NUMERICAL INVESTIGATION OF THE FIRE PERFORMANCE OF PILOTI-TYPE RC BUILDING STRUCTURES

Jinwon SHIN[1], Yurim CHU[2], Taejin KIM[3], and Sung-Mo CHOI[4]

SUMMARY

Multiple piloti-type residential buildings have been constructed in Korea due to the convenience of parking. The 2016 Gyeongju and 2017 Pohang earthquakes have resulted in significant damage to the piloti buildings due to the seismic vulnerability of the soft-story structures and the fire in the car park. The study is aimed at numerically investigating the requirements of seismic retrofit strategies such as insulated carbon or glass fiber reinforced polymer (FRP) strengthening for prevention of the fire spread through structural fire analysis. A representative type of existing piloti buildings was selected, and a construction method suitable in terms of fire resistance of structural members has been proposed by conducting seismic performance evaluation and seismic retrofit design. The structural fire responses for the CFRP-strengthened reinforced concrete column subjected to the ASTM E119 standard temperature curve were numerically simulated using coupled thermomechanical analysis to demonstrate the efficacy of the fire protection system. The load bearing capacity of the column under a sustained axial load for the four-hour fire exposure was discussed, and recommendations were provided.

Keywords: *fire performance; piloti-type RC buildings; seismic retrofit; numerical analysis; CFRP*

INTRODUCTION

Many of the low-rise multi-family houses and neighborhood living facilities in Korea have adopted the reinforced concrete (RC) piloti-type structures, which typically consists of frame structures to reserve parking spaces on the first or lower floors or to use as commercial facilities and wall structures to secure living spaces on the upper floors (e.g., Baek et al., 2014). The recent earthquakes in Gyeongju and Pohang led to extensive property damage owing to the seismic vulnerabilities of various piloti buildings and the loss of parked vehicles caused by fire. In the event of a fire, these piloti buildings have a high risk of fire spread due to a kind of stack effect. The typical structural vulnerabilities of the piloti building against earthquakes are the lack of the overall building performance due to excessive inter-story drift of the piloti floor and insufficient strength of the piloti column itself (e.g., Kim et al., 2020). Seismic retrofit should be done to resolve these issues, and it should have the fire resistance in order to achieve the prolonged structural performance.

The increasing applications of fiber reinforced polymer (FRP) materials, often used for the purpose of seismic retrofit in civil engineering, have been seen over the last two decades due to their benefits as strengthening and reinforcing materials (e.g., Firmo et al., 2015). The most common applications are wrapping carbon or glass FRP sheets to the exterior of RC beams, columns and slabs to enhance their strength and ductility. However, the FRP strengthening systems in buildings have a limited use due to the degradations in their mechanical and bond properties at elevated temperatures. Numerous researchers have studied to ensure the fire resistance of the FRP-

[1] *Professor, Department of Architectural Engineering, Catholic Kwandong University, Korea, e-mail: jshin@cku.ac.kr*
[2] *Senior Engineer, TI Structural Engineers, Korea, e-mail: chuyurim92@naver.com*
[3] *President/CEO, TI Structural Engineers, Korea, e-mail: taejin@tistructure.com*
4 Professor, Department of Architectural Engineering, University of Seoul, Korea, e-mail: smc@uos.ac.kr

strengthening systems (e.g., Bisby et al., 2005, Cree et al., 2012, Kodur et al., 2019, Bhatt et al., 2021), but there are few studies to consider the fire resistance of the RC structural elements in piloti-type structures.

This study presents a numerical investigation on the structural fire performance of RC columns strengthened with carbon FRP (CFRP) in piloti-type building structures. A nonlinear pushover analysis for a representative type of piloti structure was performed to evaluate its seismic performance. For RC columns that do not achieve the goal of the seismic performance, the seismic retrofit with CFRP sheets was applied. Since the CFRP sheets should ensure their structural performance at elevated temperatures, the fire performance of the CFRP-strengthened RC column with fire protection for a sustained axial load was evaluated numerically through coupled thermomechanical analyses comprising heat transfer and thermal-stress analysis. Temperature dependent thermal and mechanical properties for the structural components were considered in the numerical model to address the degradation in the mechanical properties at elevated temperatures (Bisby et al., 2014, Kodur et al., 2019). This numerical methodology was validated based on the relevant past experimental data in Cree et al, (2012). The structural fire performance of the seismically strengthened columns was investigated using the validated numerical model. Available design guidance was also discussed. Recommendations for future study were finally provided.

SEISMIC RETROFIT FOR PILOTI STRUCTURES

Piloti structure

A piloti-type reinforced concrete (RC) structure in Kim (2021) were considered, as shown in Fig. 1, as target structures for evaluating the fire resistance. This piloti structure had four stories with a height of 11.5 m and a total floor area of 1,249 m^2 and 648 m^2. The first story height for the open car park was 3.4 m. The compressive strength of concrete, f_{ck}, of the existing concrete members was estimated to be 17 MPa based on Non-Destructive Testing (NDT); the yield strength of reinforcing bars, f_y, was 400 MPa. The ceiling was made of metal for prevention of an outer flashover (Noh, 2019) that can occurs when a flame penetrates the ceiling and propagates inward between the ceiling and the slab because this was beyond the scope of this study.

Fig. 1 Piloti structure

The RC columns were considered for primary members which need seismic retrofit for shear in that the shear failures for the RC columns of piloti structures occurred in the last earthquakes (e.g., 2016 Gyeongju and 2017 Pohang earthquakes). Fig. 2 shows the floor plan of the open car park at the first story, which has four types of columns, TC1, TC2, TC3 and TC4, as shown in Table 1. The shear strengths in the 2 and 3 directions before seismic retrofit were calculated to be 110 kN to 217 kN.

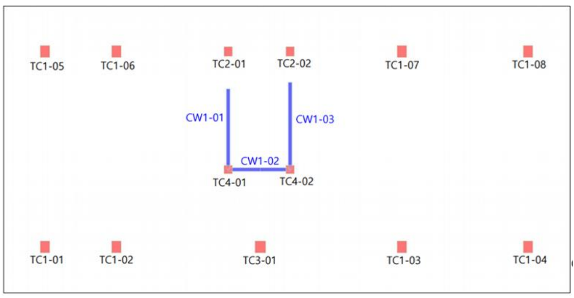
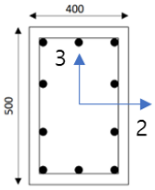
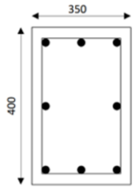
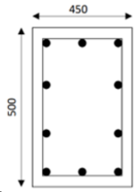
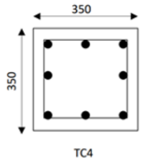


Fig. 2 Floor plan of open car park at the first story

Table 1. RC columns for seismic retrofit

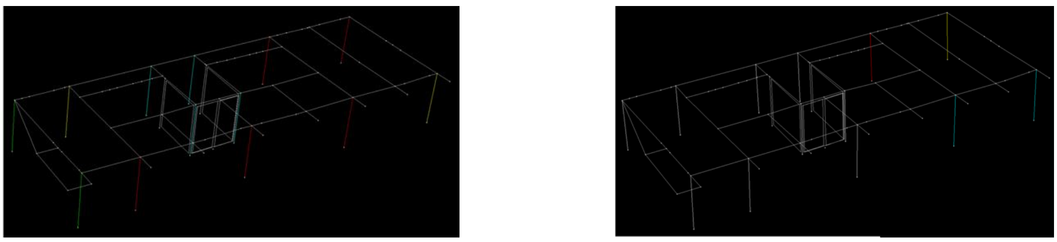
Column ID	TC1	TC2	TC3	TC4
Cross section of column				
Longitudinal rebar	10-D22	8-D22	10-D22	8-D22
Stirrup	10@300	10@300	10@300	10@300
V_{n3} (kN)	188	119	217	110
V_{n2} (kN)	181	130	194	110

Seismic evaluation and retrofit

A nonlinear pushover analysis for the considered piloti structure, shown in Fig. 1, was performed using Midas Gen Midas, 2017), which is a software used for structural analysis and enables an optimal design in the civil engineering and architecture domains, to evaluate the seismic performance of the existing piloti structure. The effective peak ground acceleration, *S*, was 0.18g and the site classification was estimated to be S3. The design earthquake spectral acceleration parameter at short periods (SDS) was 0.462g, and that at for 1.0 s (SDS) was 0.1944g. The seismic performance objective for the piloti structure was to obtain live safety (LS) at design basis earthquake (DBE) and collapse prevention (CP) at maximum considered earthquake (MCE). The entire model shape of piloti structure and the modeling of structural components at the 1st story are shown in Fig. 3. Fig. 4 shows pushrover analyses in the X and Y directions (2 and 3 directions in Table 1) to evaluate the structural members at performance point.

(a) Shape of piloti structure (b) Modeling of structural members at the 1st story

Fig. 3 Modeling of piloti structure for seismic performance evaluation



(a) X-directional pushover (b) Y-directional pushover

Fig. 4 Pushover analysis at MCE level; evaluation of members at performance point

It appeared that the shear capacities for the columns of TC1 and TC3 did not reach the shear forces at their performance points as a result of the pushover analysis. The axial, shear and bending forces at performance points for the TC1 and TC3 columns are listed in Table 2. The maximum shear forces for the TC1 and TC3 columns were computed to be 202.2 kN and 239.8 kN, respectively, at performance points in the 2 directions for both. To resist these forces, the RC columns were strengthened with two layers of carbon fiber reinforced polymers (CFRP), as shown in Fig. 5, which enabled improvement primarily in ductility and some in strength, including the confinement effects completely wrapping the column with CFRP sheets. The CFRP sheets of carbon NR43R was used. The mechanical properties of the carbon-NR43R FRP are provided in Table 5. This CFRP has the tensile strength of 2677.2 MPa, and the elastic modulus of 372652.7 MPa, the failure strain of 0.011, and the thickness of 0.165 mm per layer.

Table 2 Member forces at performance point

Members	P_u	V_{u2}	V_{u3}	M_{u2} (kN-m)		M_{u2} (kN-m)	
				i-end	j-end	i-end	j-end
TC1	-1127	202.4	20.53	31.34	30.19	306.5	300.7
TC3	-914	239.8	2.6	2.946	4.848	365.4	350.7

(a) TC1 (4003500 mm) (b) TC3 (4503500 mm)

Fig. 5 CFRP strengthening of RC column

Table 5 Properties of NR43R CFRP sheets

Property	Value	Note
Type of CFRP	NR43R	2 layers
Fiber type	Carbon	
Tensile strength (MPa)	2677.22	
Elastic modulus (MPa)	372652.7	
Failure strain	0.011	
Thickness (mm)	0.165	per layer

The shear strength of the RC column of TC1 was increased from 181 kN to 251 kN in the 2 direction with CFRP strengthening of two layers of the 0.165-mm CFRP sheet. The increased shear strength was greater than the shear force, 202.4 kN, respectively, at performance point, as shown in Table 6. For TC3, the shear strength with seismic retrofit was 261 kN greater than the shear force of 239.8 kN at performance point. The CFRP-strengthened TC1 and TC3 RC columns were seismically enhanced and was evaluated for their fire resistance in the following sections.

Table 6 Results of CFRP strengthening for TC1 and TC3

Shear strength	TC1		TC3	
	Before retrofit	After retrofit	Before retrofit	After retrofit
V_{n2} (kN)	181	251	194	261
V_{u2} (kN)	202.4		239.8	
V_{n3} (kN)	188	270	217	270
V_{u3} (kN)	20.53		2.6	

FIRE ANALYSIS FOR CFRP-STRENGTHENED RC COLUMNS

RC column against standard fire curve

The fire analysis of the RC columns strengthened with CFRP (TC1 and TC3) were conducted using the ASTM E119 standard temperature curve (ASTM, 2022), as shown in Fig. 6. The fire performances of the columns were evaluated using a finite element code, ABAQUS (2018), for their axial strengths. A sequentially coupled thermal-mechanical method, which was the fully couped procedure, was adopted for the fire performance evaluation; heat transfer analyses were performed first, and the temperature distributions on the columns were remapped into thermal-stress analyses. Since the CFRP was vulnerable under high temperatures, the CFRP-strengthened columns were insulated to meet the failure criteria, which is described later.

A validation study was preceded to demonstrate the numerical methodology of the fire analysis. The past experimental studies in Cree et al. (2012) were adopted for the numerical validation. Cree et al. provides test results of temperature and axial deformation for insulated FRP-strengthened and insulated RC columns subjected to a sustained axial load and the ASTM E119 standard temperature curve. Based on the validated numerical model of the insulated CFRP-strengthened RC columns, a finite element analysis to assess the fire resistance performance was carried out. The appropriate thickness of insulation materials to protect the structural performance of the insulated CFRP sheets and the columns at elevated temperatures were examined based on predetermined performance criteria. The axial strength of the columns at elevated temperature were investigated in terms of axial deformation and load carrying capacity.

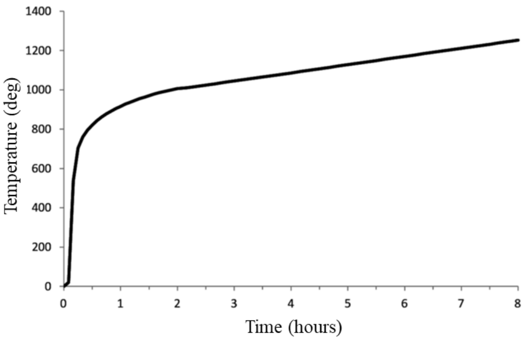


Fig. 6 ASTM E199 standard temperature curve

Calculation of axial load capacity

The design axial strength, ϕP_n, for a RC column is determined based on ACI design guidelines (ACI, 2017):

$$\phi P_n = 0.85\phi\left[0.85 f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right] \tag{1}$$

where ϕ is 0.7 for circular columns and 0.85 for rectangular columns; A_g is gross cross-sectional area of concrete; and A_{st} is area of longitudinal reinforcement in compression. The confining pressure, ρ_f, is calculated per ACI (2002):

$$\rho_f = \frac{4nt_f}{d} \tag{2}$$

where n is number of layers of CFRP; t_f is wrap thickness per layer; and d is diameter of column. The effective ultimate strength of the CFRP, f_{fe}, is calculated as:

$$f_{fe} = C_E f_{fu} \tag{3}$$

where C_E is take as 0.85 for CFRP with an interior conditioned exposure (ACI, 2002). The confining pressure at ultimate, f_{lat}, is:

$$f_{lat} = \frac{\kappa_a \rho_f f_{fe}}{2} \tag{4}$$

where κ_a is confinement effectiveness factor (0.85 for spiral columns, 0.8 for tied columns). The confined ultimate strength of the concrete, f_{cc}', is:

$$f_{cc}' = f_{co}'\left(2.25\sqrt{1+\frac{7.94 f_{lat}}{f_{co}'}} - \frac{f_{lat}}{f_{co}'} - 1.25\right) \tag{5}$$

The ultimate strength of the FRP-strengthened column is:

$$\phi P_{n,\max} = 0.85\phi\left[0.85\psi_f f_{cc}'\left(A_g - A_{st}\right) + f_y A_{st}\right] \tag{6}$$

Fire protection

Fire protection was provided using Sikacretes®-213F, which is a fire insulation material, spray-applied on the surface of FRP-strengthened columns (Cree et al., 2012). The insulation is a cement-based, dry mix fire protection mortar for wet sprayed application developed for fire-resistant tunnel linings. The considered test specimen of the rectangular column had an average insulation thickness of 40 mm. The phyllosilicate aggregates included in the insulation material are considerably efficient in resisting the heat generated from the hydrocarbon fires.

Procedure

The procedure to numerically investigate the fire performance of column exposed to the ASTM E119 standard temperature curve is shown in Fig. 7. For the given insulated FRP-strengthened RC column, the axial strength at ambient temperature of 20°C was simulated using a displacement-controlled loading until the column failed. A transient heat transfer analysis of was then performed to obtain the temperature distributions over time, where temperature-dependent thermal properties (e.g., density, conductivity, specific heat) were considered. The column was exposed to fire for approximately four hours. The temperature data on the column was fully remapped into the thermal stress analysis, where the column was subjected to the sustained axial load in Table 2, obtained previously in the pushover analysis, and the ASTM E119 standard temperature curve. The fire endurance of the column was evaluated by confirming that the column exposed to the four-hour fire resists the sustained axial load. Temperature dependent elastic-plastic material properties were applied to model the degradation of mechanical properties at elevated temperatures. When the column failed under the sustained load, the thickness of the insulation material was adjusted, and the thermal-structural analysis was repeated until the axial load capacity reduced at high temperatures holds above the applied load.

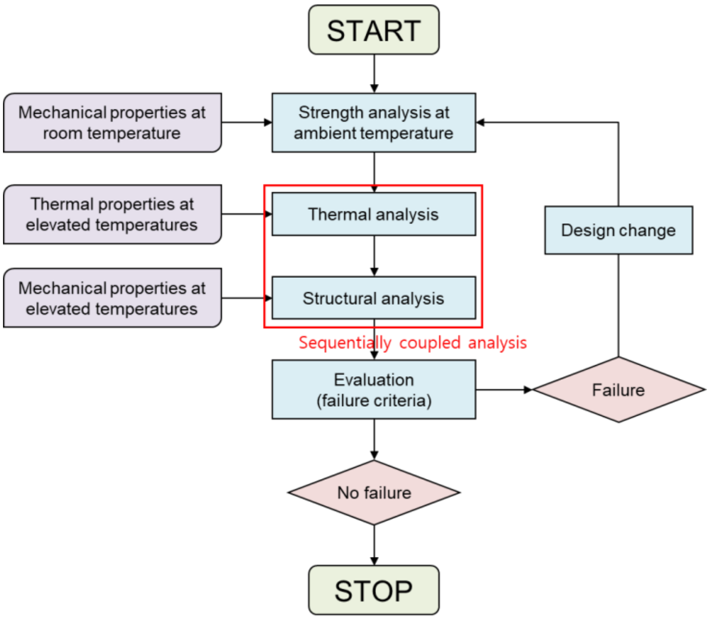


Fig. 7 Procedure of structural fire evaluation

Performance criteria

Three performance criteria have been considered for the insulated CFRP-strengthened RC columns subjected to fire based on Bisby et al. (2014). Criterion 1 is that the average temperature of the CFRP sheet should not exceed the glass-transition temperature of the polymer matrix. The FRP confinement effect can be lost when this temperature is exceeded. This has been considered as a design criterion to allow the use of FRP in buildings. The insulation was assumed not to crack or debond. The FRP matrix consists of a specific epoxy resin, where the glass-transition temperature, T_g, was assumed to be 93°C. Criterion 2 is that the temperature at the outside face of the FRP shell should not exceed the ignition temperature of the polymer matrix. This criterion is to guard against the evolution of toxic gas and smoke and to reduce the potential for the increase of flame spread. The ignition temperature was assumed to be 450°C (Bisby, 2003). Criterion 3 is that the load-bearing capacity of the column should not fall below the full unfactored service load of the strengthened member. This is equivalent to the traditional ULC-S101 failure criterion currently used for RC columns in buildings in Canada.

NUMERICAL MODELING AND VALIDATION

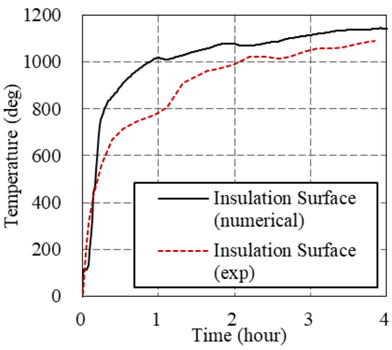
Numerical modeling

A numerical model of the RC column of the piloti structure with insulated CFRP strengthening was constructed using a finite element code, ABAQUS (2018). Concrete was modeled using eight node brick elements (C3D8R) with the constitutive model of concrete damage plasticity (CDP), which is a continuum, plasticity-based, damage model for concrete, available in the code. The temperature-dependent thermal and mechanical properties were determined based on Eurocode-2 (2014). Reinforcing bars were modeled using beam elements with bilinear curves. The thermal and temperature properties were determined with reference to Buchanan and Abu (2017). For modeling CFRP sheet, S4R shell elements were used in consideration of the thin layers. The NR43R CFRP sheets were used with the properties given in Table 5. Orthotropic characteristics were involved in the material definition of CFRP. The temperature dependent thermal and mechanical properties were defined based on Bisby et al. (2005). The bond behavior between CFRP sheets and concrete was also considered, and the bond slip was modeled with quadratic traction based on Dai et al. (2013), where the fracture energy of 15.1 N/m was used. The insulation material was modeled using solid elements and the thermal properties were determined based on Bhatt et al. (2021).

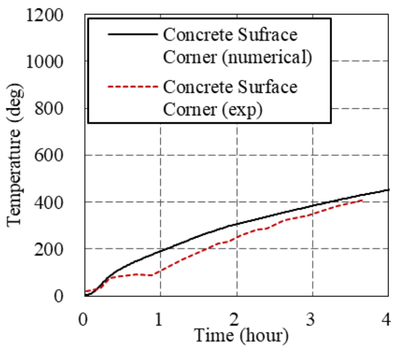
The reinforcing steels were embedded into concrete, assuming perfect bond between them. Surface to surfaces contacts were applied to the surfaces between insulation materials, FRP and concrete. The bond strength at the surface between FRP sheets and concrete was also considered in the numerical model based on Ikramullah et al. (2020), where the interfacial shear strength was defined with cohesive properties including stiffness coefficients (K_{nn} of 2700 N/mm^3, K_{tt} of 2700 N/mm^3, and K_{ss} of 2700 N/mm^3), fracture energy of 15.15 N/mm, and damage initiation (t_{nn} = 270 N/mm2, t_{tt} = 270 N/mm^2, t_{ss} = 270 N/mm^2).

Validation

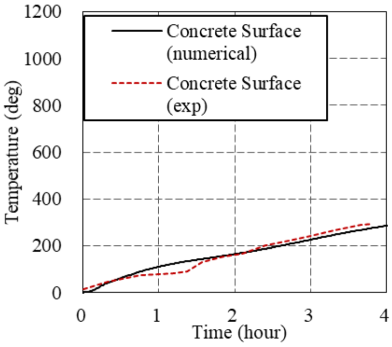
The results of the transient heat transfer analysis were compared to the test results in Cree et al. (2012), as shown in Fig. 8. Temperatures on the insulation surface, concrete surface (surface between CFRP and concrete), concrete surface corner and center of concrete (cover 153) were compared with the test results. The numerical predictions and test results were in good agreement with an acceptable accuracy. The heat transfer model was thus considered to be validated. The temperature distributions over time on the cross section of the FRP-strengthened RC column are shown in Fig. 9. It is observed that the insulation materials effectively reduce the temperatures of the structural components.



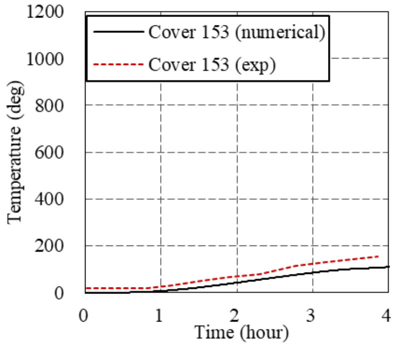
(a) Insulation surface



(b) Concrete surface corner

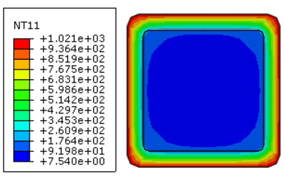


(c) Concrete surface

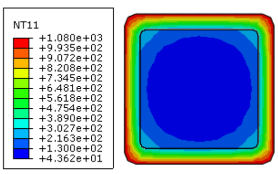


(d) Cover 153

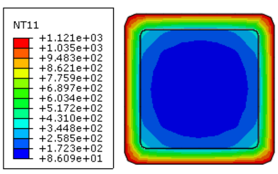
Fig. 8 Comparison of temperatures between numerical predictions and test results



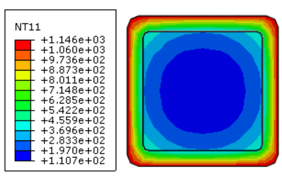
(a) After 1 hour



(b) 2 hours



(c) 3 hours

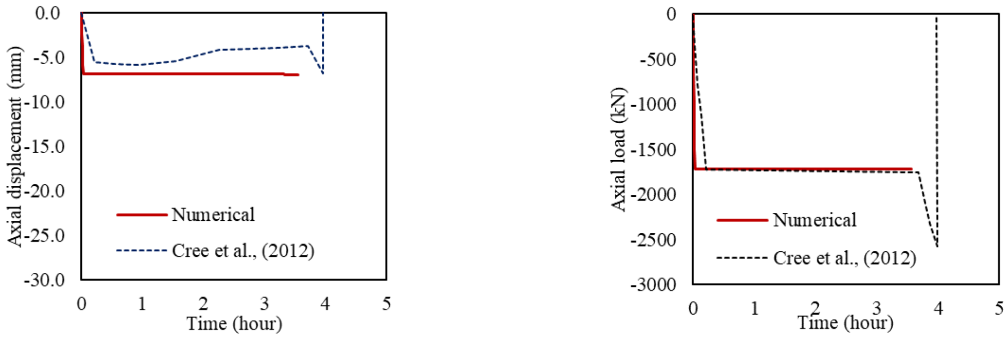


(d) 4 hours

Fig. 9 Temperature distributions on cross section of CFRP-strengthened RC column

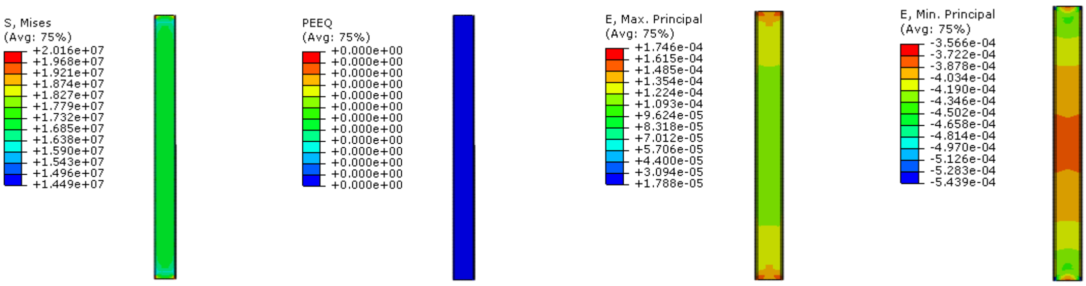
The numerical results obtained from the thermal-stress analysis compared with the test results are shown in Fig.

10. It is seen that the computed axial deformation and axial load with time are comparable to the test measurements, noting that the analysis was run for approximately 3.8 hours and the test results thereafter were out of the scope of this study. The sequentially coupled thermal-stress analysis was therefore considered to be validated. The fire performance of the columns in Fig. 5. were evaluated using the validated numerical methodology for heat transfer and thermal-stress analyses in the following section. The stress and strain distributions of the CFRP-strengthened RC column is shown in Fig. 11. All the structural components behaved in the elastic range. The maximum compressive and tensile strain can be identified in Figs. 11(c) and 11(d) using the maximum and minimum principal strain distributions.



(a) Axial displacement (b) Axial load

Fig. 10 Comparison of axial displacements and loads between numerical predictions and test measurements



(a) Von Mises stress (b) PEEQ (c) Maximum principal strain of concrete (d) Minimum principal strain of concrete

Fig. 11 Stress and strain distributions of CFRP-strengthened RC column for validation

FIRE PERFORMANCE OF COLUMNS IN PILOTI STRUCTURE

Thermal performance

The fire performance of the insulated CFRP-strengthened RC column, shown in Fig. 5(a), was evaluated using the numerical methodology presented in the previous section with the similar sequentially coupled thermal-stress procedure comprising transient heat transfer and thermal-stress analysis. The ASTM E119 standard temperature curve was used as the fire load. Fig. 12 shows the temperature histories for the components of the strengthened column calculated from the heat transfer analysis. The temperatures on the CFRP and RC column were significantly reduced due to the insulation materials. The maximum temperatures on the structural components after fire exposure of four hours were 293°C for the concrete surface, 283°C for the longitudinal reinforcing bars, 210°C for the stirrup. The onset of the glass-transition temperature of 93°C was reached after 32 minutes, indicating the insulation system can maintain the temperature of the concrete surface below the onset of the glass-transition temperature for approximately 0.5 hours. The temperatures of all the structural components were below the ignition temperature of polymer matrix for four hours. It is also observed that the insulation system led to the temperatures of all the components of the column less than 200°C for two hours. The temperature distributions for the components of the column are shown in Fig. 13. It is seen that the insulation system effectively protects the column against high temperatures. The structural performance of the column with the insulation system is discussed in the following section.

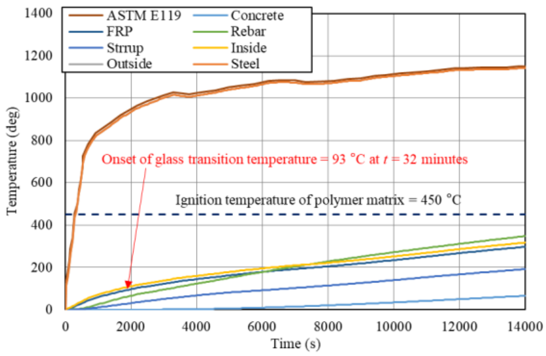
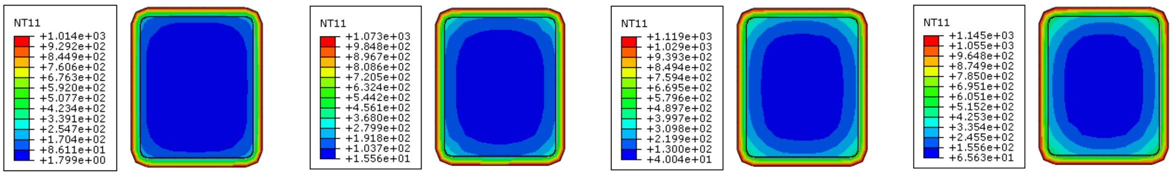


Fig. 12 Temperature histories calculated for components of FRP-strengthened RC column



(a) After 1 hour (b) 2 hours (c) 3 hours (d) 4 hours

Fig. 13 Temperature distribution on cross section of FRP-strengthened RC column of piloti structure

Structural performance

The fire endurance of the CFRP-strengthened RC column for the sustained axial load was assessed through thermal-stress analysis. Fig. 14 shows axial displacement history for the ASTM E119 temperature curve. The axial displacement gradually increases with time, but no sharp increase in displacement was observed, indicating that the column did not failure and the axial load capacity with the seismic retrofit and the thickness of the insulation materials was sufficient to resist the sustained axial load. The stress and strain distributions of FRP-strengthened RC column is shown in Fig. 15. The responses of all the components after the four-hours fire exposure were in the elastic range.

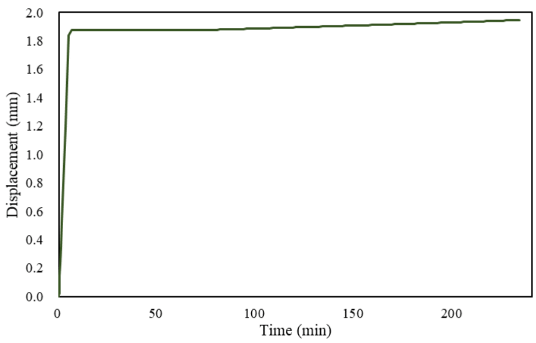
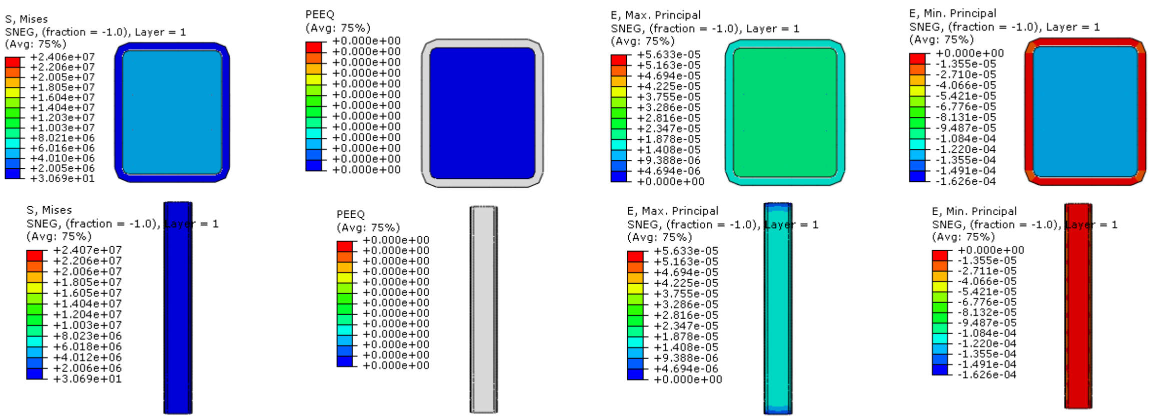


Fig. 14 Axial displacement history of column at elevated temperatures



(a) Von Mises stress (b) PEEQ (c) Max. Principal strain (d) Min. Principal strain

Fig. 15 Stress and strain distribution of FRP-strengthened RC column of piloti structure

CONCLUSIONS

A numerical study was performed to investigate the fire resistance of RC columns strengthened seismically using the CFRP sheets in piloti structures. A nonlinear pushover analysis was conducted to evaluate the seismic performance of the structural members of the piloti structures, and it revealed that the seismic retrofit for two columns, TC1 and TC3, was required against the shear forces at the performance points. The RC columns was strengthened using CFRP sheets to exhibit their seismic performance. The structural fire performance of the CFRP-strengthened RC column was then evaluated utilizing the sequentially coupled thermal-stress analysis method. The following conclusions can be drawn from the results of this study:

1. The applied insulation system was effective in protecting the CFRP- strengthened RC column such that they were able to achieve the four-hour fire endurance ratings according to ASTM E119.
2. The insulation fire protection could maintain the temperature of the CFRP below the onset of the glass-transition temperature of 93°C for 32 minutes, but this did not affect the axial load capacity of the column for the four-hour fire exposure
3. The temperatures of all the components of the column with the insulation materials were below the ignition temperature of polymer matrix, indicating the efficiency of the insulation system.
4. The proposed numerical model developed based on the past experimental results well predicted the structural fire behavior of the insulated CFRP-strengthened RC column of piloti structure. This numerical model can be utilized to optimize various design parameters affecting the fire resistance, including the thickness of the fire insulation.

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