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Research Article

Investigation of the cyclic load behavior of reinforced concrete frames exposed to high temperatures using the finite element method

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ABSTRACT

In recent years, many experimental and numerical studies have carried out to design structures resistant to high temperature effects caused by events such as fire. Reinforced concrete structures exposed to high temperature effects due to events such as fire, etc., also exposed to cyclic loading effects such as earthquakes during their lifetime. This study numerically investigates the behaviour of reinforced concrete frame members under cyclic lateral loading after exposure to high temperature effects In this context, 13 reinforced concrete frame members were modelled with the ABAQUS program and analysed under cyclic lateral loading after being exposed to 200, 400, 600 and 800 °C for 60, 120 and 180 minutes. Because of the thermo-mechanical analysis, the lateral load - horizontal displacement relationship and stiffness - horizontal displacement relationship of the reinforced concrete frames compared with the reference element that not exposed to high temperature effect. The results obtained confirm that the lateral load and stiffness capacities obtained from thermo-mechanical analysis with the CDP material model in ABAQUS are in good agreement with the experimental results. The use of the degree of high temperature and the duration of exposure as two variable parameters at the same time constitutes the main innovation that distinguishes the study from the literature. Because of the study, it recommended that reinforced concrete frames exposed to temperatures of 600 °C and above for 60, 120 and 180 minutes should not be use without strengthening. It thought that these suggestions would be useful to the reader in the rapid evaluation of fire-exposed reinforced concrete structural elements.

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INTRODUCTION

Buildings in urban areas should designed to provide a safe and healthy environment for their inhabitants throughout their service life. Reinforced concrete structures, where life safety criteria met, are widely used in the construction industry and are preferred for their durability, reliability and long life. However, there are situations where the behaviour of these structures can change significantly when

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expose to different environmental factors and loading. One of the conditions that affect the behaviour of reinforced concrete structures is high temperature effects. Situations such as fire, explosion, industrial accident or other disasters can cause significant temperature increases in reinforced concrete structures. High temperature exposure is a serious condition that causes changes in material properties and this can affect the overall behaviour of a structural element [1]. Reinforced concrete structures may rarely collapse in fire events, but fire-induced damage to structural elements is inevitable because of material deterioration and thermal expansion. Many studies have conducted on the effect of high temperatures on the mechanical strength of reinforced concrete structural components. In these studies, it was determined that the mechanical properties of concrete deteriorated after exposure to fire and that these deteriorations did not fully recover after the concrete cooled [2, 3]. Concrete exposed to fire may suffer significant mass loss. Eurocode 4 [4] suggests that when the maximum temperature of concrete is above 300 °C, there is a 10% greater loss of strength after cooling from the maximum temperature. Tests performed by Li and Franssen (2011) [5] revealed that the additional compressive strength reduction in the cooling phase can be as high as 20% of the strength at maximum temperatures. Studies on cooling rate have shown that the strength of concrete decreases more rapidly as the temperature decreases, such as cooling the concrete by giving water or spraying [6-9]. This explained by the fact that if the temperature decreases at a higher rate, a greater 'thermal shock' occurs in the concrete [9]. Some studies in the literature have investigated the changes in the strength and durability of concretes produced from materials with different properties when exposed to acid and heat effects. Teshnizi et al. [10] conducted thermo-mechanical tests of concretes produced using various combinations of zeolite, metakaolin, slag and Portland cement. The test results showed that the specimens made of slag-derived material had better resistance and the highest average compressive strength in strength tests after exposure to temperatures of 300 °C and 500 °C. However, the mechanical properties of reinforced concrete steel also negatively affected after exposure to high temperatures [11, 12]. Chinthapalli and Agarwal [13] investigated the effect of confining reinforcement on the compressive behavior of reinforced concrete columns exposed to high temperatures. Two different concrete strength classes (30 MPa and 50 MPa), two different heating times (140 minutes and 200 minutes) and two different confining reinforcement ratios (0.25% and 2.14% by volume) used in the study. The test results showed that the increase in the amount of confining reinforcement significantly increased the axial load carrying capacity of reinforced concrete columns exposed to high temperatures. Tests measuring the effect of high temperature on concrete components mostly examined on reinforced concrete frame elements. Junhua et al. [14, 15] in the research conducted by, it was determined that the bearing capacity of the reinforced concrete column - beam connection area exposed to fire decreased significantly. It was determined that if the fire exposure time was 75 minutes, the bearing capacity of the column and beam connection area decreased by 13.2% - 15.8%, and if it was 120 minutes, the decrease in the bearing capacity increased to 33.1% - 34.9%. El-Hawary et al. [16, 17] have experimentally shown in their studies that the duration of fire exposure has a significant effect on the behaviour of beams. Xiao et al [18] tested four one-span and one-story reinforced concrete frame members under cyclic load effect after subjecting them to high temperature effect. As a result of the study, it was determined that the stiffness change between the beams and columns forming the frame changed the earthquake behavior of the structure. It was also found that the failure mode was undesirable. However, in most studies in the literature, the change in the dynamic behaviour of reinforced concrete structural elements after high temperature impact has not been investigated [19, 20, 21, and 23]. In addition, in some of the studies, the cases of reinforced concrete structures that were first exposed to earthquake effects and then exposed to high temperature effects such as fire were discussed [23, 24].

In recent years, fire etc. Many advances have made regarding the design of reinforced concrete structures exposed to high temperature effects due to events, and studies in this field continue. Studies show that the finite element modelling technique is suitable for performing time-dependent analyses under thermal conditions. Dabbaghi et al. [25] investigated the behavior of lightweight aggregate reinforced concrete beams subjected to temperatures of 25, 250, 500 and 750 °C under quasi-static cyclic loading. In order to verify the experimental results obtained, they modelled reinforced concrete beams using the OpenSees program. The results confirmed that the numerical findings from the material models in OpenSees are able to provide the beam capacity as well as the failure behavior in strength and stiffness in good agreement with the experimental results. Melo et al. [26] conducted a series of large-scale experiments on reinforced concrete columns with stainless steel reinforcement. The experimental results compared with numerical analysis in OpenSees. Although there are many package programs that perform structural analysis of concrete, steel and reinforced concrete elements using the finite element modelling technique, commercial programs such as ANSYS and ABAQUS are most preferred [27]. Prakash and Srivastana [28] developed a computationally efficient framework for thermo – mechanical analysis of three-dimensional reinforced concrete frames. The developed model uses three coupling modes between thermal, mass transport and structural analysis, and it has concluded that the model is more efficient in predicting the thermo-mechanical properties of steel structures. Ozbolt et al. [29] numerically examined the impact properties of reinforced concrete slabs affected by thermal-induced damage. In numerical modelling, a scaled plane model dependent on velocity and temperature was use. With the help of the correlation established between the experimental and numerical results, it concluded that the scaled plane model depending on temperature and speed could reasonably imitate the experimental studies. Ali et al. [30] numerically investigated the nonlinear behavior of composite steel-reinforced concrete shear walls under reversed cyclic loading using ABAQUS software. Modelling details of structural components, contact conditions between steel and concrete and boundary conditions explained in the study. In addition, load-displacement curves and ultimate strength values verified by comparing with experimental data. Zhou et al. [31] developed a method to predict the lateral load carrying capacity of composite shear walls with steel plates for which finite element models were created using ABAQUS program. The results of the method developed in the study are in good agreement with the finite element results.

The strength and performance decreases in reinforced concrete structures and elements exposed to high temperature effects due to events such as fire, etc. will make the structure and its elements more risky in cases such as earthquakes that may occur later if measures not taken. For this reason, the earthquake behavior of structures and elements that have experienced fire is a subject that requires more research. The aim of this study is to determine the change in the earthquake behaviour of reinforced concrete frames exposed to high temperature effects. In this context, it is aimed to investigate the behaviour of reinforced concrete frames exposed to high temperature effects of 200, 400, 600 and 800 °C for 60, 120 and 180 minutes under cyclic load effects using the ABAQUS package program and to compare the obtained data with experimental results. In most of the studies in the literature, reinforced concrete elements exposed to the temperatures specified in ISO 834 [32] for varying periods or heating continued until the target temperatures completely passed through the cross-section of the test members. In other words, the duration of high temperature impact alone or the degree of temperature alone used as a variable parameter. In this study, two parameters, both high temperature and exposure time, considered as variables at the same time. It is thought that the study is original in this respect and will contribute to the literature. In addition to this, it is thought that the findings obtained in the study will be very useful to the reader in the rapid evaluation of a structure that has undergone fire, in terms of not using the frames exposed to which temperature degrees and for which periods without strengthening.

MATERIALS AND METHODS

In this study, the behavior of 13 reinforced concrete frames subjected to different temperatures (200, 400, 600 and 800 °C) for different durations (60, 120 and 180) under cyclic lateral load was numerically investigated using ABAQUS package. In many studies in the literature [5, 19, 20, 21], it is seen that the high temperature applied to reinforced concrete structures and elements generally varies

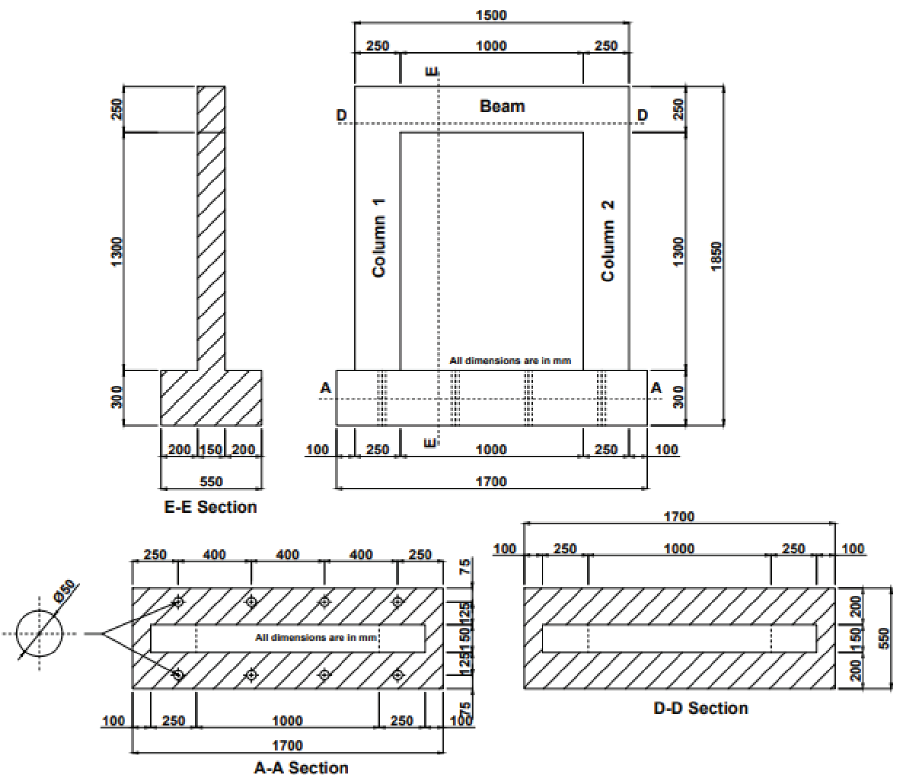


Figure 1. Dimension and cross-section properties of reinforced concrete frame elements.

between 200 °C and 1200 °C. Considering the deterioration of concrete and steel materials under the influence of high temperature, the critical temperature levels that will be decisive in this study are considered to be minimum 21 °C and maximum 800 °C. 21 °C is known as room temperature and was used to evaluate the reference test member, which was not exposed to high temperature. 200 °C is considered as the critical temperature since the modulus of elasticity of concrete does not change up to 121 °C. Since the structure of C-S-H gels breaks down at 550 °C and above, 600 °C was chosen as the critical temperature in this study. Since concrete loses all its properties at 900 °C, 800 °C is set as the upper limit for the critical temperature.

Dimensions, cross-sectional properties and reinforcement details of all test elements are the same and shown in Figure 1 and Figure 2.

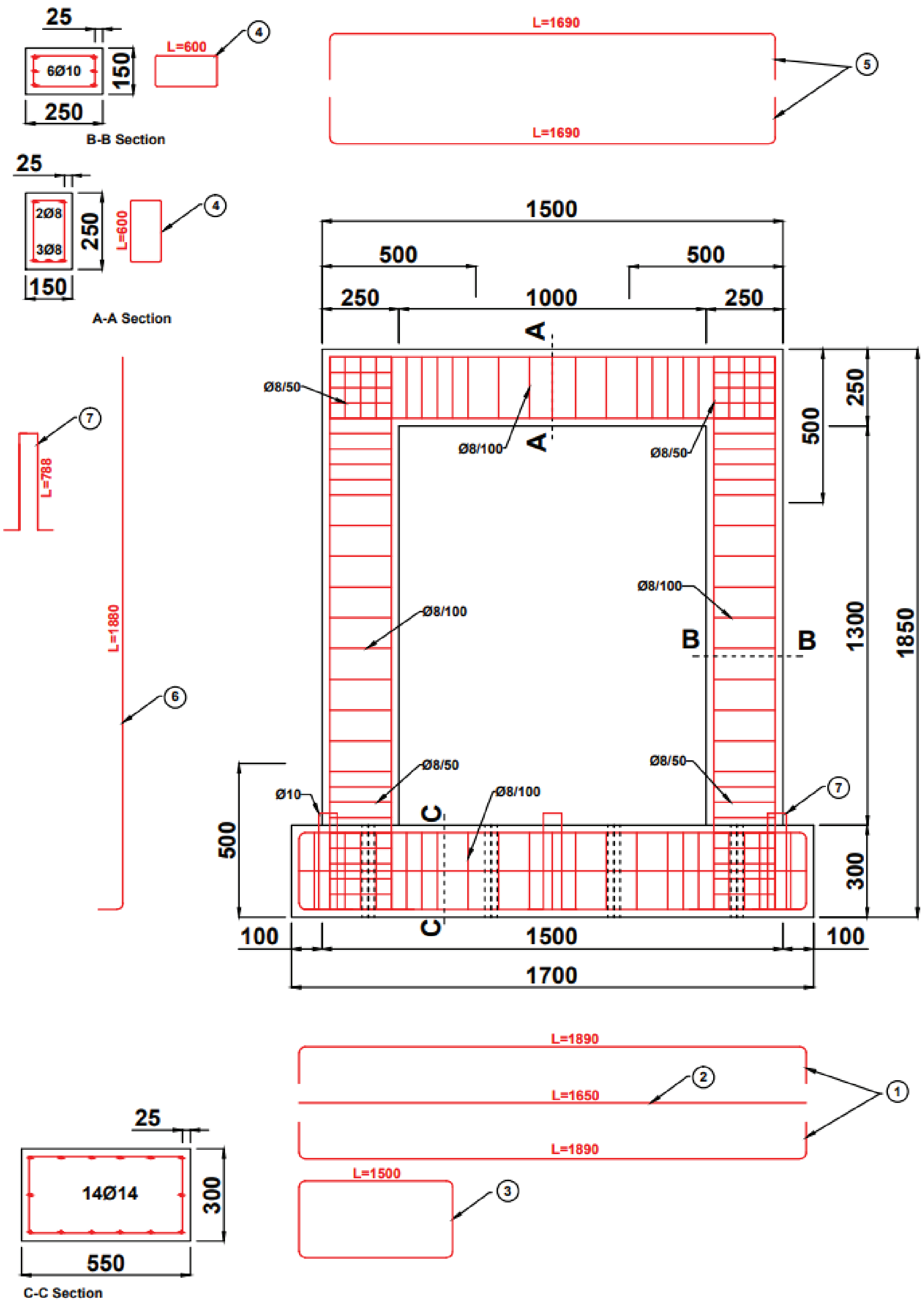


Figure 2. Reinforcement properties of reinforced concrete frame elements.

In all reinforced concrete frame test elements, the concrete strength class considered as C25/30 and the reinforcement strength class considered as B420C.

RELIABILITY OF THE FINITE ELEMENT MODELLING METHOD USED

It is of great importance for this study to demonstrate the reliability of the finite element modelling method used in modelling the reinforced concrete frame and to verify the numerical analysis. In this direction, the lateral load - horizontal displacement relationship obtained from the repeated horizontal loading tests performed on RCF_13_ Reference element, which not exposed to high temperature effect, and the damage zones caused by the cracks occurring on the frame compared by numerical analysis (Figure 3).

In the numerical study, in the cycle when RCF_13_ Reference element reached its maximum load carrying capacity, the horizontal load level was determined as 74 kN in pushing and 72 kN in pulling. In the experimental study, the horizontal load level now when the maximum load carrying capacity was reached was found to be 75 kN in pushing and 62.5 kN in pulling. When the experimental and numerical results compared, it was determined that there was a difference of 1.33% in pushing and 13.19% in pulling. Considering that there are too many details on the model in nonlinear analysis, this difference can be say to be at an acceptable level. As a result of the nonlinear finite element analysis of the RCF_13_Reference test element, the cracks formed were concentrated in the column - beam joints and in the areas close to the bearing and support as in the experimental study.

CREATING FINITE ELEMENT MODELS OF REINFORCED CONCRETE FRAMES

The numerical model considered in this study implemented in ABAQUS program. The numerical analyses carried out in two stages and in the first stage, thermal analysis of the reinforced concrete frame, for which a finite element model made, performed at different times and temperatures. In the second stage, the thermally damaged frames tested under cyclic lateral loading. Figure 4 shows the process of evaluating the performance of reinforced concrete frames under cyclic lateral load after high temperature.

In numerical analyses performed using the finite element method, reducing the element size is extremely important in reaching the results of the analysis. Decreasing the finite element size will cause the number of elements to increase, causing the analysis time to extend and last for days. On the other hand, while increasing the element size shortens the analysis time, it causes convergence errors to increase and the analysis to be incomplete. In preparing the finite element model of the reinforced concrete frame, an effort made to determine the most appropriate finite element mesh in order to shorten the analysis time and to observe the crushing and cracking that would occur in the concrete. For this purpose, the finite element model of the

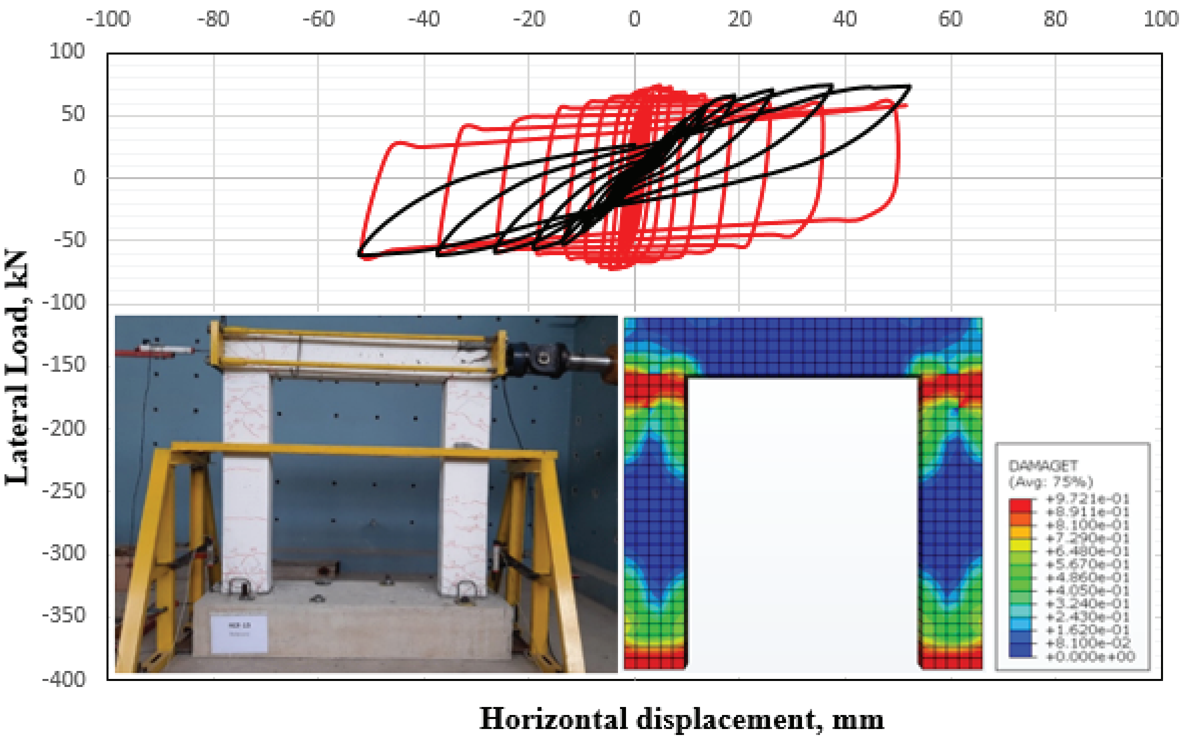


Figure 3. Determining the reliability of the finite element modelling method.

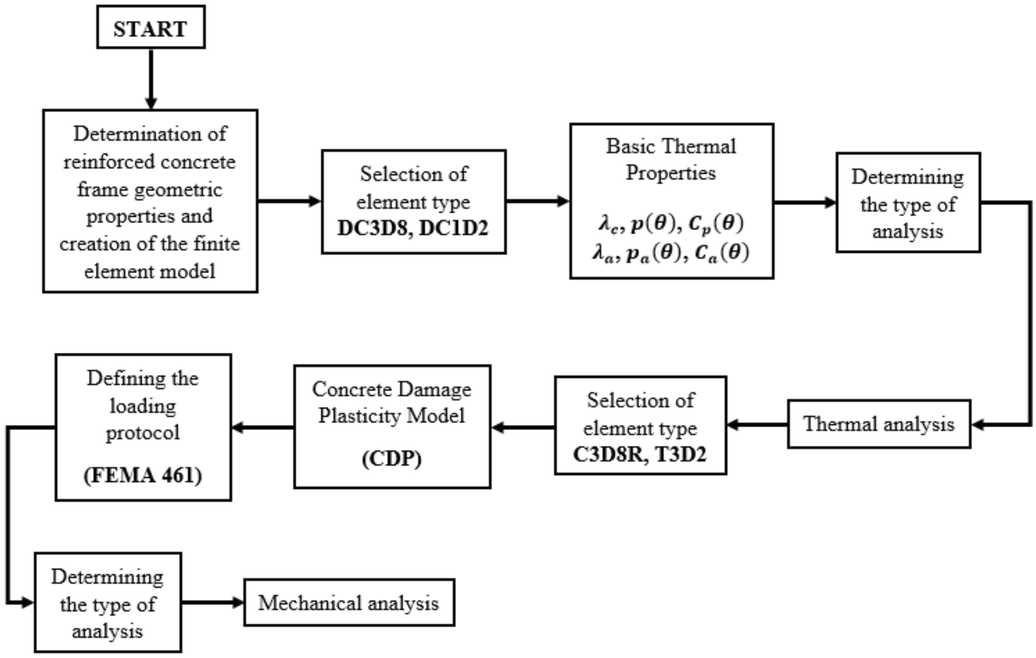


Figure 4. The process of evaluating the cyclic lateral load performance of reinforced concrete frames after high temperature.

reinforced concrete frame modelled using 35423 elements with 50mmx50mm dimensions and hexahedral mesh type. Figure 5 shows the models of the reinforced concrete frame element divided into finite element networks.

As seen in Figure 5, the foundation part of the frame not modelled in the finite element model of the reinforced concrete frame in order to prevent the increase in the number of finite elements and to shorten the analysis times. Instead, both columns connected to the reference point RP-2 from the nodes at their lower ends by assuming anchored bearings (Figure 3). In the finite element model, the lateral load distributed equally to both end regions of the beam. For this purpose, the nodes at both end regions of the beam combined at the RP-1 reference point on the beam axis and cyclic lateral load applied to this reference point (Figure 5).

SIMULATING THE EFFECT OF HIGH TEMPERATURE

In the numerical analysis of the exposure of reinforced concrete frame test elements to high temperature effects in the ABAQUS program, the 8-node continuous heat transfer

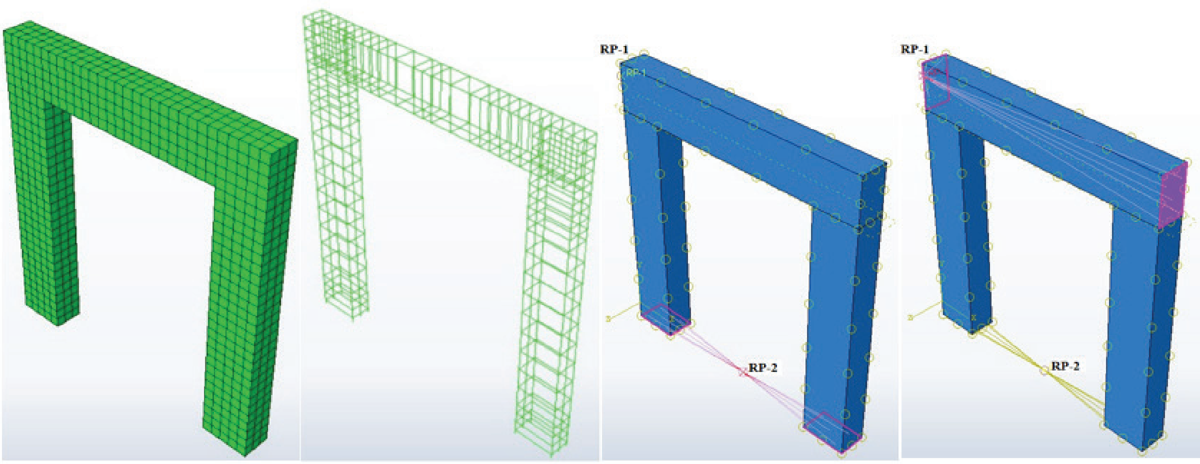


Figure 5. Finite element model of reinforced concrete frame test elements.

element (DC3D8) type, which can best give the results of the effects of high temperature on the concrete material, selected. In the numerical modelling of transverse and longitudinal reinforcement in reinforced concrete frame test elements, a 2-node, heat transfer element (DC1D2) type used. The thermal analysis method chosen to implement the high temperature effect in the ABAQUS program. In this analysis method, the thermal analysis model of concrete and steel defined by three basic thermal properties. These described as thermal conductivity coefficient, density and specific heat.

The thermal conductivity coefficient of concrete (λ_c) was calculated using Equations 1-2 for the upper limit and lower limit, respectively, as specified in TS EN 1992-1-2 (Figure 6).

$$\lambda_c = 2 - 0.2451(\theta/100) + 0.0107(\theta/100)^2 \quad 20°C \leq \theta \leq 1200°C \tag{1}$$

$$\lambda_c = 1.36 - 0.136(\theta/100) + 0.0057(\theta/100)^2 \quad 20°C \leq \theta \leq 1200°C \tag{2}$$

The change in concrete density with temperature was calculated using Equation 3-6, as specified in TS EN 1992-1-2 (Figure 5).

$$p(\theta) = p(20°C) \qquad 20\ °C \leq \theta \leq 115 \tag{3}$$

$$p(\theta) = p(20°C)(1 - 0.02(\theta - 115)/85) \quad 115\ °C < \theta \leq 200 \tag{4}$$

$$p(\theta) = p(20°C)(0.98 - 0.03(\theta - 200)/200) \quad 200\ °C < \theta \leq 400 \tag{5}$$

$$p(\theta) = p(20°C)(0.95 - 0.07(\theta - 400)/800) \quad 400\ °C < \theta \leq 1200 \tag{6}$$

The change of concrete specific heat with temperature was calculated using Equation 7-10, as specified in TS EN 1992-1-2 (Figure 6).

$$C_p(\theta) = 900 \qquad 20\ °C \leq \theta \leq 115\ °C \tag{7}$$

$$C_p(\theta) = 900 + (\theta - 100) \quad 100\ °C < \theta \leq 200\ °C \tag{8}$$

$$C_p(\theta) = 1000 + (\theta - 200)/2 \quad 200°C < \theta \leq 400°C \tag{9}$$

$$C_p(\theta) = 1100 \qquad 400\ °C < \theta \leq 1200\ °C \tag{10}$$

The thermal conductivity coefficient of steel (λ_a) was calculated using Equation 11-12 as specified in TS EN 1993-1-2 (Figure 6).

$$\lambda_a = 54 - 3.33x10^{-2}\theta_a \quad 20°C \leq \theta < 800°C \tag{11}$$

$$\lambda_a = 27.3 \quad 800°C \leq \theta \leq 1200°C \tag{12}$$

Steel density is independent of temperature and can generally be taken as p_a = 7850 kg/m^3 (Figure 6).

The change in the specific heat of steel with temperature was calculated using Equation 13-16, as specified in TS EN 1993-1-2 (Figure 6).

$$C_a = 425 + 7.73\ x\ 10^{-1}\theta_a - 1.69\ x\ 10^{-3}{\theta_a}^2 + 2.22\ x\ 10^{-6}{\theta_a}^3 \quad 20\ °C \leq \theta < 600\ °C \tag{13}$$

$$C_a = 666 + 13002/(738 - \theta_a) \quad 600\ °C \leq \theta < 735°C \tag{14}$$

$$C_a = 545 + 17820/(\theta_a - 731) \quad 735\ °C \leq \theta < 900\ °C \tag{15}$$

$$C_a = 650 \quad 900\ °C \leq \theta \leq 1200\ °C \tag{16}$$

High temperatures of 200, 400, 600 and 800 °C applied to reinforced concrete frames for 60, 120 and 180 minutes (Table 1). In order to avoid confusion among reinforced concrete frames, a nomenclature such as Sample Name_ Exposed Temperature_Exposure Time used. In the reinforced concrete frames for which the finite element model was made, the target temperatures were reached within the first hour, and then these target temperatures were kept constant for 60, 120 and 180 minutes (Figure 7).

APPLICATION OF CYCLIC LATERAL LOAD EFFECT

In numerical studies, it is necessary to use material models that reflect idealized states of the real properties of many materials such as concrete and reinforced concrete steel. The elastic properties of the concrete material used in the ABAQUS program given in Table 2.

ABAQUS package program allows the definition of 3 different material models for concrete materials. These materials models; concrete damaged plasticity, concrete smeared cracking and concrete cracking model for concrete. The reinforced concrete frame test element exposed to both pressure and tensile effects during cyclic lateral loading. Cracking behaviour due to crushing and tensile effects due to pressure effects modelled using the CDP (Concrete Damage Plasticity Model) material model in the ABAQUS package program (Figure 8).

While creating the CDP material model, pressure effects calculated in accordance with TS EN 1992-1-2 [34] as specified in Equation 17.

$$\sigma_\theta = \frac{3\varepsilon f_{c\theta}}{\varepsilon_{c1\theta}\left(2+\left(\frac{\varepsilon}{\varepsilon_{c1\theta}}\right)^3\right)} \tag{17}$$

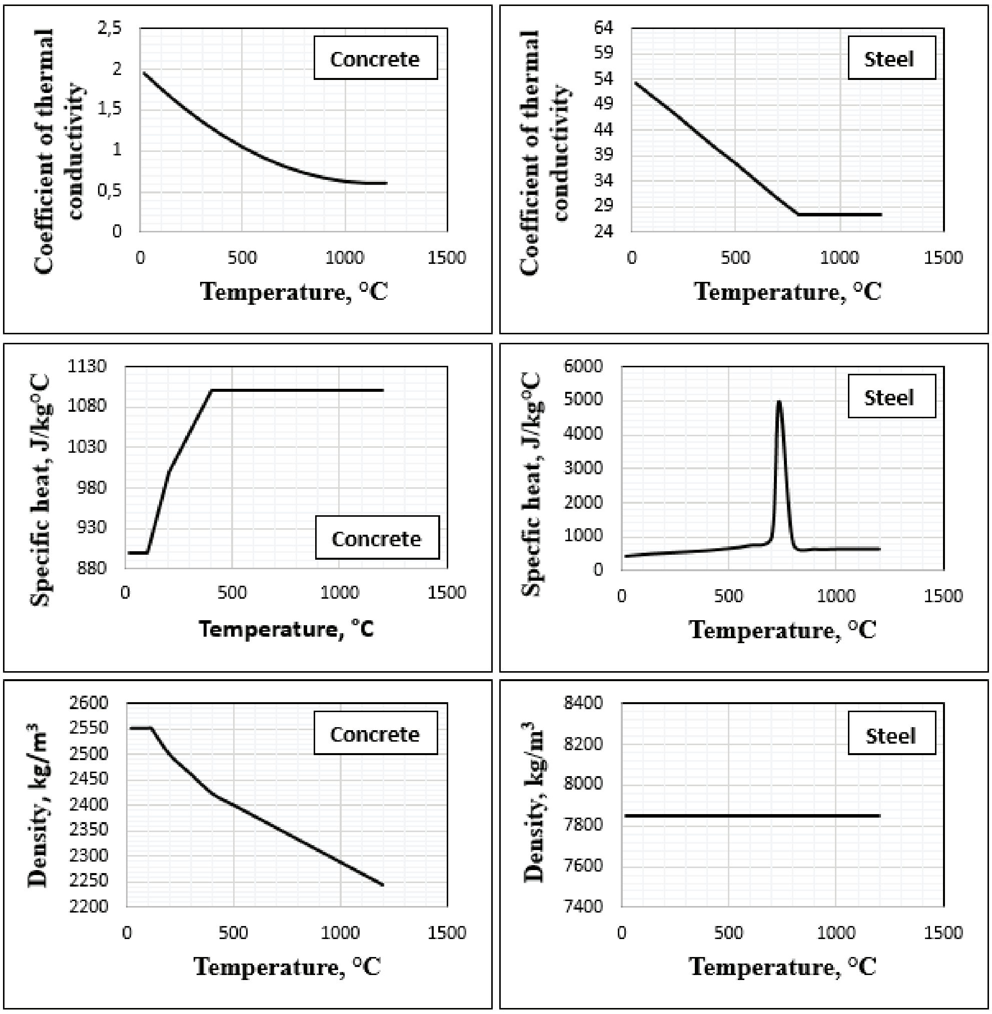


Figure 6. Basic thermal properties for concrete and steel.

Table 1. Nomenclature of reinforced concrete frame elements and different heating conditions

Group No	Sample Name	Temperature Level (C°)
1	RCF_1_200_60	200
	RCF_4_400_60	400
	RCF_7_600_60	600
	RCF_10_800_60	800
2	RCF_2_200_120	200
	RCF_5_400_120	400
	RCF_8_600_120	600
	RCF_11_800_120	800
3	RCF_3_200_180	200
	RCF_6_400_180	400
	RCF_9_600_180	600
	RCF_12_800_180	800
4	RCF_13_REFERENCES	-

Table 2. Elastic properties of concrete material

Young's Modulus (N/mm²)	Poisson's Ratio	Mass Density (kg/m³)
30250	0.2	2400

Here $\varepsilon_{c1\theta}$ refers to the thermal unit deformation of concrete due to different temperatures and, $f_{c\theta}$ refers to the compressive strength of concrete at different temperatures, and these values are given in Table 3 [34].

Tensile effects in the CDP material model applied by reducing the characteristic tensile strength (f_{ckt})of concrete using Equation 18 in accordance with TS EN 1992-1-2. Here $k_{ct}(\theta)$, is the change of the reduction coefficient with temperature and was determined using Equation 19-20.

$$f_{ckt}(\theta) = k_{ct}(\theta) f_{ckt} \tag{18}$$

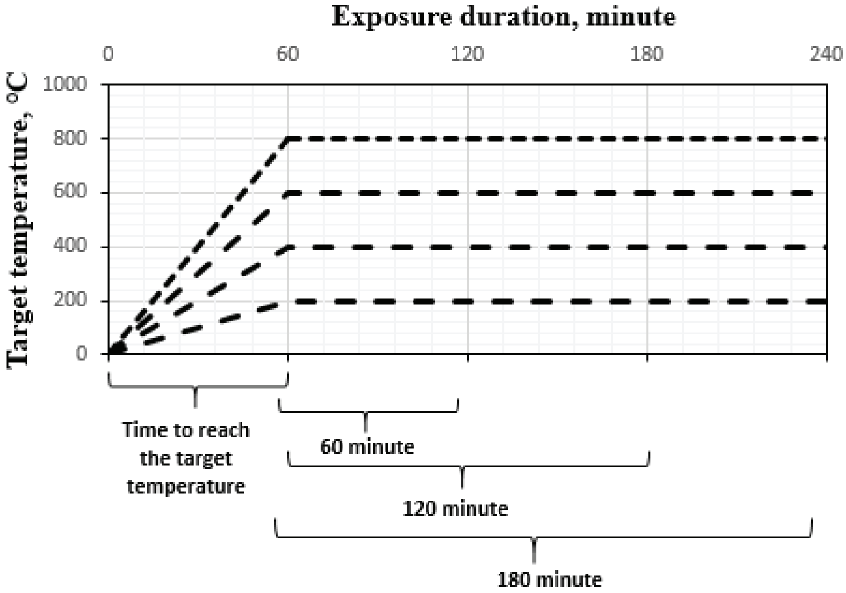


Figure 7. Thermal analysis.

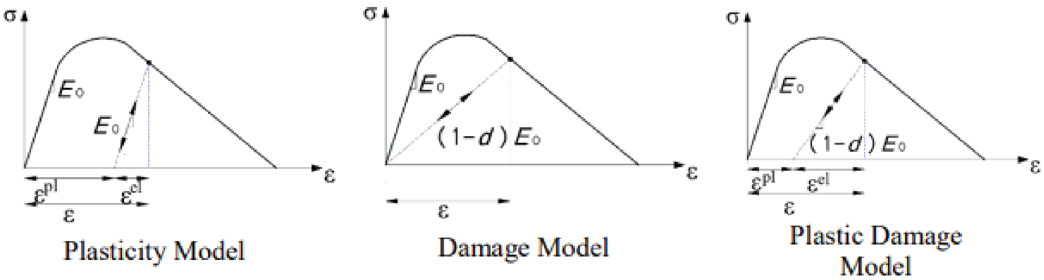


Figure 8. Relationship between damage and plasticity.

Table 3. Determination of the stress – strain relationship of concrete under high temperature

$f_{c\theta}/f_{ck}$	$\varepsilon_{c1\theta}$	$\varepsilon_{cu1\theta}$	θ °C
1.00	0.0025	0.020	20
1.00	0.004	0.0225	100
0.97	0.0055	0.0250	200
0.91	0.0070	0.0275	300
0.85	0.0100	0.0300	400
0.74	0.0150	0.0325	500
0.60	0.0250	0.0350	600
0.43	0.0250	0.0375	700
0.27	0.0250	0.0400	800

$$k_{ct}(\theta) = 1.0 \quad 20°C \leq \theta \leq 100°C \tag{19}$$

$$k_{ct}(\theta) = 1.0 - 1.0\,(\theta - 100)/100 \quad 100°C < \theta \leq 600°C \tag{20}$$

The pressure and tensile effects defined in the CDP material model shown in Figure 9a and Figure 9b, respectively.

Rigidity losses occur in the reinforced concrete frame element due to cracks that occur in the concrete under the effect of cyclic lateral loads. In order to represent the decrease in the stiffness of concrete, the stiffness reduction parameters dc for pressure and dt for tension have been defined. The change of stiffness reduction parameters due to inelastic deformation shown in Figure 10a and Figure 10b.

For the CDP material model used in line with finite element modelling, the parameters defined as dilation angle (ψ), eccentricity (e), viscosity (μ), the ratio of the yield stress in biaxial loading to the yield stress in uniaxial loading case f_{bo}/f_{co} must be entered into the ABAQUS package program.

Dilation angle (ψ), is the numerical expression of the volumetric change in the material under shear stress or shear deformation. Changing the dilation angle may result in a more rigid or more elastic material behaviour under the same deformations. In many studies in the literature [22, 23, 24, 27], the dilation angle was chosen between 5° and

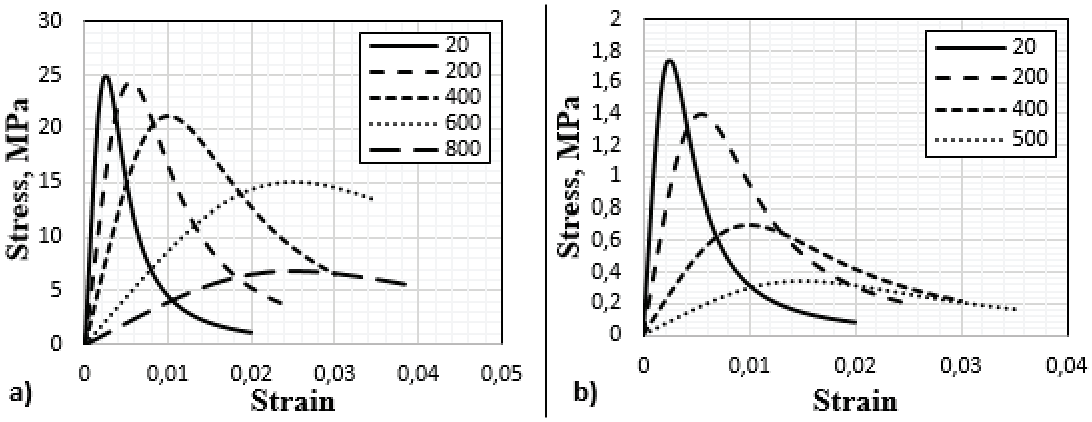


Figure 9. Idealized stress – strain curves of concrete a) For axial compression b) For axial tension.

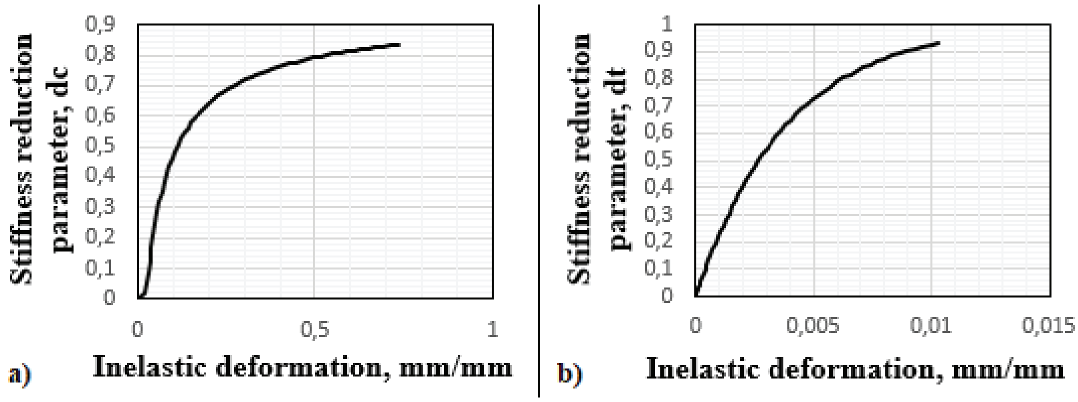


Figure 10. Change of stiffness reduction parameter due to inelastic deformation a) for compression b) for tension.

45°, aiming to be compatible with experimental studies. In this study, many parametric studies performed with different values of the dilation angle for the lateral load - horizontal displacement relationship of the RCF_13_Reference experimental element under repeated cyclic lateral load, and the closest values to the experimental study obtained when the dilation angle was 32° (Figure 11). Increasing the dilation angle causes the material to exhibit a more rigid behavior and gradually increases the lateral load carrying capacity (Figure 11).

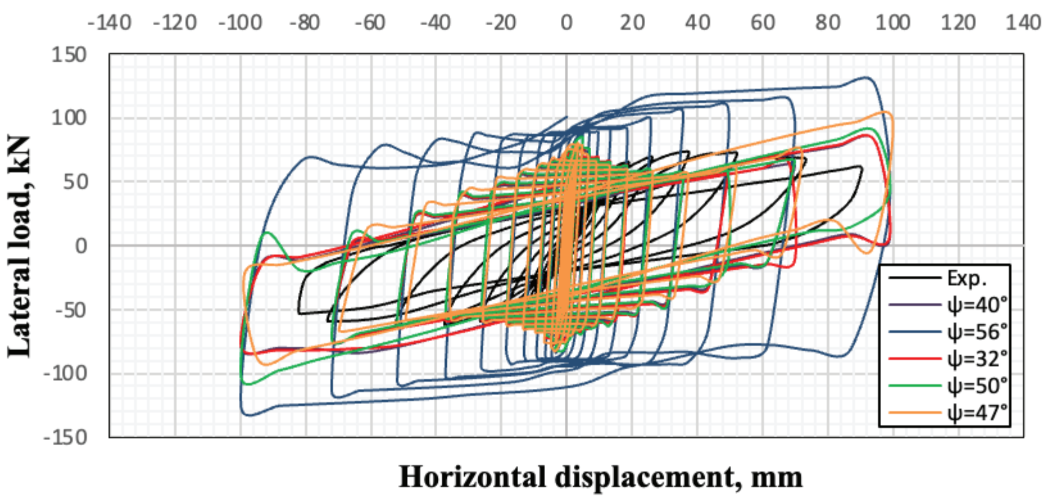


Figure 11. Effect of change of dilation angle on lateral load - horizontal displacement relationship.

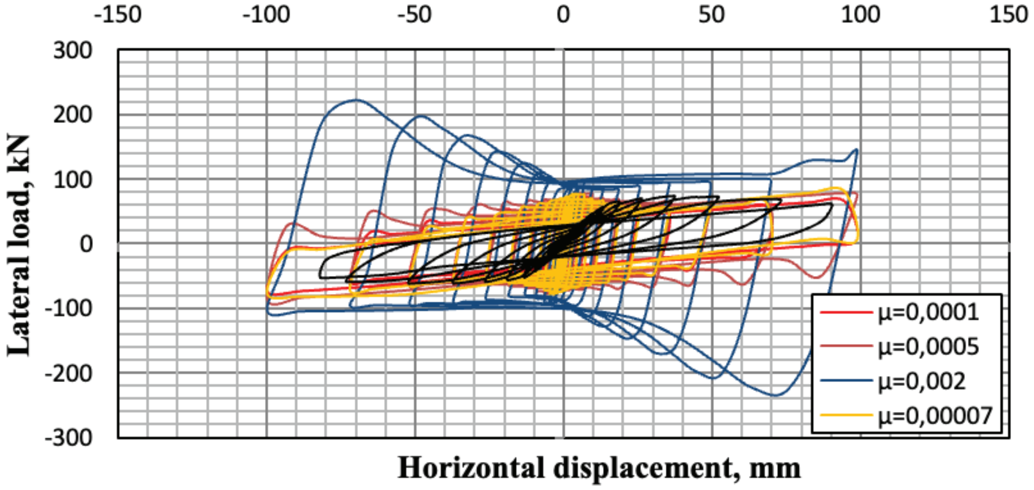


Figure 12. Effect of viscosity change on lateral load - horizontal displacement relationship.

Table 4. Elastoplastic properties of steel material

Young's Modulus (N/mm^2)	Poisson's Ratio	Mass Density (kg/m^3)	Yield Stress (MPa)
210000	0.3	7850	420

Eccentricity (e), is a small positive number that describes the speed at which the Hyperbolic flow potential approaches its asymptote (ABAQUS, 2018). As the eccentricity approaches zero, the flow potential becomes a straight line. The default eccentricity used in the literature [38, 39, 40] and in this study was taken as 0.1.

The ratio f_{bo}/f_{co} defined as the ratio of the yield stress in biaxial loading to the yield stress in uniaxial loading, taken as 1.16 in the literature [41, 42] and in this study.

Viscosity (μ), is the parameter that enables the concrete material equations to be arranged as visco-plastic in numerical analyses. Softening and stiffness losses occurring in cross-sections in material models create convergence problems in analyses, and the viscosity parameter ensures that such problems minimized. The default viscosity value by the ABAQUS package program is 0, and in studies in the literature [39, 41, 43, 44, 45] the viscosity value was chosen between 1 x 10^{-7} and 667 x 10^{-3}. In this study, several parametric studies performed to determine the effect of different values of the viscosity on the lateral load - horizontal displacement relationship of the RCF_13_Reference experimental element, and the closest values to the experimental study obtained when the viscosity was 1 x 10^{-4} (Figure 12). Increasing the viscosity value from 10^{-4} to 10^{-3} minimizes the softening and stiffness losses occurring in the material, thus greatly increasing the lateral load carrying capacity.

In many studies in the literature [36, 37, 46, 47], it is seen that the use of elastoplastic material models in the finite element model of reinforced concrete steel is common. In this study, elastoplastic material model used for steel elements. In this model, after the steel material reaches its yield stress, it undergoes plastic deformations without any increase in stress (Table 4). The steel material model can be defined by entering the elasticity modulus (E_s), poisson ratio (ν), yield stress (f_y) and plastic deformation (ε^{pl}) parameters of the steel into the ABAQUS program (Figure 13).

In the numerical analysis of the application of cyclic horizontal load to reinforced concrete frame test elements in the ABAQUS program; The 3-dimensional and 8-node continuous solid element (C3D8R) type, which can give the

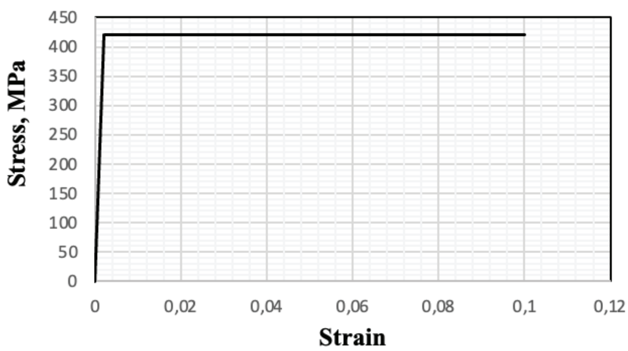


Figure 13. Stress – strain relationship of steel reinforcement.

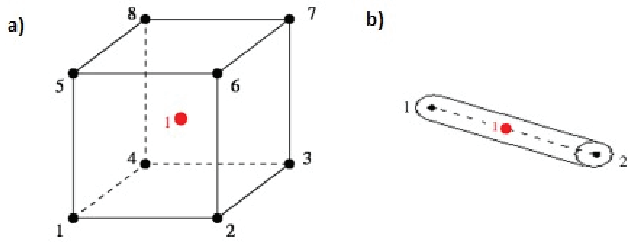


Figure 14. a) C3D8R element b) T3D2 element.

best results regarding load, displacement, plastic deformation, cracking and crushing in concrete, was chosen (Figure 14a). In the numerical modelling of transverse and longitudinal reinforcement in reinforced concrete frame test elements, the 2-node, 3-dimensional rod element (T3D2) type was used (Figure 14b).

It generally accepted that the adherence between concrete and reinforcement materials used in finite element analysis is complete. In this direction, it assumed that there is a complete fit between the C3D8R concrete element used in the study and the T3D2 reinforcement element. Since the reinforced concrete frames considered in the study examined only under the effect of cyclic lateral load after being subject to high temperature effect, the mechanical analysis method chosen. Within the scope of the study, the load protocol specified as quasi-static in the FEMA 461 (2007) regulation used in the application of cyclic lateral loading to reinforced concrete frames for which finite element models made. According to this regulation, the loading protocol should start with a value smaller than the formation of the first cracks in the test element. Accordingly, because of the evaluation, the loading protocol started with a horizontal displacement of 0.24 mm. As specified in the FEMA 461 (2007) regulation, 0.24 mm horizontal displacement was applied for the first 6 cycles, and in each subsequent cycle, this value was increased by 40% and cyclic loading continued (Figure 15).

FINDINGS AND DISCUSSION

Thermal Analysis

The thermal analysis of reinforced concrete frames subjected to different temperatures (200, 400, 600 and 800 °C) for different durations (1, 2 and 3 hours) using the finite element method was used to investigate the temperature distribution within the frame section.

Figure 16 shows the analytical temperature profiles of column and beam web sections of reinforced concrete frame elements that exposed to high temperature effects for 180 minutes on all four sides. It is clear that as the exposure time increases, the temperature increases towards the center of the section. In addition, it seen that the temperature inside the section changes at various depths due to the limited thermal conductivity of concrete.

Lateral Load-Horizontal Displacement Relationship

The strength envelope curves obtained because of thermo-mechanical analyses of reinforced concrete frame elements made in the ABAQUS program given in Figure 17, Figure 18 and Figure 19. In all groups where reinforced concrete frame elements subjected to high temperature effects for 60, 120 and 180 minutes, lateral load carrying capacity decreased depending on the temperature increase. Lateral load carrying capacity of the reinforced concrete frame element exposed to 800 °C for 60 minutes decreased by 28.04% in push and 39.10% in tension. Lateral load carrying capacity of the reinforced concrete frame element exposed to 800 °C for 120 minutes decreased by 35.82% in push and 45.83% in tension. Lateral load carrying capacity of the reinforced concrete frame element exposed to 800 °C for 180 minutes decreased by 38.59% in push and 46.57% in tension (Figure 20).

The decrease in the lateral load carrying capacity of reinforced concrete frame elements exposed to 200 °C for 60, 120 and 180 minutes was obtained as 12.04%, 11.62%, 11.72% in push and 27.38%, 27.53%, 27.75%

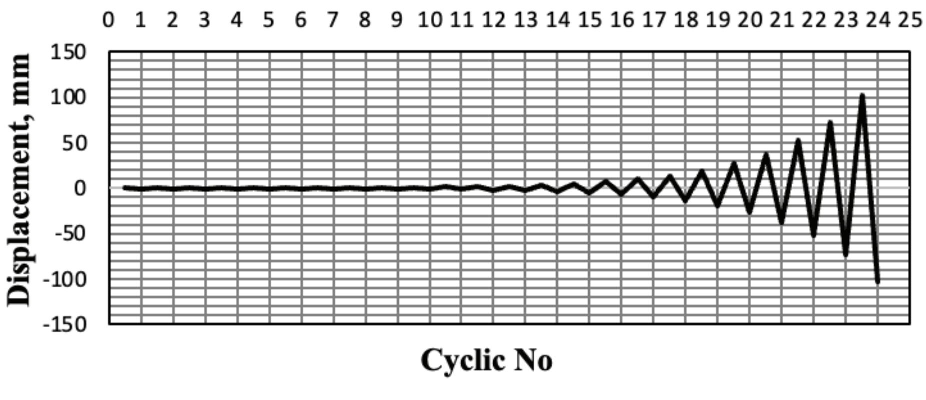


Figure 15. Cyclic lateral loading protocol.

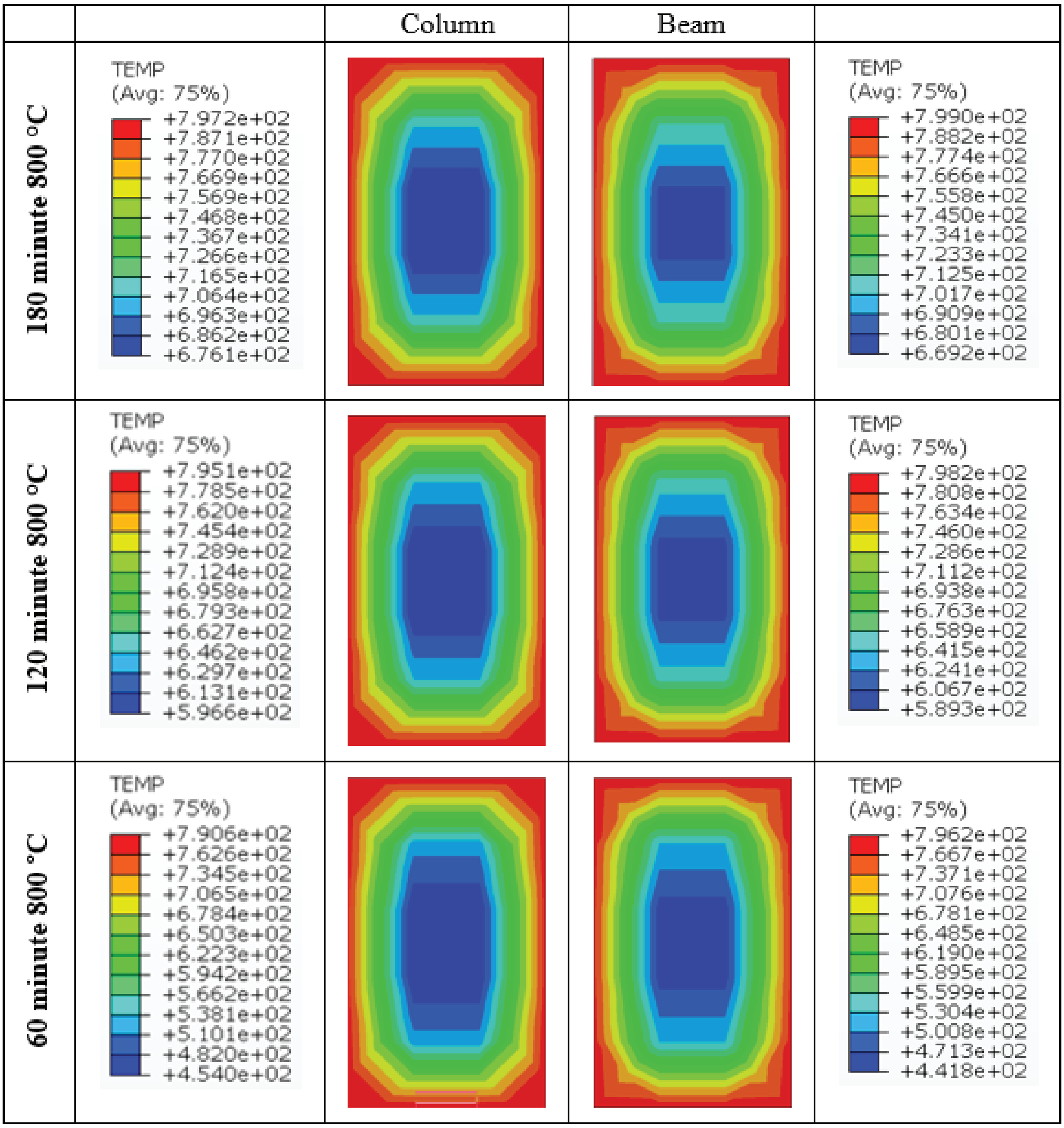


Figure 16. Heat transfer study for a reinforced concrete frame sample where 800°C temperature is applied for 60, 120 and 180 minutes.

in tension, respectively. The decrease in the lateral load carrying capacity of reinforced concrete frame elements exposed to 400 °C for 60, 120 and 180 minutes was respectively 13.27%, 14.58%, 14.70% in push and 31.01%, 31.99%, 32.47% in tension. The decrease in the lateral load carrying capacity of reinforced concrete frame elements exposed to 600 °C for 60, 120 and 180 minutes was respectively 24.41%, 26.42%, 28.23% in push and 34.46%, 37.97%, 41.47% in tension. The decrease in the lateral load carrying capacity of reinforced concrete frame elements exposed to 800 °C for 60, 120 and 180 minutes was 28.04%, 35.82%, 38.59% in push and 39.10%, 45.83%, 46.57% in tension, respectively.

Table 5 summarizes the changes with temperature in the reduction in maximum lateral load carrying capacity of all samples tested in this study compared to the RCF_13_Reference element.

It is extremely important to regain the service capability of structures by restoring the lateral load carrying

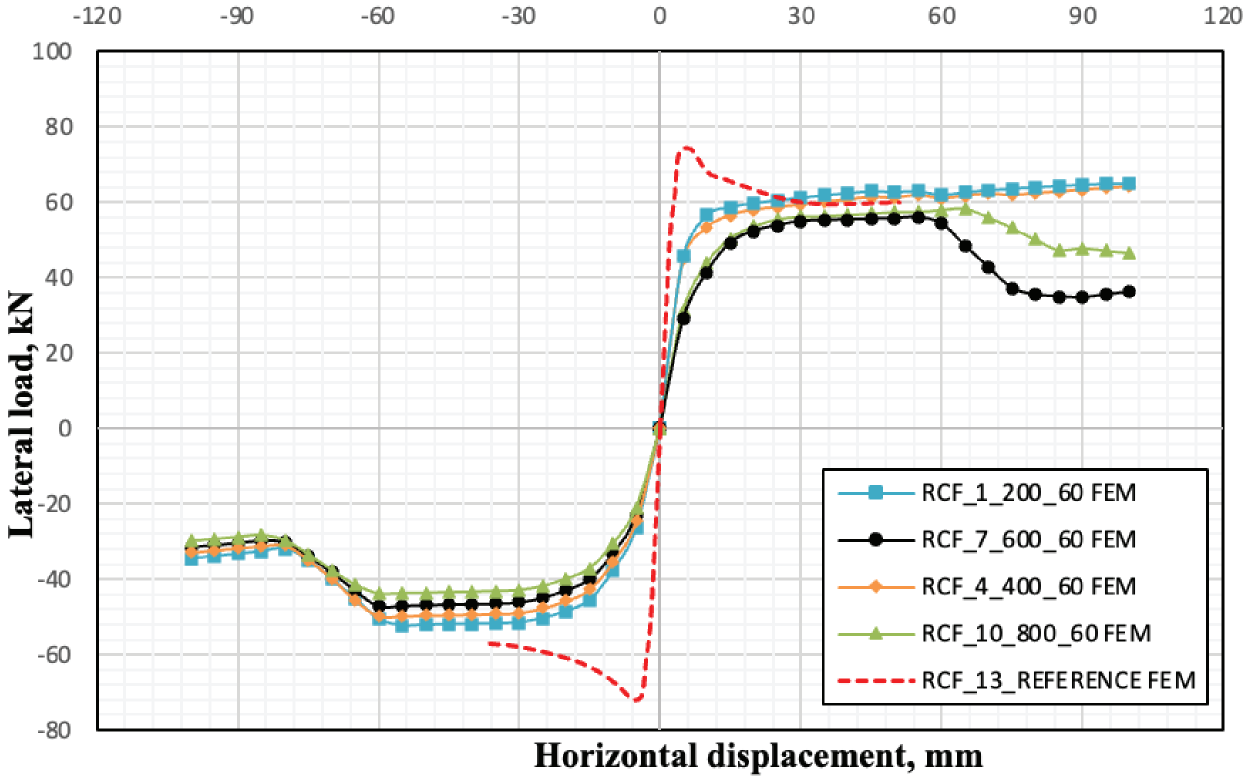


Figure 17. Strength envelope curves (60 minute).

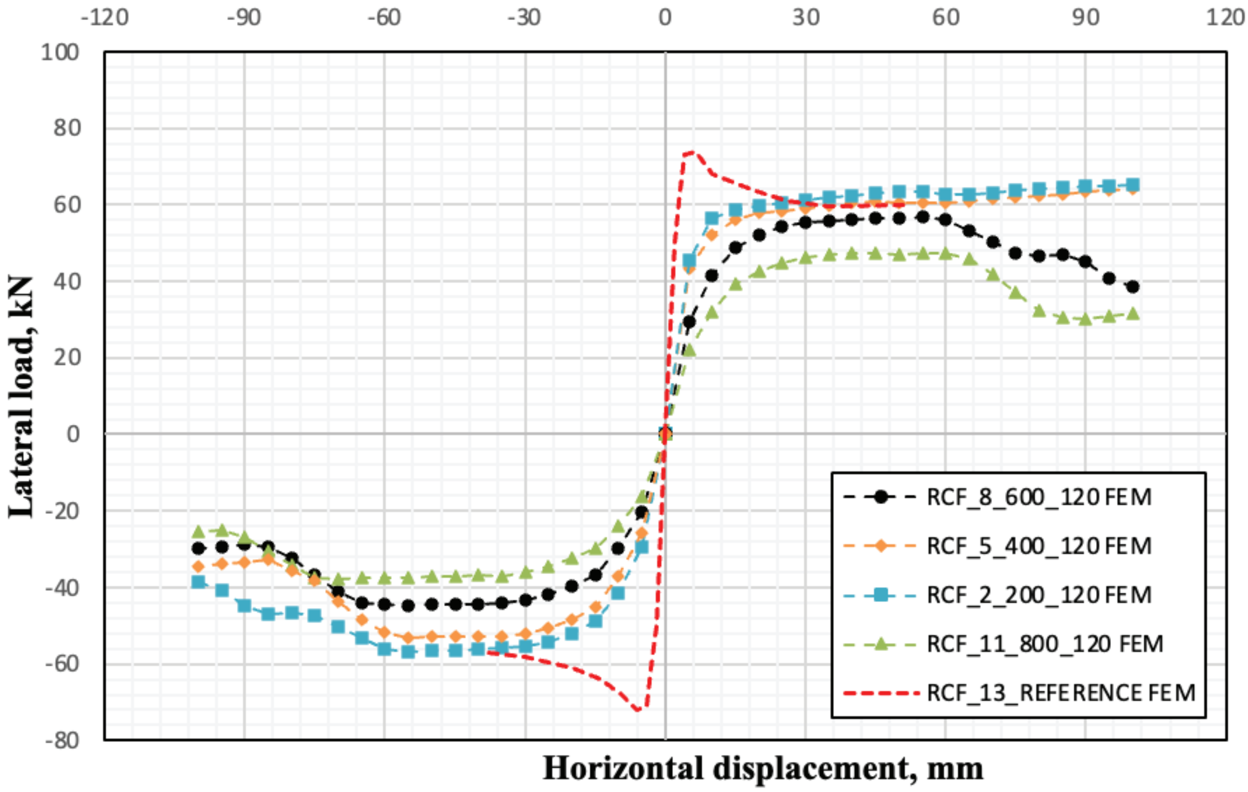


Figure 18. Strength envelope curves (120 minute).

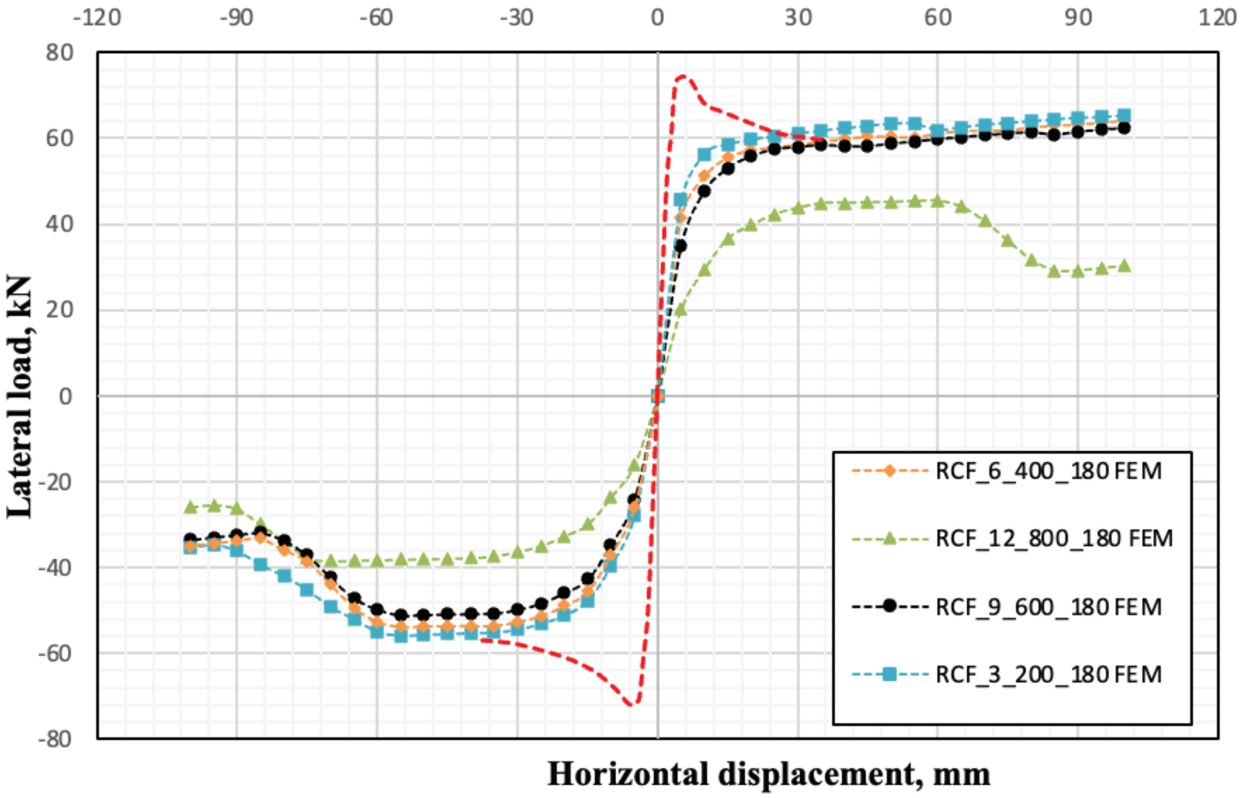


Figure 19. Strength envelope curves (180 minute).

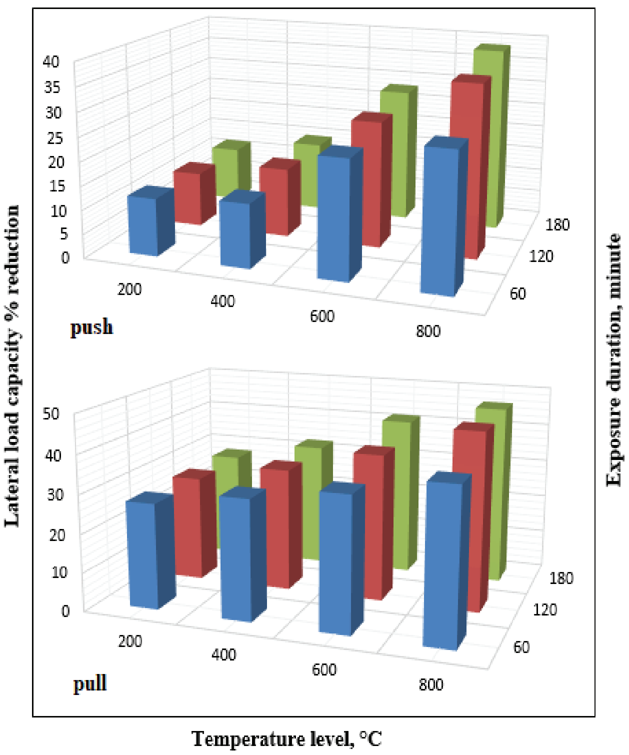


Figure 20. Lateral load carrying capacity - temperature relationship.

performance of structural elements exposed to high temperature effects such as fire. In general, reduced capacity of reinforced concrete structural members poses varying levels of threat to structural safety, requiring performance improvement through repair measures. C - S - H gels in the chemical structure of concrete break down when the temperature rises to 550 °C. Poon et al. [48] determined that if the ambient temperature is below 600°C at the time of fire, concrete can regain its initial strength without any repair provided that appropriate curing conditions are provided. Scanning electron microscopy (SEM) studies have shown that healing results from a series of rehydration processes that regenerate calcium-silicate-hydrate (C-S-H) [48]. When Table 5 is examined, it can be said that the reduction in the lateral load carrying capacity is more than 15% for temperatures of 600 °C and above. In this case, temperatures above 550 °C are critical since a decrease in the lateral load carrying capacity of the frame element below 85% of the total capacity may cause collapse.

Stiffness - Horizontal Displacement Relationship

As a result of thermo-mechanical analysis in all groups where reinforced concrete frame elements were exposed to high temperature effects for 60, 120 and 180 minutes, frame element stiffnesses decreased as the

Table 5. Elastoplastic properties of steel material

Temperature level	60 minute		120 minute		180 minute	
	Push	Pull	Push	Pull	Push	Pull
200	12.04	27.38	11.62	27.53	11.72	27.75
400	13.27	31.01	14.58	31.99	14.70	32.47
600	24.41	34.46	26.42	37.97	28.23	41.47
800	28.04	39.10	35.82	45.83	38.59	46.57

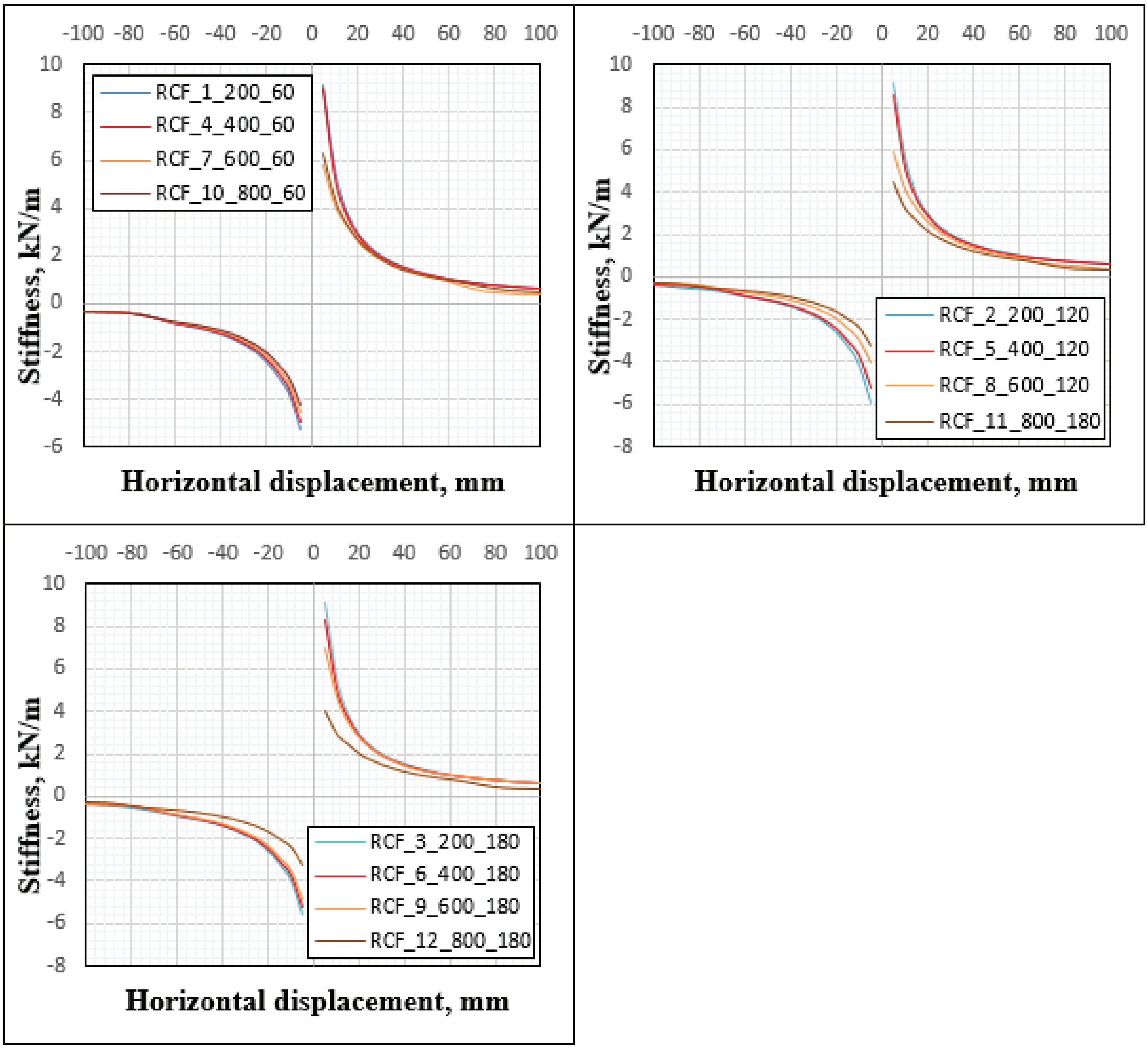


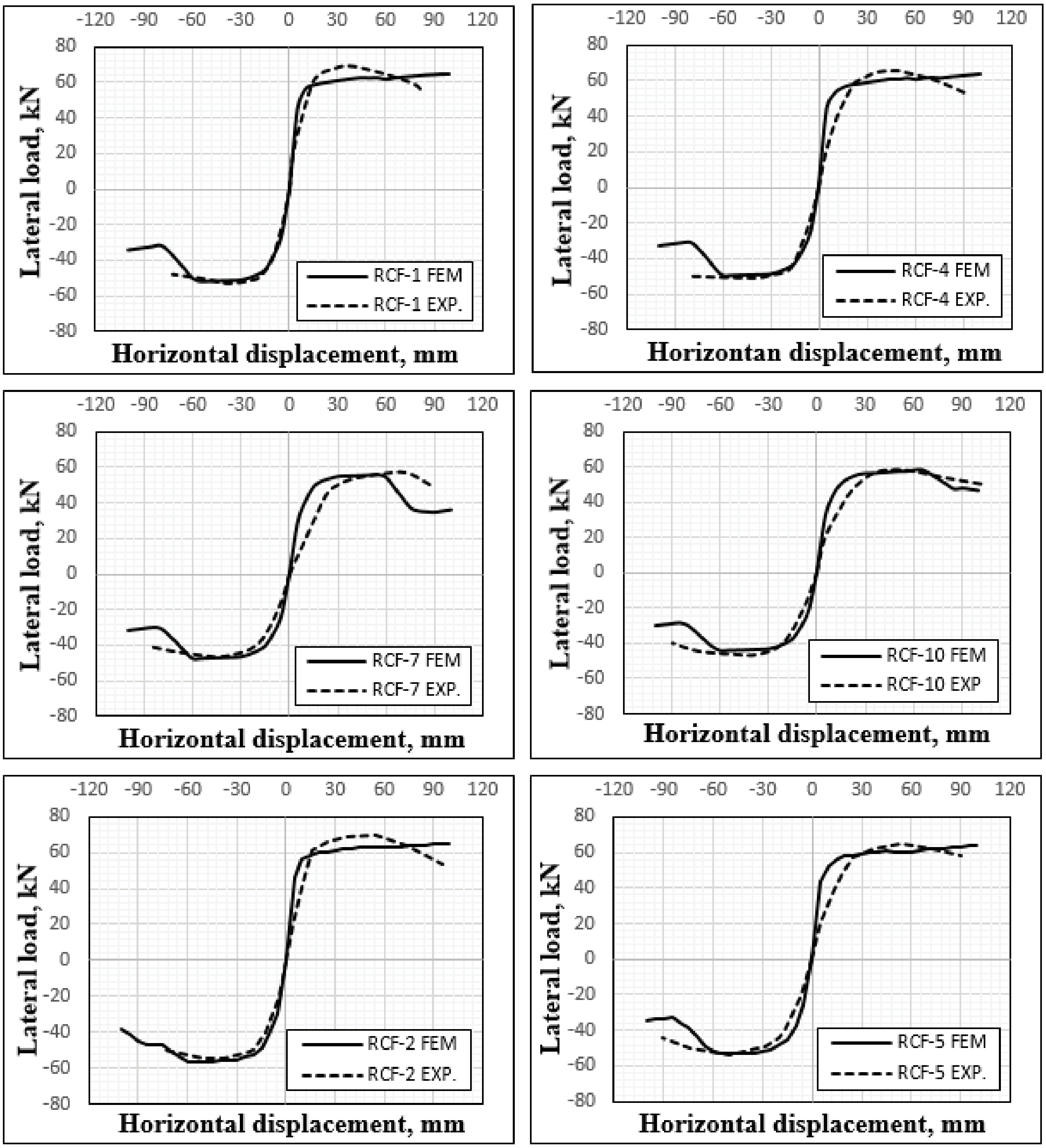
Figure 21. Change of stiffness – lateral displacement relationship with temperature.

number of cycles increased due to the increase in temperature (Figure 21). As a result of thermal analyses, the stiffness of reinforced concrete frames exposed to 800 °C for 60, 120 and 180 minutes, respectively; It was determined that losses occurred up to 76.80%, 82.20%, 83.92% in pushing and 83.24%, 87%, 87.16% in pulling.

The frame elements lost their remaining stiffness in the cyclic loading analyses performed after the thermal analysis. Table 6 summarizes the change in stiffness losses with temperature after thermal analysis of all samples tested in this study compared to the RCF_13_Reference element.

Validation of Numerical Analysis

In this study, a comparison was made using the strength envelope curves obtained from experimental data in order to verify the thermo-mechanical analysis of reinforced concrete frame elements using the finite element method (Figure 22).



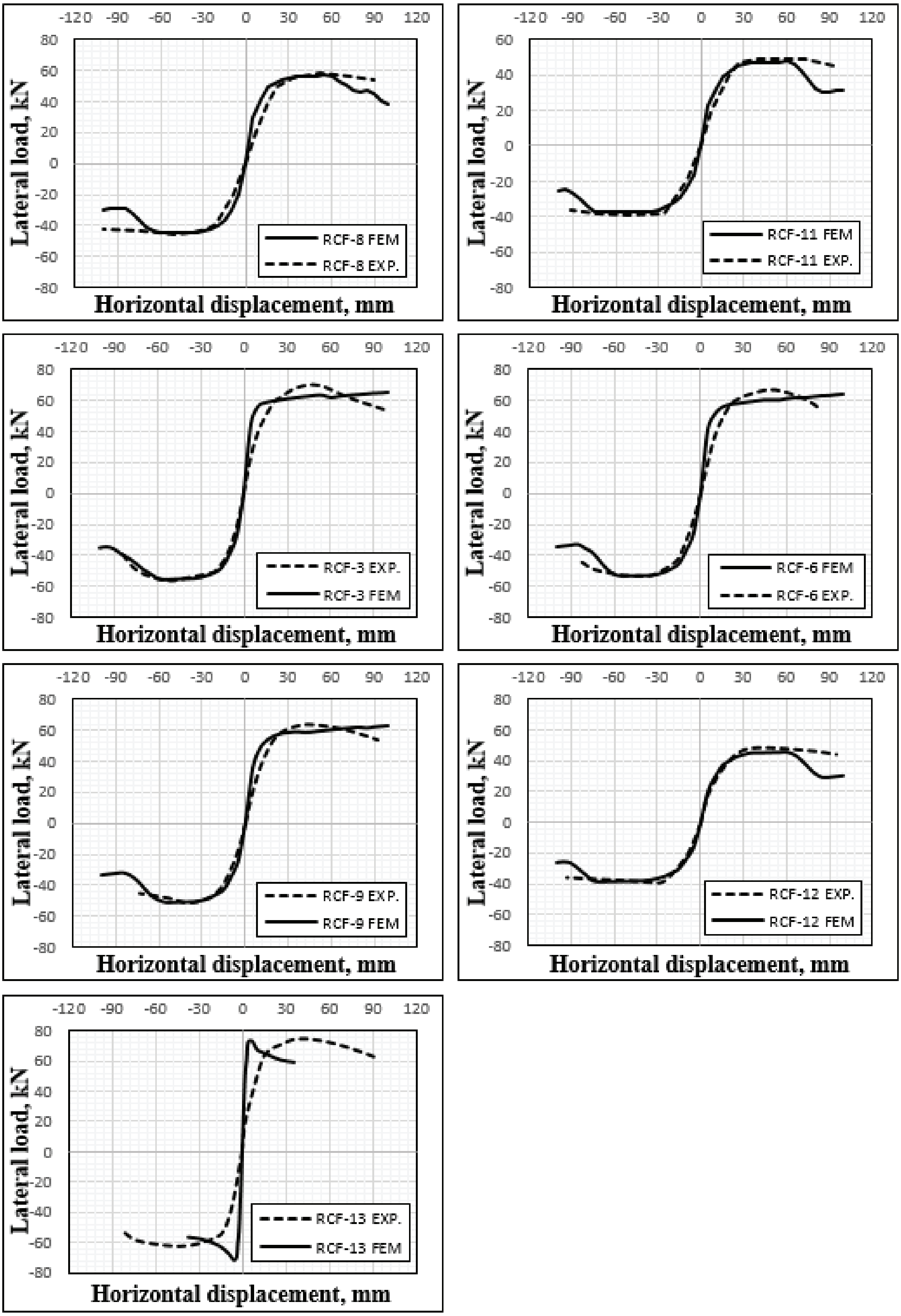


Figure 22. Comparison of strength envelope curves of reinforced concrete frame elements .

CONCLUSION

In this study, the behaviour of reinforced concrete frames exposed to four different temperature levels for three different periods under cyclic load effects numerically examined. In this context, the main purpose of the study is to examine the changes in lateral load carrying capacity and stiffness capacity with temperature, using the finite element method, if reinforced concrete frames that have exposed to high temperature effects due to events such as fire later exposed to lateral load effects such as earthquakes. In addition, in this study, the analysis made using the finite element method verified by comparing them with experimental results. The results obtained in this research are as follows:

- As the level of temperature to which the reinforced concrete frame elements are exposed and the duration of exposure increases, there is a decrease of up to 46.57% in the lateral load carrying capacity. This decrease is highest in element RCF_12_800_180, which exposed to 800 °C for 180 minutes. When similar studies in the literature [14, 15] where reinforced concrete frame members were subjected to cyclic lateral loading after high temperature effect are examined; it is seen that the losses in bearing capacity are in the range of 33.1%-34.9% when the duration of high temperature effect is 120 minutes and support the findings obtained in this study.
- The lowest decrease in horizontal load carrying capacity occurred in the elements exposed to 200 °C. In reinforced concrete frame members exposed to 200 °C, the difference in the duration of high temperature exposure did not change the lateral load carrying capacity much.
- After thermal analysis, the stiffness of reinforced concrete frame elements decreased by up to 87.16%. Losses in stiffness showed an increasing trend in all groups in both push and pull due to the increase in temperature. However, at temperatures of 200, 400 and 600 °C, it was determined that there was an inconsistency in the stiffness losses in the tensile cycles due to the prolongation of the high temperature acting time. The main reason for this is thought to be the opening and closing of cracks in the concrete during the high temperature exposure time.
- Concrete C - S - H gels start to decompose and lose their properties at temperatures above 550 °C. With the decrease in binding properties, the concrete strength of reinforced concrete frames also decreases. When all this situation is evaluated, it is recommended to reinforce the frames where ambient temperatures of 600 °C and above are effective for 1, 2 and 3 hours.
- For C - S - H gels to repair themselves, the temperature should be 550 °C and below. At ambient temperatures of 400 °C and below, the loss of lateral load carrying capacity should not be greater than 15% and it is considered that there is no need to strengthen the frames exposed to the specified temperature and below for 1, 2 and 3 hours.
- The CDP (Concrete Damage Plasticity Model) material model, based on both damage and plastic behaviour, accurately simulates the behaviour of reinforced concrete frames subjected to thermal effects under cyclic lateral loading.

AUTHORSHIP CONTRIBUTIONS

Authors equally contributed to this work.

DATA AVAILABILITY STATEMENT

The authors confirm that the data that supports the findings of this study are available within the article. Raw data that support the finding of this study are available from the corresponding author, upon reasonable request.

CONFLICT OF INTEREST

The author declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

ETHICS

There are no ethical issues with the publication of this manuscript.

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