Influence of Fire-Flame Temperature and Duration on the Behavior of Reinforced Concrete Beams with Construction Joints

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ABSTRACT

Structural members' durability and strength depend on the member's fire resistance. This study simulates the structural response of a reinforced concrete beam with a construction joint exposed to fire. The commercial finite element software ABAQUS was used to validate the laboratory findings. The testing program tested five reinforced concrete beams with the dimensions of (200x300x2700 mm), having identical reinforcing details and a concrete compressive strength (fc'=35 MPa). These beams had a 45° angled connection at the center. Four beams were exposed to fire flames at two temperature levels (600 °C and 800 °C) and for 1.0 and 2.0 hr. periods, respectively. The fifth beam is the control beam that was not exposed to fire. Laboratory results show that the worst exposure on the beam's construction joint was at 800 °C with an exposure period of 2 hrs. This exposure reduces the bond between the joint's two surfaces, creating a slipping effect in which disconnection occurs after loading. After 1 and 2 hours of exposure to fire at 600 °C, the residual flexural strength was 85% and 72% of that of the control beam, respectively. Whereas, beams exposed to fire for 1 and 2 hours at 800 °C showed flexural strengths lower than the control beam at 41% and 28%, respectively. Regarding the modulus of elasticity and compressive strength, they both showed residual values of (63.5, 59.2, 50.9, and 47%), and (28, 25, 19, and 16%), respectively.

Keywords: Construction joint, Finite element method, Elevated temperature, Fire duration.

تاثير مدة ودرجة حرارة التعرض للهب على سلوك العتبات الكونكريتية المقواة بالوصلات الانشائية

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الخلاصة

تعتمد متانة وقوة العناصر الهيكلية على مقاومة الأعضاء للحريق. تحاكي هذه الدراسة تصرف العتبة الخرسانية المسلحة الحاوية على مفصل انشائي المعرضة للحريق. تم استخدام برنامج العناصر المحدودة التجاري ABAQUS للتحقق من صحة النتائج المختبرية. من خلال البرنامج تم اختبار خمس عتبات خرسانية مسلحة بأبعاد (200x2700x300 ملم)، ولها تفاصيل تسليح متطابقة، ومقاومة انضغاط الخرسانة (fc'=35 MPa). كان لهذه العتبات مفصل انشائي بزاوية 45 درجة في المركز. تم تعريض أربع عتبات الى مستويين من درجات الحرارة (600 درجة مئوية و800 درجة مئوية) وفترات تعرض 1.0 و2.0 ساعة، على التوالي. النموذج الخامس كان غير محترق. تبين إن أسوأ تأثير على المفصل الانشائي يحدث للعتبة المعرضة لدرجة حرارة 800 درجة مئوية مع مدة ساعتين لأن هذا التسخين يقلل من الروابط بين سطحين في المفصل ويشبه الانزلاق والانفصال بين المفصل بعد تعرضه للحمل. في نمذجة الحريق بعد ساعة وساعتين عند 600 درجة مئوية، كانت قوة الانحناء المتبقية 85٪ و72٪ على التوالي. بينما عند 800 درجة مئوية، كانت 41٪ و28٪ على التوالي وبنفس الفترة الزمنية. فيما يخص المواصفات الميكانيكية للخرسانة المتصلبة، مثل قوتها الانضغاطية ومعامل المرونة. كانت كل من مقاومة الانضغاط ومعامل المرونة المتبقية هي (63.5، 59.2، 50.9 و47) ٪، (28، 25، 19 و16) ٪ على التوالي.

الكلمات المفتاحية: وصلة البناء، طريقة العناصر المحدودة، درجة الحرارة المرتفعة، مدة التعرض للهب.

1. INTRODUCTION

Buildings should have enough structural fire resistance to survive the effects of fire or, at the at least, let residents to escape before strength and/or stability failure occurs since one of the issues they face is the exposure to extreme temperatures**(Kizilkanat et al., 2013; Kadhum and Alwaan, 2013).** Exposure of reinforced concrete buildings to an accidental fire may result in cracking and loss in the bearing capacity of their major components, i.e. slabs, beams, and columns **(Izzet, 2018; Izzat, 2015)**. Even though concrete naturally resists fire, structures made of the material must be constructed in a way that allows them to endure the impacts of a fire. Even if steel reinforcement and concrete lose strength and modulus of elasticity with increasing temperature, structural elements are still required to sustain dead and live loads. Also, when flames are completely formed, the expansion of structural components creates stresses and strains that must be resisted. It is very uncommon for builders to disobey fire safety regulations, leading to potentially expensive mistakes. Throughout the course of its useful life, a concrete building may be subjected to a wide range of stresses, but fire-induced heat would rank among the highest **(Chung and Consolazio, 2005; Mohammed and Fawzi, 2016)**.

With extreme heat, the mechanical properties of concrete weaken, causing the structure to slowly crumble. As an example, the high temperatures produced by the ensuing fire after the incident in 2001 contributed significantly to the collapse of the World Trade Center (WTC). Since then, engineers have realized how crucial it is to consider how high temperatures would affect the design of structural components. As a result, during the last two decades, there have been several experimental investigations of how high temperatures affect the performance of concrete structures**(Kodur and Bisby, 2005; Hussen and Mohammed, 2022)**.

Numerical studies on the behaviour of RC beams in an explosion have been published due to the intricacy of the problem**(Abbasi and Hogg, 2004; Al-Jasmi and Al-Thairy, 2019).** Using ABAQUS, a finite element programme, the refractory performance of simply supported RC beams improved with stranded mesh and polymer mortar (SMPM) was computationally modelled. Concrete's mechanical and thermal characteristics are embraced as follows **(Eurocode 4, 2005)** and others produced by earlier research projects, as well as assessments of temperature and displacement, are carefully taken into account. The numerical results show that the association between displacement and temperature may be accurately replicated by ABAQUS.

 (Gao et al., 2013; Mehrath, 2008) developed a 3D Finite element model to predict the mechanical and thermal performance of RC beams in harsh environments.

(Issa et al., 2014; El-Tayeb et al., 2017) RC beams and frames' reactions to thermal stresses in the same situation were numerically examined using the finite element program ABAQUS. While modeling the beams and frames, the authors took into account the nonlinearity of the materials (including their cracking behavior) and the linearity, nonlinearity, and uniformity of the temperature gradients. The results demonstrate that the trend (linear or nonlinear) of the temperature gradient has a significant impact on the behavior of beams and frames. The nonlinear temperature gradient was thus proposed for inclusion in the study. **(Bentz et al., 2006; Li et al., 2020)** tested the effects of 800 °Con the mechanical properties of high-performance concrete (HPC) and ordinary concrete. There are also published numerical analyses of RC beams with joint members used in construction. **(Cervera et al., 2022; Shaarbaf, 2009)** considered a method for simulating the interaction between the two interacting bodies at the interface, a pair of nodes were positioned at the same starting geometric point. A pair of nonlinear springs, normal and tangential to the interface, were used to connect these two nodes. In the analysis of structures. **(Feng et al., 2018; Niu et al., 2022)** Two-dimensional friction-gap closure interfaces have been the focus of a suggested solution. These methods may be used for a wide range of issues, with various degrees of success depending on how well the real geometry and loading circumstances fit the mathematical model. An iterative strategy was used in which an interface element in two dimensions that may separate or slide and occurs at a finite element's node. This element illustrates a pair of parallel surfaces that may either stay in touch with one another along the normal or separate from one another along the tangent. In addition to elastic difficulties, the given strategy was deemed "general" and "extensible" enough to apply to a wide variety of other situations. **(Desai and Gens, 2002; Dudziak, 2021; El-Borgi and Çömez, 2017)** by introducing the concept of a contact node pair, the authors improved the method for elastic contact problems with friction. Two contacting node displacements and nodal forces were treated as a single variable. Instead of using the flexibility matrix of each body throughout the iterations, the compliance matrix was used at all of the contact node pairings. The compatibility of displacements along the contact area was employed for sticking node pairs in both the tangential and normal directions, while the friction law of Coulomb was used for

sliding nodes. It was conceivable to devise an effective solution for situations involving frictional contact that would save both the computer's storage space and the time it would take to calculate the solution. **(Potts et al., 2001; Al-Sherrawi and Mahmoud, 2018)** used a contact surface between two concretes of various ages as a test case, which can be described as the nonlinear behavior of the system, a two-dimensional interface element with special properties. The shear stiffness properties of the element were determined using a proposed shearing stress-shear deformation (slip) relationship. Normal deformations were also taken into account. A nonlinear finite element program was used to implement the developed interface element. Several specimens were used to validate the proposed interface element, and there was good agreement between the analytical and experimental results. **(Oner et al., 2015; Al-Sherrawi, 2003; Kadhum and Al-Zaidee, 2021)** prepared ten beams made from reinforced concrete and featuring a rectangular cross-section and were tested as basic supports until they broke. Eight beams had horizontal construction joints (HCJ) in a variety of numbers and locations, while the other two beams had none at all. Each of the beams that were put through the flexure test had the same number and kind of longitudinal and transverse reinforcement, as well as identical concrete properties, and they were all designed to fail in the same way. The experiments showed that adding HCJ to reinforced concrete beams reduces cracking and ultimate loads while increasing ultimate deflection but has no significant effect on the value of the beam's deflection at the first crack. The purpose of this study is to determine the effects of fire exposure on the structural behaviour of beams with construction joints at two different temperatures (600–800 °C) and for two different durations (1.0 and 2.0 hours).

2. NUMERICAL ANALYSIS

Finite element analysis was used for the numerical analysis. First, a numerical model is proposed for the pertinent geometrical and material parameters of the reinforced concrete beam model at elevated temperatures. Subsequently, the proposed numerical model was verified against the experimental trials carried out in this investigation **(Abbood and Kharnoob, 2023)**. The finite element approach requires many steps before a model's final volume can be constructed, such as the identification of critical points, lines, and regions, as well as the definition of components. In addition, the inclusion of real constants to each element, and the development of the material model. These processes must be completed before the model's volume can be created. The majority of Finite Element Analysis (FEA) software packages provide nodal, element solutions for the full issue solution, making it easy to compute and visualize any unknown parameter **(Umran, 2002; Alia et al., 2019; Logan, 2012)**.

Because FEM can be used for a wide range of real problems with little training and adaptable software, it has recently seen a surge in popularity.

Many typical 3D structural investigations have been made affordable because of the significant developments in computer-aided and finite element methods during the previous three decades. ABAQUS, a suite of engineering simulation applications, makes use of FEM **(Abaqus, 2014).**

The completion of the modeling process requires the generation of three distinct parts. A tetrahedral element was utilized to simulate the beam comprising of two sections, which facilitated the creation of a construction joint in the initial part. The second part of the project involved the modeling of steel reinforcement using a 3D process. The third section of the

presentation discussed the process of generating steel supports, which were modeled as 3D-solid objects. The components are depicted in detail in **Figs. 1** to **3**.
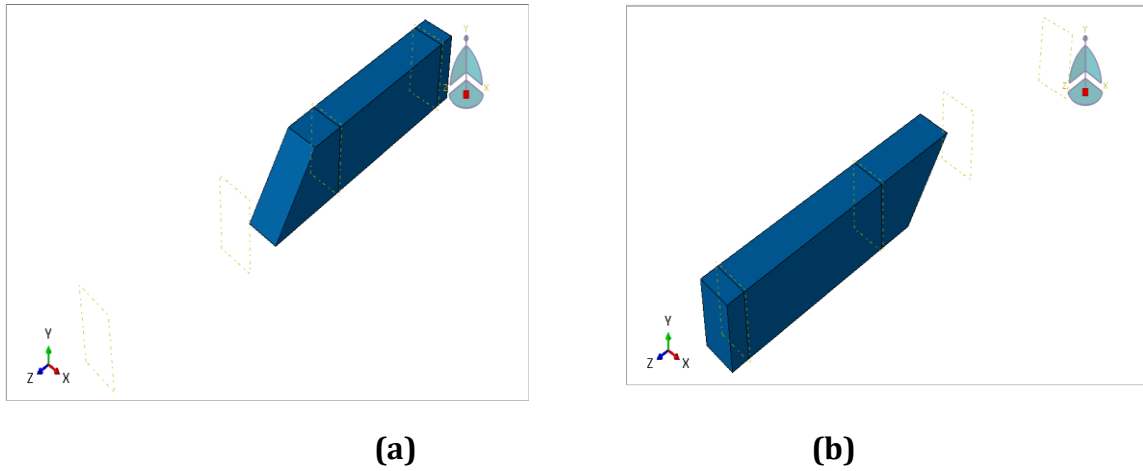

| (a) | (b) |

Figure 1. Volume Parts of the Concrete Beam (a) Right side and (b) Left side.
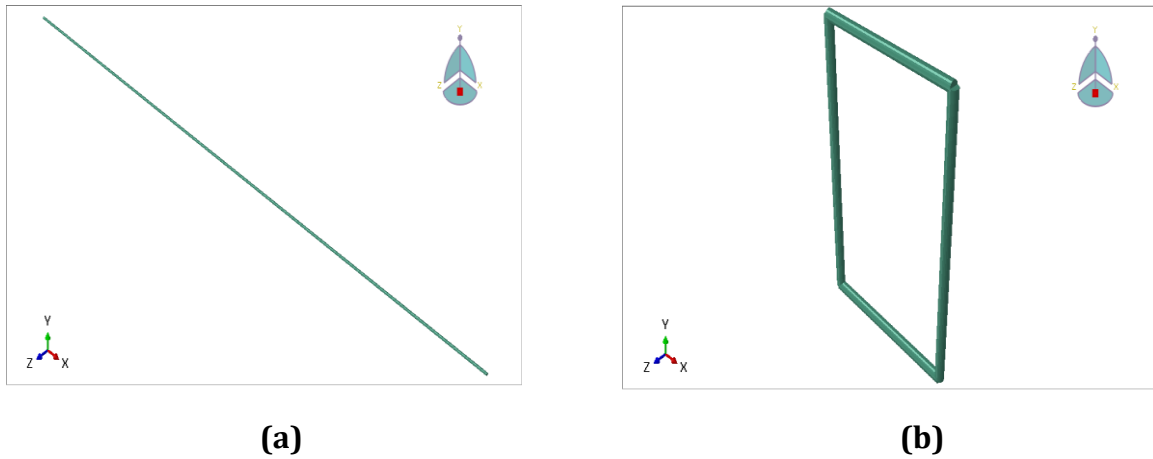

| (a) | (b) |

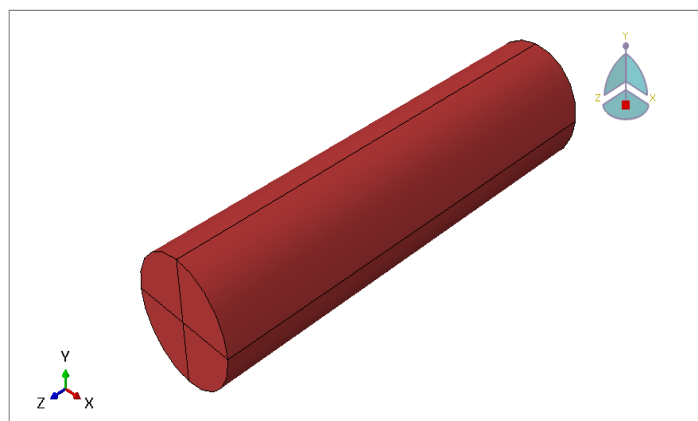**Figure 2**. (3D) Parts of the Steel Reinforcement longitudinal (a) reinforced steel and (b) stirrup.

Figure 3. (3D) Parts of the Steel Rod.

The process involved determining the material properties after creating the specimen parts and selecting suitable interaction methods between all components of the specimen. The user suggests utilizing an assembly module method, as shown in **Fig. 4**, to execute the acquired model geometry through the creation of part instances.
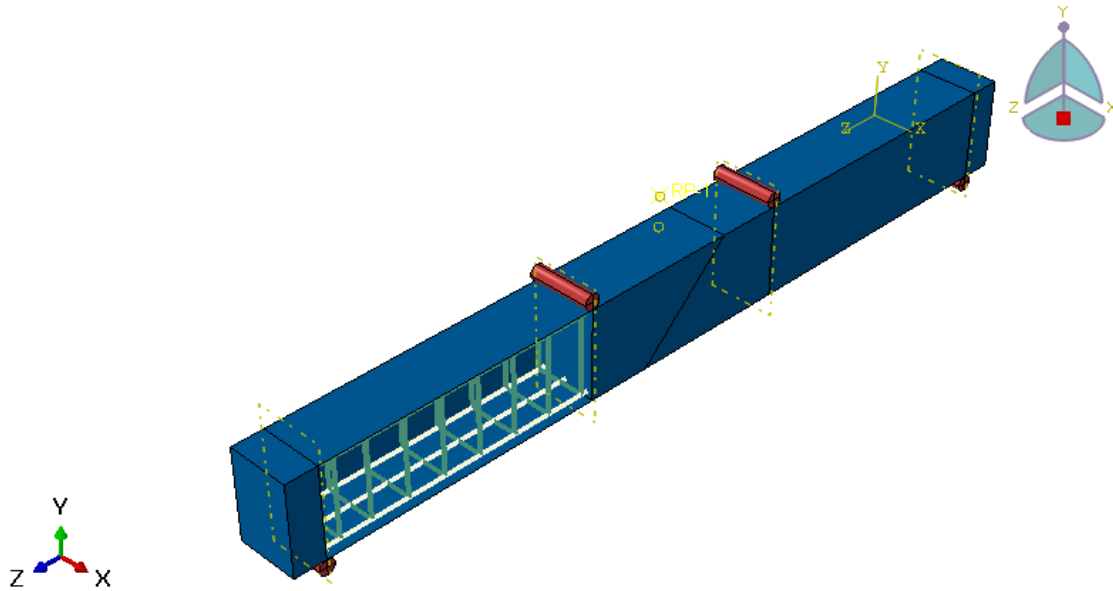

Figure 4. Model Assembly in ABAQUS.

The test procedure utilized to evaluate the models was the flexural load test. All specimens shown in **Fig. 5** exhibited applied displacement as a result.
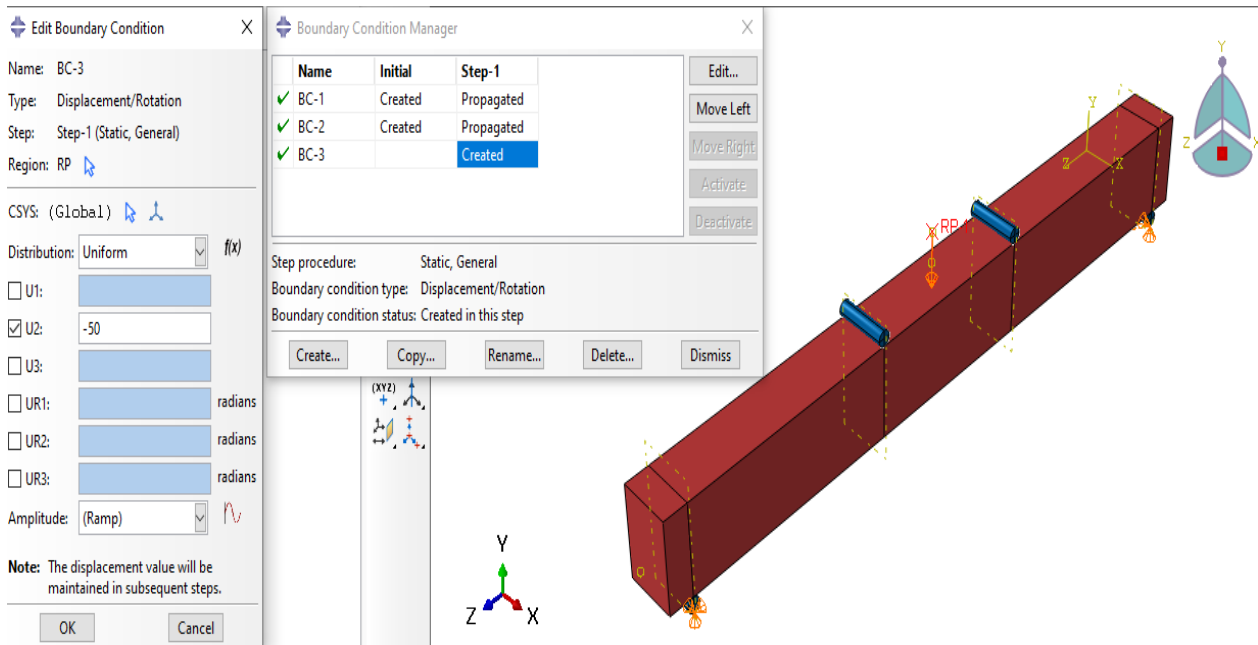
Figure 5. Displacement control application.

During a simulation for the specimens, two surfaces are joined by applying embedded region limits, and all nodes on the combined surface are forced to move at the same rate as the closest point on the master surface. This is done so that the simulation can accurately represent the behavior of the specimens. In **Fig. 6**, the model's formulation of the linking interaction states is shown. The building joint concrete beam interface was modeled in a second step using a surface-to-surface contact, as shown in **Fig. 7**. The normal property can restrict the amount to which the slave nodes are penetrable and to which tensile stress is passed across the interface by creating a hard contact connection between the nodes. Touching two surfaces has negative effects in terms of the tangential characteristic. To model the effects of flames on beam surfaces, we employed the surface film constraint in ABAQUS/Explicit, as shown in **Fig. 8**.
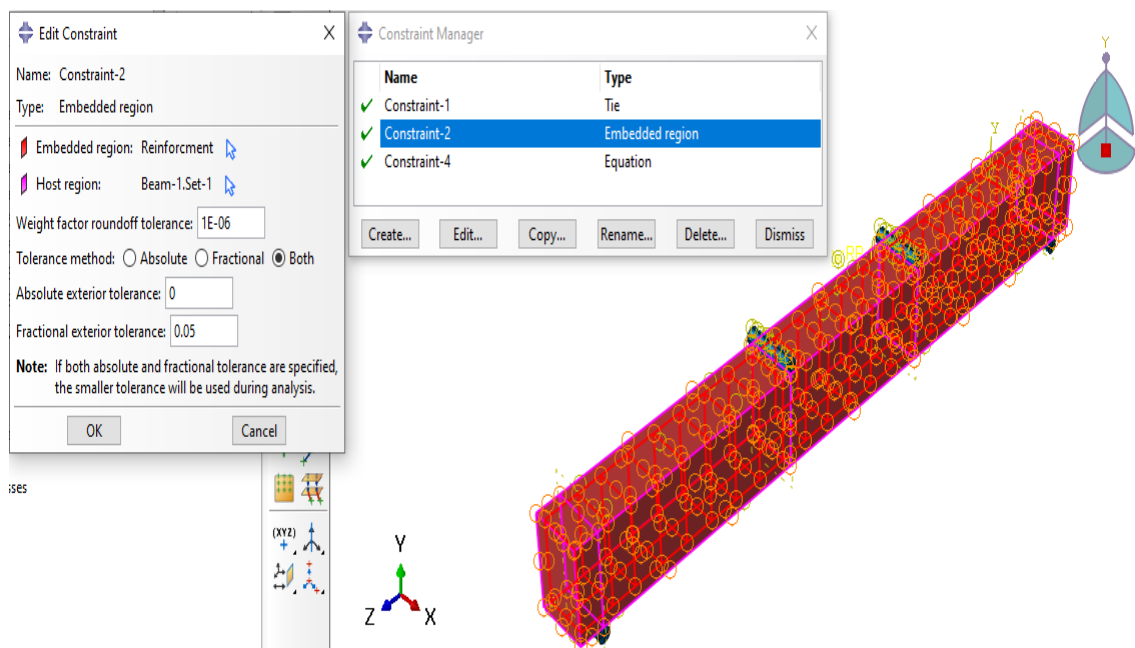

Figure 6. Formulation of connecting interaction -states in the model.

The integration rules provided by these components are derived from the specimen's experimental reaction. Reinforcing steel may be represented in 3D using either a solid, a beam, or a truss element. Using solid components is not preferred because of the high computational cost involved. Truss elements are employed and modeled as an embedded element, and their attachment to concrete is assumed to be totally bonded since reinforcing bars do not give particularly high bending stiffness. The finite element models for the concrete beam, support rods, all use a linear brick hexahedral C3D8R element, which is a continuum element (C) with 3D eight nodes (8) and reduced integration (R). Both simple and complicated nonlinear studies including stress, plasticity, and massive deformations may make use of the solid components as shown in **Fig. 9**. In contrast, a linear 3D two-node truss element (T3D2) with three degrees of freedom at each node was employed to model the embedded reinforcing bars and stirrups as shown in **Figs. 10 and 11** respectively.
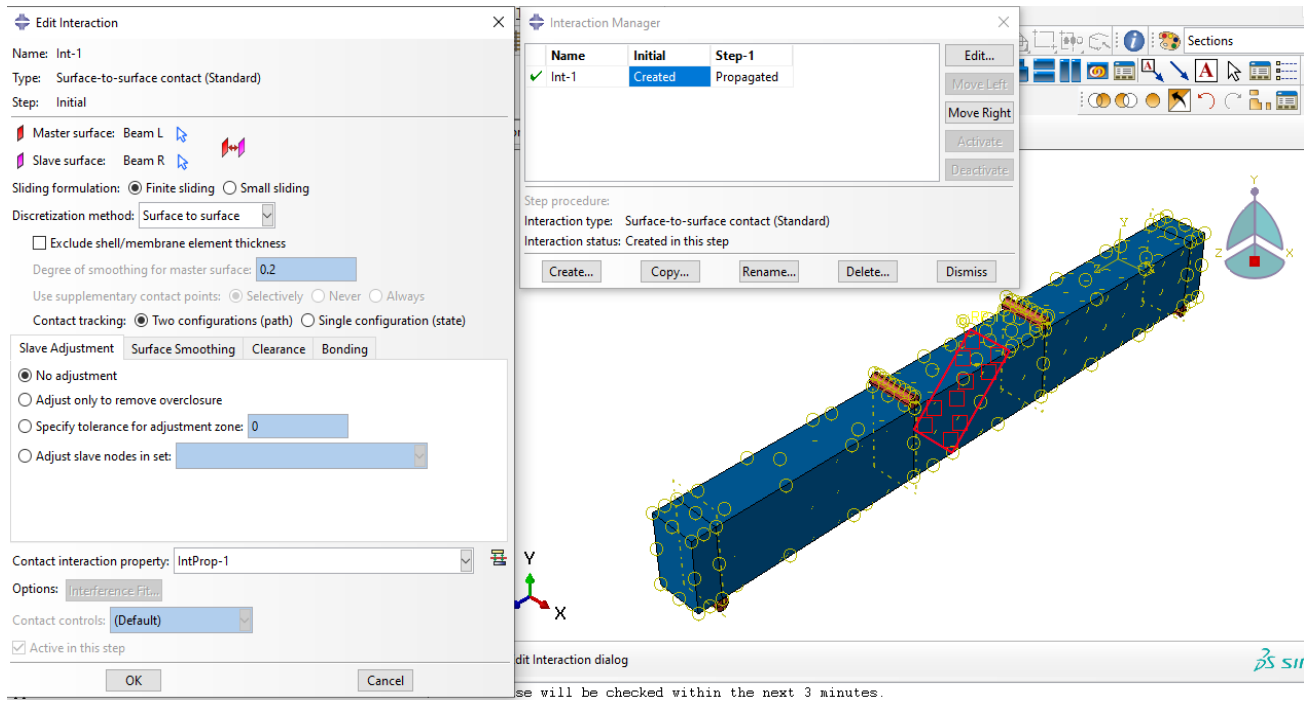
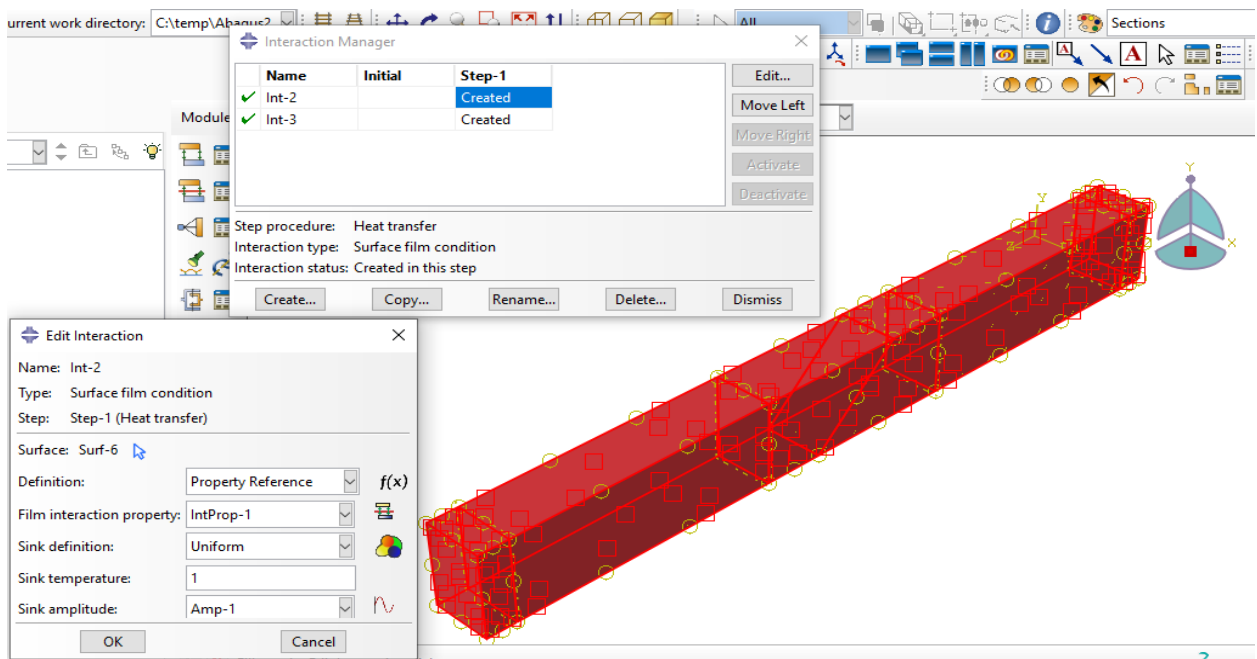**Figure 7.** Contact interaction, (Surface to surface contact), for construction joint.

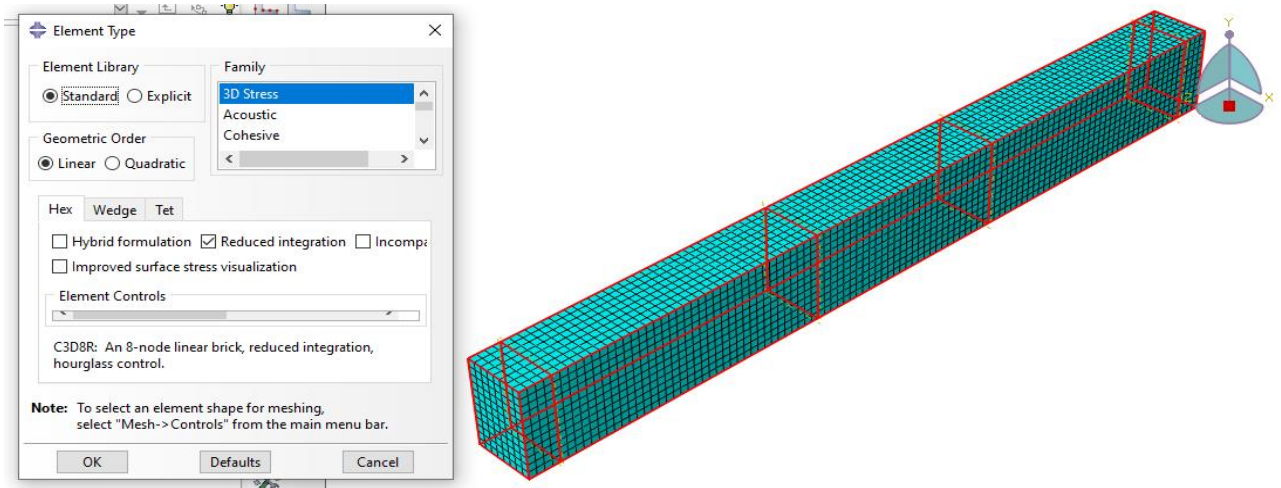**Figure 8.** Interaction for the Elevated Temperature.
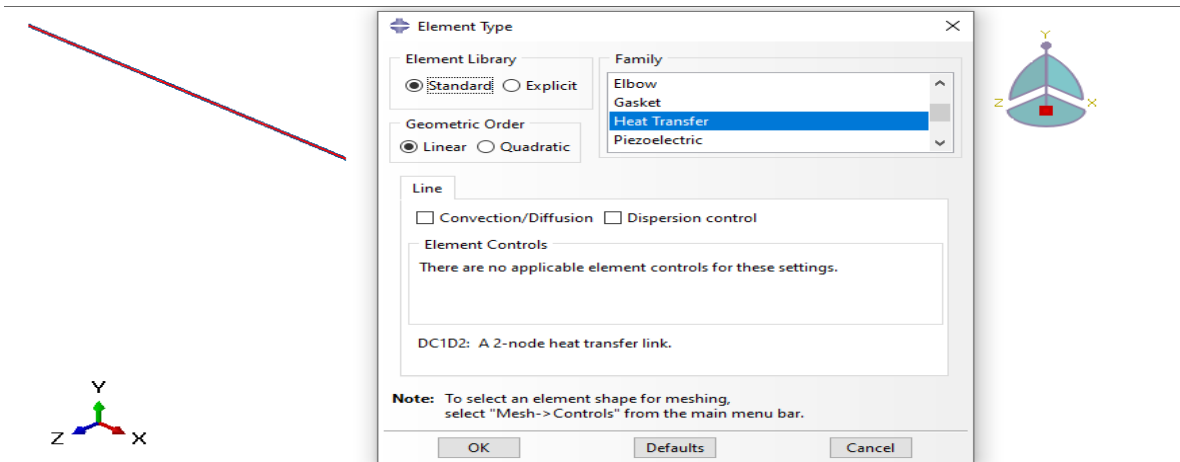
Figure 9. Meshing of Modeled Solid Beam Part.

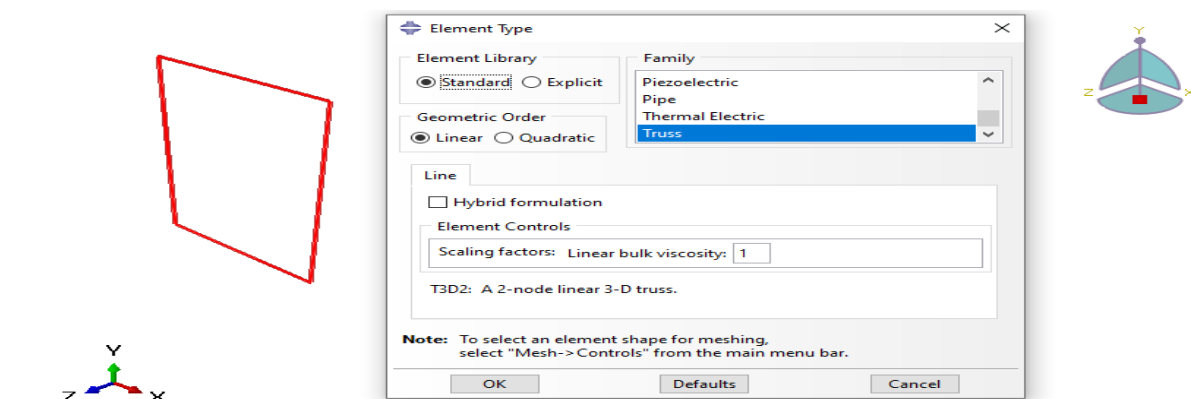

Figure 10. Meshing of Modeled Steel Main Reinforcing

Figure 11. Meshing of the Model Steel Stirrup Reinforcing Parts.

3. NUMERICAL APPLICATIONS AND DISCUSSIONS

3.1 Numerical Module Validation

When beams with construction joints were put through the simulation of being exposed to fire flame, a numerical analytical model was utilized to predict how they would behave. The conclusions of the numerical analysis and the experimental data were compared in terms of the ultimate load, the behavior of the load-deflection relationship, the load-strain response, and the layout of the cracks **(Abbood and Kharnoob, 2023).**

3.1.1 Ultimate Deflection and Ultimate Load Capacity.

The findings of the nonlinear finite element analysis (FEA) are compared with the experimental data, which includes the maximum load and the amount of deflection that occurred. The specimens were named following a specific system consisting of three parts: the first part consisted of the letter F referring to the fire; the letter was followed by a number referring burning period (1 hour and 2 hours); the last part of the name was a number referred to the burning temperature (600 ℃ and 800 ℃) and this number was split from the main name by (-). **Table 1** summarizes the information of the beam utilized in this study. one of the beams named C left without burning as a control beam. of the other two groups consist of two beams, one of them was burned for 1hr named F1 and the other was burned for 2 hrs which is named F2, after exposing to high temperature which was, 600 ℃, and 800 ℃. **Table 1** also shows details of the burning temperature of each beam. **Table 2** provides a summary of the ultimate load as well as the deflection. The results of the FEA and the tests reveal that they agree with one another quite well. There was never more than a fourteen percent chance of an error occurring with any one specimen. The comparison demonstrates that there is a striking degree of consensus on the directional bias of the final deflection. The amount of inaccuracy that occurred with the sample did not go over 4%. Both the percentage differences found via experimentation and those found in the data are within acceptable levels. This is often the consequence of making inaccurate assumptions during the numerical analysis, such as the fact that the approved solution approach in the ABAQUS program assumes complete interaction between the concrete and steel rebar components. Another common cause is that the numerical analysis was performed using an outdated version of the software. The load–deflection curves measured at the middle of the specimen are shown in **Figs. 12** to **16**, where we can observe a comparison between the experimental and calculated versions of these curves. When comparing the deflection response between the experimental and the numerical work, , there is often great agreement. It was also found that the calculated load-deflection curves were more consistent than the measured ones. Finite element analysis relies heavily on the assumptions of material homogeneity and complete boundary conditions for the concrete and steel reinforcement.

Table 1. Details of the modeling beams

Beam Specimens designation	Burning Temperature ℃	Period of Exposure (hr)
C	Without *	----
F1-600	600	1
F2-600	600	2
F1-800	800	1
F2-800	800	2

Table 2. Comparison Between Experimental and FEA for Ultimate Load and Deflection.

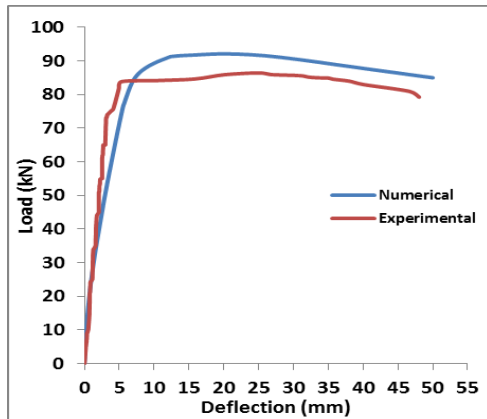| Beam Specimens	Experiment Result		Finite Element Analysis		Ratio	
	Pu (kN)	Δu (mm)	Pu (kN)	Δu (mm)	$\left\|\dfrac{Pu\ Exp\ -\ Pu\ FE}{Pu\ Exp}\right\|_0$	$\left\|\dfrac{\Delta u\ Exp - \Delta u\ FE}{\Delta u\ Exp}\right\|\%$
C	86.4	48	92	50	6	4
F1-600	72.9	51	79	52	8	1
F2-600	61.7	53.6	67	54.4	8	1
F1-800	35.9	56	41	58	14	3
F2-800	24.9	60	23.5	61.9	5	3

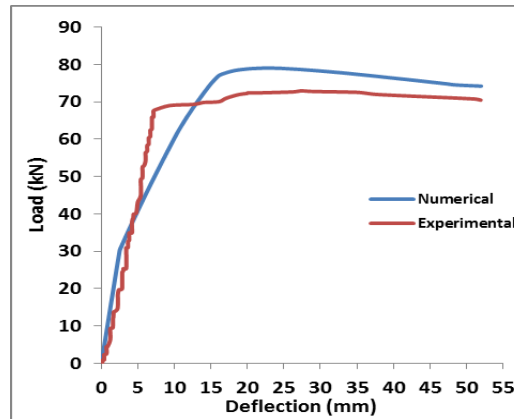**Figure 12**. Experiment and Analytical Load-Deflection (Control).

Figure 13.Experiment and Analytical Load Deflection (F1-600).
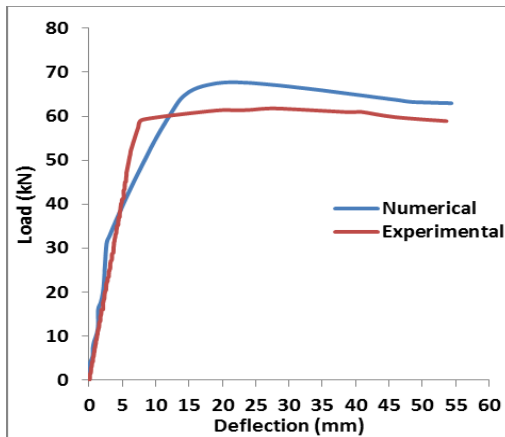

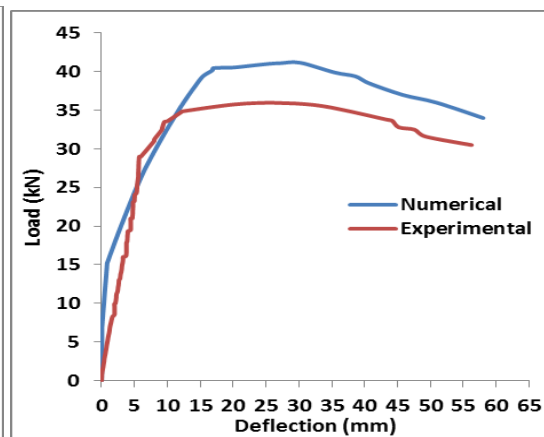**Figure 14**. Experiment and Analytical Load Deflection (F2-600).

Figure 15. Experiment and Analytical Load Deflection (F1-800).

FEA curves, which display the load-deflection response over the elastic zone, are notably stiffer than the corresponding experimental curves. Once the initial flexural cracks for the charred specimens were formed, the beams were immobile in both the upper and downward directions due to the finite element modelling of the supporting lines. Consequently, the tested beams' stiffness is marginally less than that of the FE models.

 In addition, the actual beam's stiffness is reduced as a result of handling and drying shrinkage, both of which induce tiny cracks in the concrete, which in turn reduces the beam's overall strength.
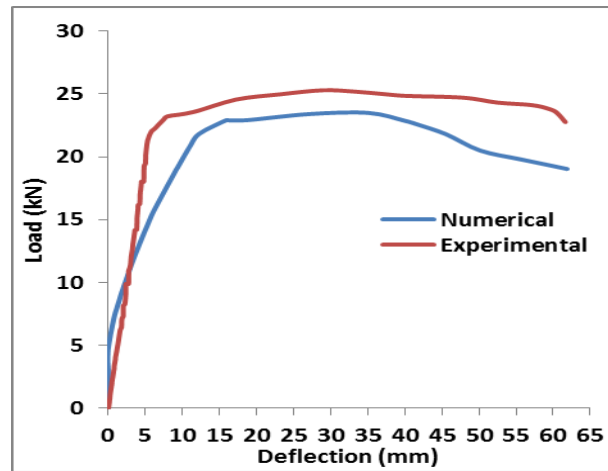

Figure 16. Experiment and Analytical Load Deflection (F2-800).

3.1.2 Strain in Concrete

For the reason of evaluating the load-concrete strain correlations, strain gauges were strategically positioned along the compression side of the concrete beam. The ultimate strain in the concrete, both experimentally and numerically, is shown in **Table 3.** for each of the tested beams. With the highest percentage difference happening for F2-800 (9%), the numerical findings indicate reasonable parallels with the experimental data. **Figs. 17 to 21**. show a comparison between the load-concrete strain curves obtained computationally and those obtained experimentally. The load-strain curves that were obtained from the finite element calculations have been found to match rather well with the experimental data for each and every specimen.

Table 3. Experimental and Numerical Ultimate Strain in the Concrete Unburned and Burned Specimens.

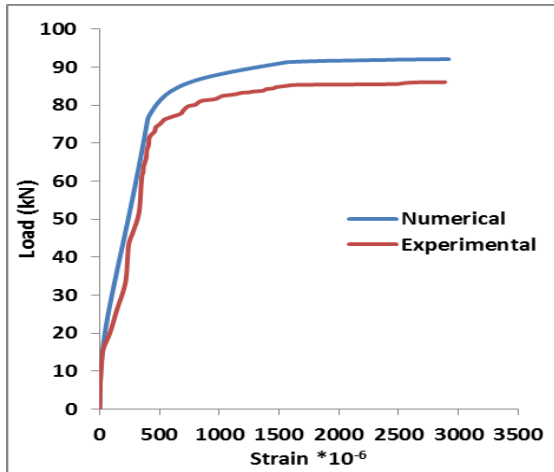| Beam Specimens	Experiment Result Strain $*10^{-6}$	Finite Element Analysis Strain $*10^{-6}$	Ratio $\left\|\dfrac{\text{S Exp.} - \text{S FEA}}{\text{S Exp.}}\right\|$ %
C	2886	2917	1
F1-600	2761	2879	4
F2-600	2587	2731	6
F1-800	1997	2137	7
F2-800	1786	1950	9

Figure 17. Load- Strain Curve for Concrete Surface of control beam.
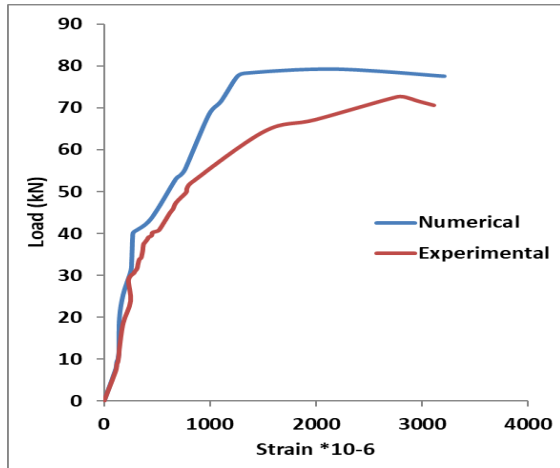

Figure 18. Load-Strain Curve for Concrete Surface specimen F1-600.
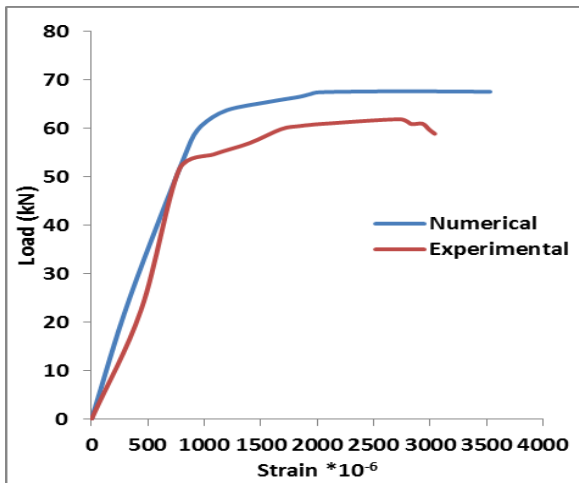

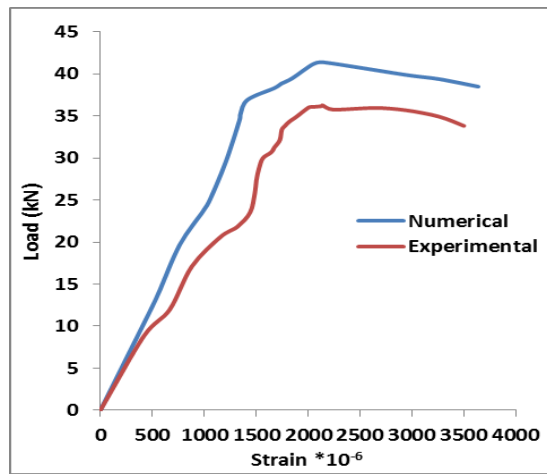**Figure 19**. Load-Strain Curve for Concrete Surface Specimen F2-600.

Figure 20. Load-Strain Curve for Concrete Surface Specimen F1-800.
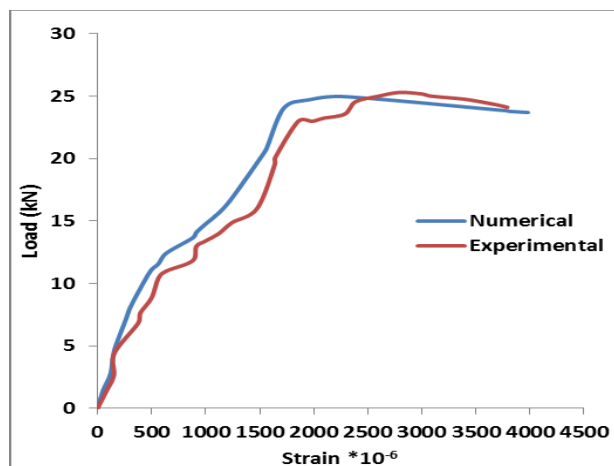

Figure 21. Load-Strain Curve for Concrete Surface specimen F2-800.

3.2 Numerical Parametric Study

Using the Finite Element modeling provided by the ABAQUS software, the effects on the ultimate load and load deflection was investigated. The response of using two separate steady-state burning temperatures (600 and 800) °C as well as burning durations (one and two hours). The following is a case study demonstrating how fire may destroy simply supported concrete beams that have a construction junction. The information was analyzed and compared to that which was acquired from a control specimen that had not been heated by fire. In the case study, the construction joint beams were put through a simulation in which they were exposed to information on the duration of the fire as well as its temperature, which ranged between 600 and 800°C.The allowed burning temperatures of 600 and 800°C, in conjunction with the experimental mechanical characteristics of the control specimens for the concrete mixes (compressive strength and elasticity modules), were taken into consideration, are shown in **Table 4.** These results demonstrate that the elasticity modulus reduction values were more substantial than the compressive modulus reduction values at the identical fire flame temperatures. At 600 °C, the modulus of elasticity held onto between (28.25)% of its initial value. The residual modulus of elasticity at 800 °C was 19.16%. Table 11 displays the findings of the investigation into the impact of fire on compressive strength for exposure times of one and two hours at 600 °C, respectively. Comparing the residual compressive strength with the cube without burning, a value of (63.5, 59.2)% was found. Following 1.0 and 2.0 hours of exposure to fire, the residual compressive strength at 800 °C was found to be (50.9,47)%.

Table 4. Experimental Mechanical Properties for Concrete Mixes.

Specimens	Temp. (℃)	Duration (Hour)	(f cu) (MPa)	Residual of compressive strength %	E (GPa)	Residual of Modulus of Elasticity%
C	25	---	40	---	36	---
F1-600		1	25.4	63.5	10.2	28
F2-600	600	2	23.69	59.2	9	25
F1-800		1	20.39	50.9	6.7	19
F2-800	800	2	18.81	47	5.9	16

4. RESULTS AND DISCUSSIONS

Table 5. presents the findings of the numerical calculations performed on the ultimate load (Pu) and its corresponding deflection (Δu) for the specimen models at the selected burning temperatures.. When burned specimens were compared to those that had not been exposed to fire, it was found that the burned specimens had a lower ultimate load while the fired specimens had a higher peak deflection. **Table 5**. demonstrates that the residual ultimate load was (86 and 73%) with respective exposure times of 1.0 and 2.0 hours at fire temperature (600 °C), whereas the residual at fire temperature (800 °C) was (45 and 26%) with respective exposure times of 1.0 and 2.0 hours. These results are in reference to the temperatures at which the burning occurred at (600 and 800°C). It was shown that the ratio of the maximum mid-span deflection of the beams that burned at 600 and 800 °C to that of the reference beam C resulted in an increase of 4%, 8.8%, 16%, and 24% correspondingly. It

demonstrates that the mid span deflection increased as the burning temperature increased; to put it another words, as the burning temperature climbed, the slope of the load-mid span deflection curve decreased; this indicates that the beam stiffness decreased, indicating that the internal faults increased. **Fig. 22**. demonstrates the test specimens' numerical load-central deflection curves at the two chosen burning temperatures (600 and 800 °C). Due to the specimen's level of fire damage, the behaviour was entirely nonlinear. The trend of the deflection curves was altered during the nonlinear stage when cracks appeared and reduced the stiffness of the specimen.

Table 5. Numerical Ultimate Load and Max Deflection.

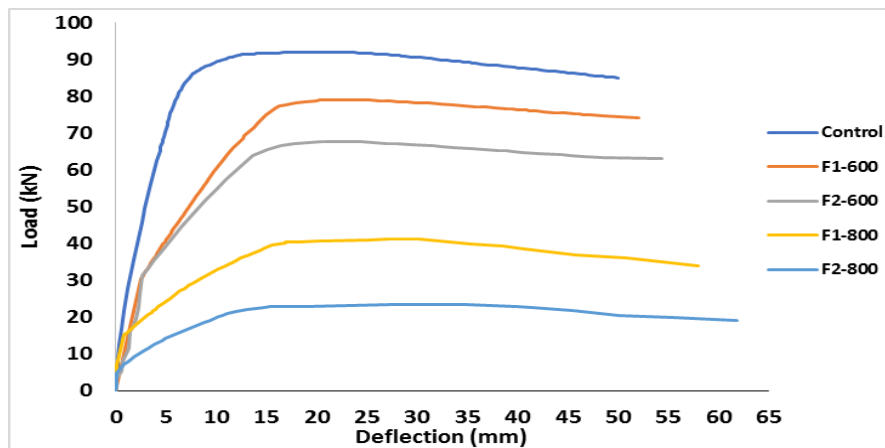|	Specimen Identification	Ultimate Load (kN)	Percentage Residual Ultimate Load %	Max Deflectionat Mid-span (mm)	% increasing In deflection
25	C-25°C	92	100	50	---
600	F1-600°C	79	86	52	4
	F2-600°C	67	73	54.4	8.8
800	F1-800°C	41	45	58	16
	F2-800°C	23.5	26	62	24

Figure 22. Load-deflection curves for beams, FEM analysis.

Calculating the area under the load-flexural deflection curve is the standard method for quantifying flexural toughness, which is often regarded as the ability to soak up energy. A comparison of the flexural toughness of burnt reinforced concrete beams and unburned control beams is shown in **Table 6.** When heated, there is a decrease in the beam's stiffness, mostly due to a fall in the mechanical qualities of the concrete. The absorbed energy was (4220.56, 3575.72, 3243.46, 2080.97 and 1277.56) kN.mm for specimens (C, F1-600, F2-600, and F1-800 and F2-800), respectively. While the percentage variation of the absorbed energy at failure load was (15, 23, 51, and 70) % for specimens (F1-600, F2-600, and F2-800) compared to the control specimen (C).

Table 6. Absorbed Energy for Specimens.

Specimens	Absorbed energy (kN.mm)	%Variation of Absorbed energy
R	4220.56	---
F1-600	3575.72	15
F2-600	3243.46	23
F1-800	2080.97	51
F2-800	1277.56	70

5. CONCLUSIONS

In this investigation, the numerical simulation of a reinforced concrete beams when subjected to fire effect at a construction joint, that's to estimate the behavior of beam after exposed to fire. Based on the results obtained in this investigation, the following can be concluded: -

1. The compressive strength dropped as the fire's heat increased. This explains why, after being exposed to fire for one or two hours at temperatures as high as 600 °C, cracks start to appear. There were 63.5% and 59.2% residual compressive strengths, respectively. In contrast to the unburned cube, the values decreased to 50.9% and 47% after the same amount of time at 800 °C.

2. Following an exposure period of one and two hours to 600°C fire temperatures, the resulting residual flexural strength was 86% and 73%, respectively. In contrast to the control beam, the values decreased to 45% and 26% after the same length of time at 800 °C. This demonstrated how exposure to fire at varying temperatures had an impact on the beam's rigidity.

3. The remaining modulus of elasticity of the concrete was 28% and 25%, respectively, following one and two hours of exposure to fire temperatures of 600 °C. In contrast to the control beam, the values decreased to 19% and 16% after the same amount of time at 800 °C.

4. It is important to appreciate that as the temperature of the fire increases, the deflection of the beam specimens also increases, resulting in a decrease in their load-bearing capacity. This phenomenon occurs because heating causes a reduction in beam stiffness and an increase in deformation.

5. The outcomes showed that the worst effect on the construction joint in the beams occurs at a temperature of 800 °C with the exposure time of 2 hrs. This is attributed to the fact that heating reduces the bonds between two surfaces in the joint and creates a slipping effect, and as a result, a disconnection occurs between the joint surfaces after loading.

NOMENCLATURE

Symbol	Description	Symbol	Description
RC	Reinforcement Concrete	HPC	High-Performance Concrete
WTC	World Trade Center	HCJ	Horizontal Construction Joints
SMPM	Stranded Mesh and Polymer Mortar	WMA	Warm Mix Asphalt
FEA	Finite Element Analysis	Pu	Ultimate Load
FEM	Finite Element Modeling	Δu	Deflection

Credit Authorship Contribution Statement

Ahmed A. Abbood: Writing – original draft. Majid Mohammad Kharnoob: Writing – review & editing, Validation, Conceptualization.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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