to normalize the failure drift. Fig. 7 shows that the ductility of the wall at failure after three-hour fire exposure ranges from 50-85% of the ductility for the wall without fire exposure.



(e) RW1 (Thomsen and Wallace, 2004)

Fig. 6 –Load-drift backbone curves of fire-damaged walls under lateral cyclic loads



(a) WSH1 (Dazio et al., 2009) (b) W1 (Liu, 2004)

(c) PW1 (Lowes et al. 2012) (d) WR20 (Oh et al. 2002)

(e) RW1 (Thomsen and Wallace, 2004)

Fig. 7– Variation of simulated response quantities with fire duration

4. Conclusion:

Preliminary results from numerical analysis were presented to quantify the impact of fire damage on the seismic resistance of flexure-controlled RC structural walls. The numerical method for the post-fire seismic performance of walls was validated against tests of fire-damaged RC walls subject to lateral reversed-cyclic loads. Results show that the simulation method can well predict the temperature history of wall sections under fire and the lateral load capacity and stiffness of fire-damaged walls under lateral reversed-cyclic loads. However, the proposed method can only simulate flexure failure, for which there is no experimental data to use in verifying the results of the models.

Based on method described above, the post-fire seismic performance of flexure-control RC structural walls was simulated. Results show that fire exposure degrades the lateral load capacity and stiffness of walls. Compared to the lateral load, the stiffness of walls is reduced more significantly by fire exposure. As the fire duration increases, the drift capacity of walls increases; however, the ductility of walls decreases. Most response quantities of walls after half-an-hour fire exposure increased or decreased tremendously, compared to those of undamaged walls. However, when the fire duration lasts longer than half an hour, the variation become less significant.

Parametric studies of post-fire seismic performance of flexure-controlled RC structural walls are currently ongoing to identify critical situations (including fire location, fire duration and cooling rate) in which the seismic performance of flexure-controlled RC structural walls would be most influenced by fire exposure.

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