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Punching Shear Behavior of Reinforced Concrete Slabs under Fire using Finite Elements

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ABSTRACT

The main aim of this paper is studied the punching shear and behavior of reinforced concrete slabs exposed to fires, the possibility of punching shear failure occurred as a result of the fires and their inability to withstand the loads. Simulation by finite element analysis is made to predict the type of failure, distribution temperature through the thickness of the slabs, deformation and punching strength. Nonlinear finite element transient thermal-structural analysis at fire conditions are analyzed by ANSYS package. The validity of the modeling is performed for the mechanical and thermal properties of materials from earlier works from literature to decrease the uncertainties in data used in the analysis. A parametric study was adopted in this study, it has many factors such as the ratios of length to thickness, fire temperature, time exposed to fire, concrete compressive strength, area exposed to fires and type of support. It can be concluded from this research the significant factors that affect the punching shear strength. However, the increasing ratio of length to thickness may be lead to increasing the deflection more than 123% at fire condition. Also, the increasing temperature leads to increasing the deflection about 40% at fire condition.

Keywords: fires, punching shear, reinforced concrete slab, finite element, and ANSYS.

سلوك القص الثاقب للبلاطات الخرسانية المسلحة تحت النار باستخدام العناصر المحدودة

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الخلاصة

الهدف الرئيسي من هذا البحث هو دراسة القص الثاقب وسلوك ألواح الخرسانة المسلحة المعرضة للحرائق ، وإمكانية حدوث القص الثاقب نتيجة للحرائق وعدم قدرتها على تحمل الاحمال الخدمية. تم إجراء المحاكاة بواسطة تحليل العناصر المحدودة للتنبؤ بنوع الفشل وتوزيع درجة حرارة خلال سمك البلاطة والتشوهات قوة التثقيب. تم تحليل التحليل الحراري-الانشائي

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للعناصر المحدودة غير الخطية في ظروف الحريق بواسطة حزمة ANSYS. يتم التحقق من صلاحية النمذجة بالنسبة لخواص المواد الميكانيكية والحرارية من بيانات الاختبار السابقة لتقليل أوجه عدم اليقين في البيانات المستخدمة في التحليل. تناولت دراسة المتغيرات في هذا البحث وهي نسبة الطول إلى السُمك، درجة الحرارة ، ومدة التعرض للحريق، قوة الانضغاط الخرسانية، المساحة المعرضة للحرائق ونوع التقيد. من الممكن استنتاج من هذا البحث اهم العوامل المؤثرة مقاومة القص الثاقب، حيث ان زيادة النسبة مابين الطول الى السمك تودي الى زيادة الانحراف للبلاطة بحدود 123% في حالة الحريق، كذلك زيادة درجة الحرارة تؤدي الى زيادة الانحراف حوالي 40% في حالة الحريق.

الكلمات الرئيسية: الحرائق, القص الثاقب , البلاطات الخرسانية المسلحة, نمذجة العناصر المحدودة، ANSYS

1. INTRODUCTION

Flat slabs are widely used in buildings, parking garages and shopping centers, it provides a large surface area, has static efficiency allows attaining large span-depth ratios, support on the columns which provided more space, easy to carry out and more economical (**Brzev and Pao, 2006**). Flat slab systems have two mechanisms for the failure: flexural and punching shear mechanisms (**Gosav, et al., 2016**). Flexural failure is preferred to the shear mechanism due to this failure is ductile failure, gives warning signs of failure happen and visual tensile cracks occurring in the member. While the failure of the shear is a brittle failure and do not give visible crack warning signs, where occurs with little displacement **(Nilson, 2010**). Punching shear failure has occurred as cone the crack patterns which started from the tensile face of the flat slab around of the column and continue diagonally to the compression zone of slab **(Brzev and Pao, 2006)**. However, Punching shear failure is a complex phenomenon and has been widely studied experimentally and numerically in the past at ambient temperature as researchers **(Yitzhaki, 1996; Marzuk and Hussein, 1990; Ngo, 2001; Oliveira, et al., 2004; Muttoni, 2008; Jaafer, and Resan, 2015)**. Concrete structures behave very well in the case of fires as an insulating material and had the ability to withstand fire for the time, but sometimes a concrete structure fails or collapses completely when exposed to fire **(Izzat, 2015)**. However, punching shear failure is more sensitive to fire conditions than ambient temperature (**George, 2012)**. The high temperature is caused deterioration in the mechanical material properties of concrete and reinforcement bar, cracks appear, increased deformation of the member, decrease in bearing-capacity. In addition to the redistribution of internal forces because of the difference in temperature between the internal structure and the surrounding structure, causing the appearance of the spalling in the concrete cover and the failure occurs gradually until the structure collapses completely **(Guo and Shi, 2011)**. The studies punching shear of flat slabs in fire conditions are very limited. **(Arna'ot, et al., 2017)**, review six experimental studies of the punching shear of reinforced concrete slabs exposed to fires **(Kordina, 1997; Salem, et al., 2012; Liao, 2013; Annerel et al., 2013; Ghoreishi, et al., 2015; Smith, 2016)**, while **(Bamonte, et al., 2009; Bamonte and Felicetti, 2009)** were numerical analysis studied of punching shear at fire condition. **(Kordina, 1997)** conducted the first test of the punching shear of reinforced concrete slabs subjected to fires. The result showed that the slabs bent down in the opposite direction to the loading direction during the first (30 minutes) of fire. After that the slabs deflected inversely. The factor safety of the punching shear strength with service load was (1.28 to 2.14). **(Salem, et al., 2012**) concluded that the punching shear strength of reinforced concrete slabs reduced about (14%) for slab subjected to fire for duration three hours compared to one hour. Concrete cover has influence on level of reinforcing steel temperature of slabs. The temperature of slabs with thin concrete cover (10mm) increased more than those with (25mm) cover by about (61% to 84%). **(Liao et al., 2013**) showed the fire resistance of the punching shear of twelve reinforced concrete slabs according to ASTM E119 time-temperature curve. The results showed that the punching shear resistance of slabs heated in tension surface occurred in duration about three hours to five hours. The slabs heated in the compressive surface have punching shear resistance up to eight hours. The fire resistance of the normal-strength concrete slabs was higher than the high-strength concrete slabs. The fire resistance of slabs with higher reinforcement ratio lowers than slabs with lower reinforcement ratio. **(Annerel et al., 2013**) showed affecting the ISO-834 standard fire curve. The punching shear failure was clearly in this study as cone shape due to developing the shear cracks through depth of slab. The slabs are bent around the columns, occurs spalling on the corner of the column and on the heated surface at depth up to (100mm), and losses all concrete cover on the heated surface. Shear forces and internal thermal restrained are generated compressive stress which concentrates around of column. **(Ghoreishi et al., 2015**) concluded that the punching shear capacity of thick slabs decrease with increase period exposure to fires and reduced reinforcement ratio. **(Smith, 2016**) showed effect real fires in the punching shear with many parametric: thickness of slabs, steel reinforcement ratio and type of support conditions. The simply supported end thick slabs under constant load failed quickly after heating began, while restrained support slabs failed during heating. The heating duration may be up to two hours. Thinner reinforced slabs failed in flexure and shear mechanisms while thicker slabs failed in punching shear mechanisms. (**Bamonte and Felicetti, 2009**) studied numerical analysis of punching shear of slabs subjected to fire. This studied focus on fire scenario and material properties deterioration**.** This study showed any slab-column compound capable to withstand the standard fire (ISO 834) curve is able to resistance natural fires, because natural fires is not severe as the standard fire. The non-uniform distribution temperature of real fire leads to collapse structures if a reduction in the elastic modules with temperature is taken into consideration. **(Bamonte et al., 2009**) studied theoretical analysis of punching shear of reinforced concrete slabs subjected to damage of fire.

However, a little studies deal with numerical investigation of punching shear of reinforced concrete slabs exposed to fires. Clear from the relevant studies a lot of nonlinear finite elements analysis has been studied for the performance of flat plates exposed to fire under service loads and thermal loads. These studied has tended to focus on deformation of slabs, distribution moment and internal force rather than nonlinear finite element punching shear of slabs at fire conditions. The studied literature review talking about finite element modeling of punching shear of slabs at temperature reaching up to (1000°C), location of fire exposed and length to thickness ratio were not studied. Finite element analysis was used in the present paper utilizing **ANSYS** package version 15, to study affect fire on the punching shear of reinforced concrete slabs.

2. METHODOLOGY

The numerical analysis of reinforced concrete slabs subjected to fire is performed by ANSYS package in multiphysics simulations two steps: transient thermal analysis then transient structural analysis, (**Balaji, et al., 2016; Hawileh and Kodur, 2018**). The temperatures at each node are calculated from thermal analysis used as input data in the structural analysis (**ANSYS R.15, 2013; Balaji, et al., 2016**). SOLID70 and LINK33 elements are used to model concrete and steel reinforcement bar in the thermal analyses, respectively. Brick SOLID70 and LINK33 elements can use in steady state and transient thermal analysis. These elements have the ability to conduct thermal load throughout the concrete model (**ANSYS R.15, 2013**). The structural analysis is performed by exchanged from thermal analysis to structural analysis. Then switch SOLID70 and LINK33 elements to SOLID65 and LINK180 elements, respectively (**Balaji, et al., 2016**; **Hawileh and Kodur, 2018).** Brick SOLID 65 is element used to modeling concrete or reinforced concrete. The brick element has the ability to crush in the compression and cracking in the tensile, the dealing of linear and nonlinear material properties and susceptive of plastic deformation and creep. LINK180 element is three dimensions used to model the reinforcement bar for structural analysis. In addition to using the SOLID185 element to model the steel plate of support. The properties of SOLID185 element are very similar to SOLID 65 element properties,but SOLID185 is not capable of simulating concrete cracking and crushing (**ANSYS R.15, 2013**). The finite element analysis including thermal and structural analysis can be summarized in **Fig.1**.

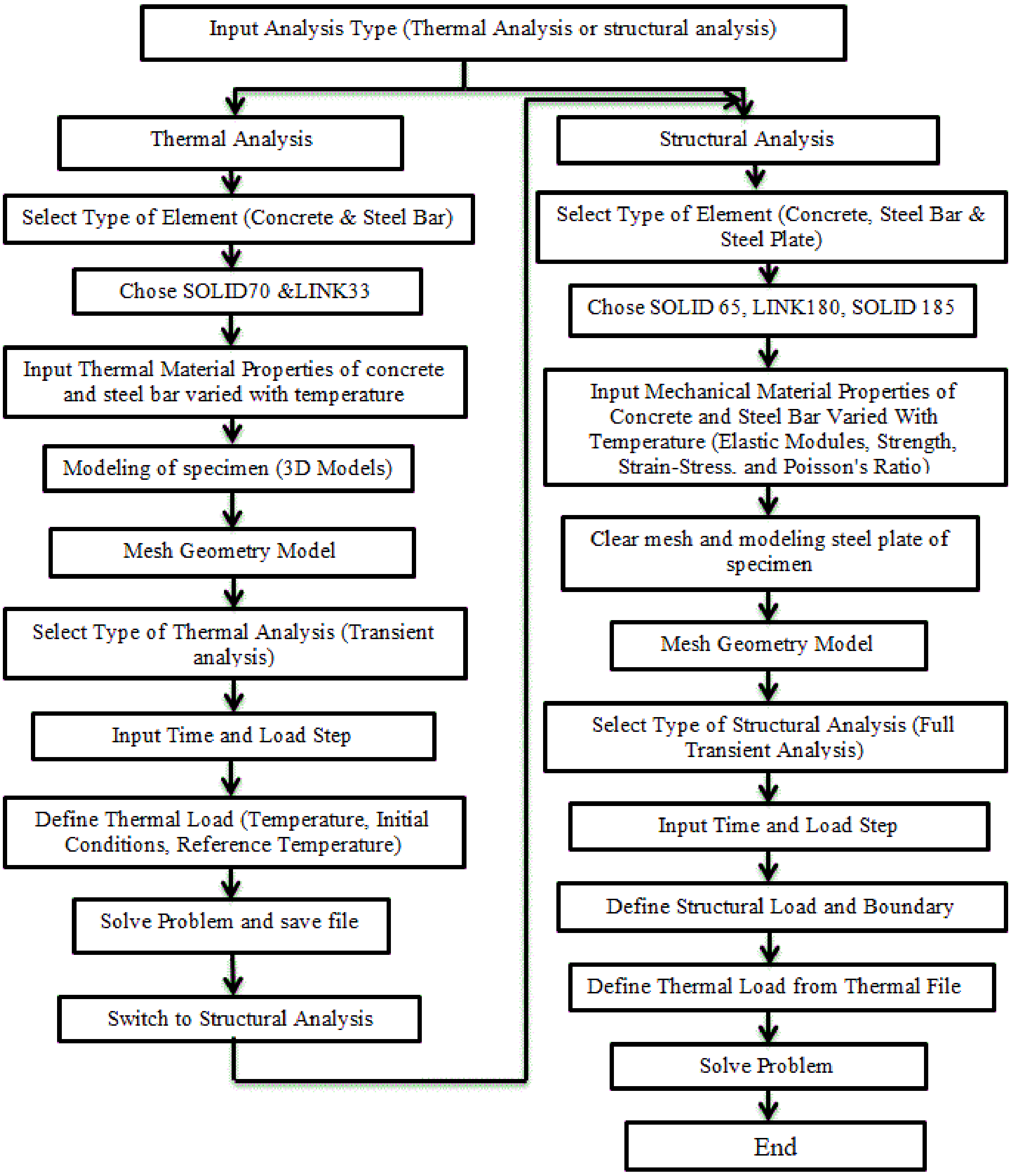

Figure 1. Finite element analysis procedure.

3. MATERIAL PROPERTIES OF NUMERICAL ANALYSIS

Reinforced concrete structure material properties do vary with temperature so the analysis is nonlinear (**Balaji, et al., 2016**). The thermal material properties of concrete and reinforcement bar, and their changed with temperature are adopted and calculated in this paper based on the **(EN 1992-1-2, 2004** and **EN 1993-1-2, 2005)** as shown in **APPENDIX A**. In the structural analysis is needed to define mechanical material properties of concrete and reinforcement bar and their variation with increasing temperature. Mechanical material properties of concrete play an important role in predicting the ultimate strength of the structure at high temperatures.

Concrete slabs exposure to fires leads to a serious deterioration in mechanical properties, **(Arna'ot, et al., 2017)**. The **(EN 1992-1-2, 2004**) gives limitation to reduction compressive strength by factor reduction as function of temperature and uniaxial stress-strain relationship, as shown in **APPENDIX A**. On the other hand, the tensile strength of concrete at ambient temperature was calculated corresponding to (**ACI Committee 318, 2011)**. At fire conditions, the tensile strength of concrete calculated according **(EN 1992-1-2, 2004)**. The elastic modulus of concrete for each specimen was calculated according to **(ACI Committee 318, 2011)**. Poisson’s ratio for concrete was assumed to be 0.2 for all analysis at each temperature (**Hawileh and Kodur, 2018)**. The structural analysis need to shear transfer coefficients. The shear transfer coefficients range from (0 to 1) and are of two types: shear transfer coefficients for open cracks (smooth cracks) which are close to (0) and a complete loss of shear transfer (βo). The second is the shear transfer coefficients for the closed cracks (βc) which are close to the value of the (1), where no loss occurs in the shear transfer **(ANSYS R.15, 2013**). Elastic modules of steel is taken equal (200GPa) at ambient temperature. At fire condition, the elastic modules and yield strength of steel taken from **(EN 1992-1-2, 2004)**. Poisson's ratio of steel bar is taken equal to (0.3). The properties of materials and numerical analysis adopted in this paper were from the Master's thesis of (**Gharbi, 2018).**

4. FAILURE CRITERIA AND NUMERICAL CONVERGENCE

The failure criteria of slabs exposed to fire by reaching strength or thermal failure limit states **(Balaji, et al., 2016)**. The thermal failure of reinforced concrete slabs is possibly occurred when the average temperature of slabs on the unexposed surface override 250ºF (139°C) above its initial temperature. The temperature of tension reinforced steel exceeds (593°C) **(ASTM, 2000)**. The strength failure occurred when the structure was unable to resist the applied loads. A deflection criterion is important limit states to show the behavior of slabs at ambient and fire conditions. The deflection criterion for determining slab failure in ambient conditions was also applied in determining the realistic failure of the slabs subjected to fire, when the maximum deflection of the slab exceeds to (L/20) (**BS 476, 1987)**. The deflection of slabs in fire conditions is expected more than deflection at ambient temperature due to deterioration in material properties of reinforced concrete.

The Newton-Raphson technique to obtain numerical convergence in the finite element each time increment is used. When temperature variation at each node exceeds (0.05)°C the divergence of the solution in thermal analysis was occur. The divergence in structural analysis can happened when force convergence more the tolerance limit of (0.01) (**ANSYS R.15, 2013)**.

5. VALIDATION MODEL

5.1 Description of the Developed Numerical Models

To investigate the behavior and punching shear strength of reinforced concrete slabs under structural loads and thermal load at fire conditions by ANSYS package has been implemented. The validity of the modeling is performed for the mechanical and thermal properties of materials from previous test data to minimize the uncertainties in data utilized in analysis. Eight validation models for numerical elements were adopted in this study using the ANSYS package. The models were performance from references **(Liao, et al., 2013; Smith, 2016; Lim and Wade, 2002; Wang, et al., 2018)**. The details of cross section and properties of these slabs are shown in **Table.1**.

Table 1. Tabulate the name of validation models and the characteristics of it.

Reference	Slab Code	Slab Size (m)	Column Size(mm)	Concrete cover (mm)	f'c MPa	fy MPa	ρ (%)	ρ_{max} (%)	Diameter and spacing (mm)	Fire Scenarios	Fire Duration (h)
Liao, et al., 2013	TS-R1-C1-N	1.8*1.8 * 0.12	180*180	20	32	420	0.92	2.32	Ø12@140mm	ASTM E119	4.75
	TS-R1-C2-N				58	420	0.92	3.27	Ø12@140mm		4
	TS-R2-C2-N				58	420	1.43	3.27	Ø12@80mm		3.75
Smith, 2016	AU75-0.8	1.4*1.4 * 0.075	120*120	15	51	550	0.8	2.36	Ø6@65mm	Ambient temperature tested	
	HU75-0.8									Real fire	2
Lim and Wade, 2002	HD12 flat slab	4.3*3.3 * 0.1	-	25	36.7	468	0.565	2.21	Ø12@200mm	ISO 834	3
Wang, et al., 2018	Slab R1	3.9*3.3 * 0.1	-	20	27.2	485	0.31	1.72	Ø8@200mm	ISO834	4

f'c : concrete compressive strength; fy: yield strength of steel and ρ : steel reinforcement ratio

5.2 Results and discussions Validation Model

The models were validated by comparing predicted parameters response of temperatures distribution, failure times and deflections from the finite element analysis by ANSYS package with fire test data. Finite element models are validated by comparing the simulated models with experimental studied data of the various researches illustrated in **Table.2**. The results showed that were achieved good agreements in punching shear strength, mid-span deflection, temperature and time exposed as shown in **Fig.2** – **Fig.13**. **Fig. 2** exhibits a comparison between simulated model of (TS-R2-C2-N) and experimental data at different depths of the slab. In the numerical analysis, the (TS-R2-C2-N) was divided into elements with thickness (20 mm) because that gives better convergence during analysis. So the comparing temperature-time curve of the simulated model was at a depth (20 mm) with the experimental specimen at depth (30 mm). The steel reinforcement bar temperature of slab (595.35ºC) at depth (20mm) is slightly more than ASTM E119 critical temperature limit of reinforced steel (593ºC) as show in **Fig 2.** Thus, the temperature failure criteria govern the failure of the tested slab specimen.

In (HU75-0.8) specimen was appeared the effect of increasing temperature to (550ºC) at ambient temperature to the increasing the deflection of slab by (46%) as shown in **Fig.6**. The results shown that was some different in middle time-temperature curve of (HU75-0.8) model as shown in **Fig.7** and **Fig.8**, that due to the uniform distribution of the temperature on exposed surface to fire in the finite element analysis. While was used real fire in the experimental test, which was non-uniform distribution of the temperature on exposed surface to fire. Failure criteria limit in the steel reinforcement temperature of **(ASTM E119, 2000**) was more than steel reinforcement temperature of slab by (148ºC) as shown in **Fig.7** at depth (11mm). The temperature of the unexposed surface slab was more than failure criteria limit of unexposed surface by (44.6ºC) as shown in **Fig.8**. So the failure criteria for the (HU75-0.8) specimen are governed by the temperature calibration of unexposed surface. **Fig.9** and **Fig.10** showed experimental and numerical time-temperature curve of (HD12 flat slab) and (Slab R1) models. The steel reinforcement temperature of (HD12 flat slab) was more than failure criteria limit in the steel reinforcement bar temperature of (**ASTM E119, 2000**) by (109.2 ºC). The failure criteria limit in the steel reinforcement bar temperature of (Slab R1) more by (58.36ºC). **Fig.12** to **Fig.17** show the variation deflection of validation models analysis.

Table 2. The details of cross section and configuration of various slabs.

Reference	Slab Code	Deflection of Experimental (mm)	Deflection of FEA (mm)	Different ratio%
(Liao, et al., 2013)	TS-R1-C1-N	29.45	29.84	1.32
	TS-R1-C2-N	21.63	21.63	0
	TS-R2-C2-N	23.83	23.8	0
(Smith, 2016)	AU75-0.8	31.8	32.296	1.56
	HU75-0.8	43.55	44.0974	1.256
(Lim and Wade, 2002)	HD12 flat slab	119.848	125.194	4.46
(Wang et al., 2018)	Slab R1	68.235	68.91	2.454

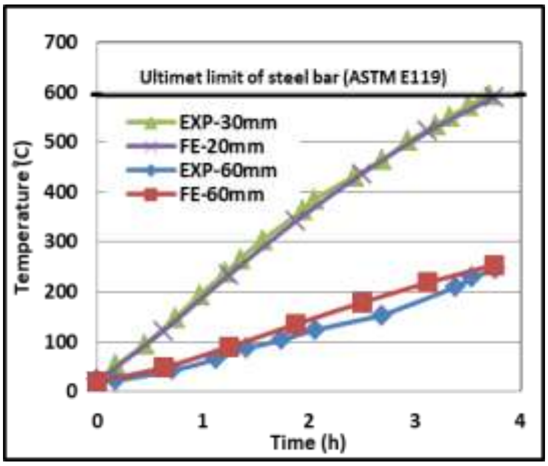

Figure 2. Experimental and numerical time-temperature curve of (TS-R2-C2-N) model.

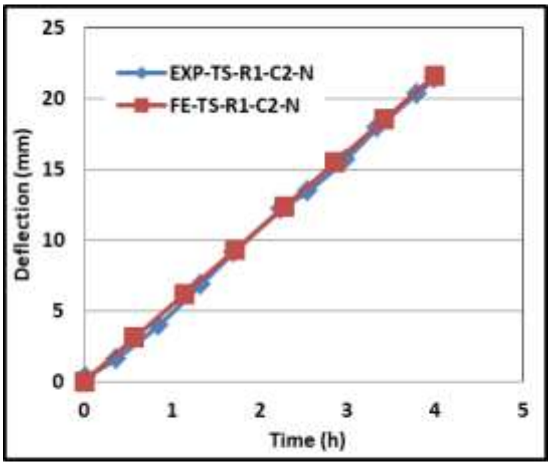

Figure 3. Experimental and numerical time-deflection curve of (TS-R1-C2-N) model.

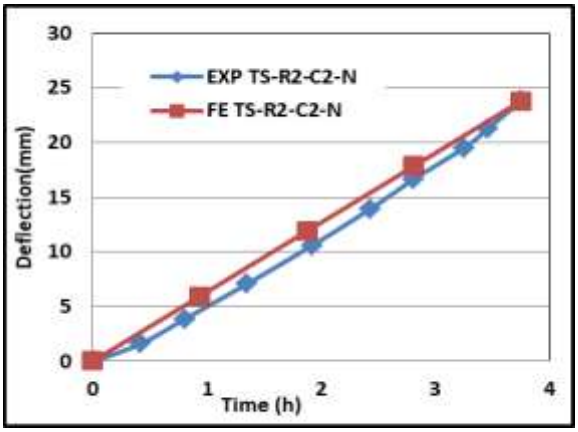

Figure 4. Experimental and numerical time-deflection curve of (TS-R2-C2 N) model.

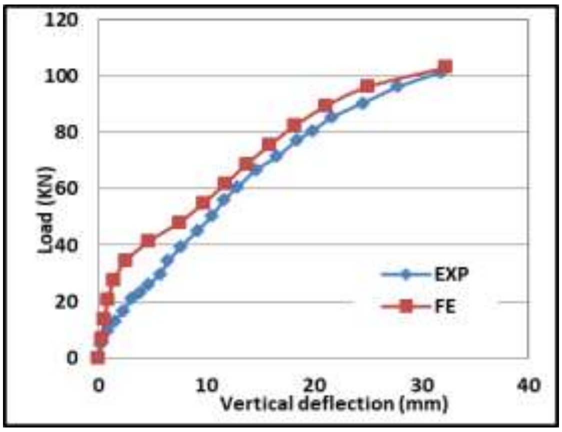

Figure 5. Experimental and numerical load-deflection curve for (AU75-0.8) model.

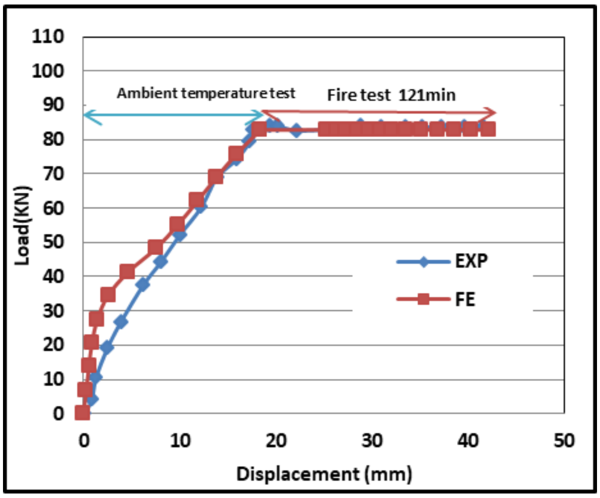

Figure 6. Experimental and numerical load-deflection curve for (HU75-0.8) model.

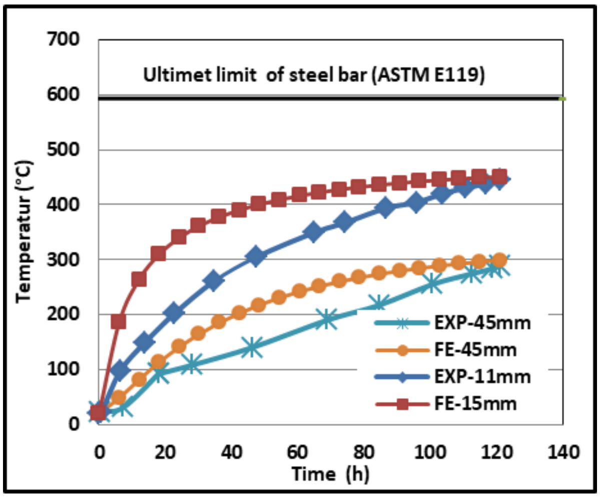

Figure 7. Experimental and numerical time-temperature curve of (HU75-0.8) model.

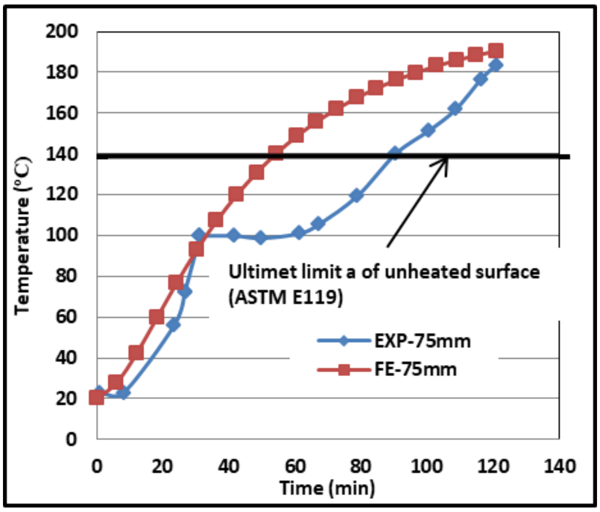

Figure 8. Experimental and numerical temperature-time curve of (HU75-0.8) at unheated surface.

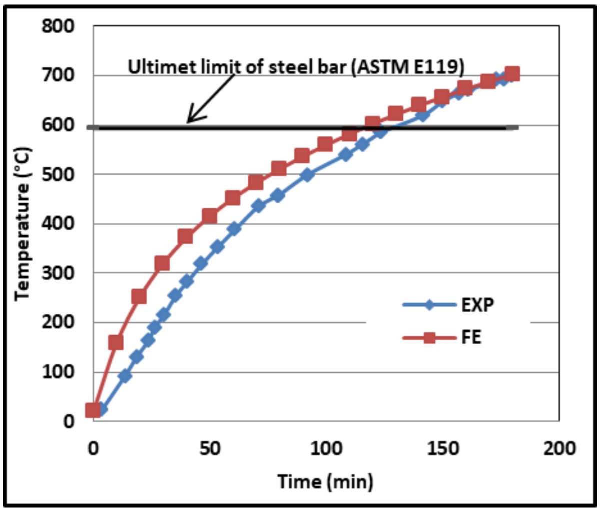

Figure 9. Experimental and numerical temperature–time curve of (HD12 flat slab) model at (25mm) depth.

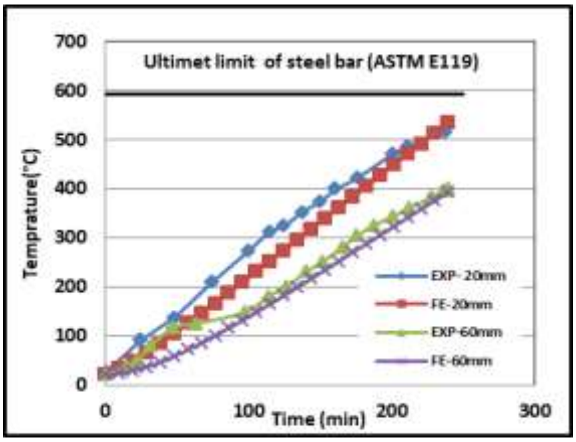

Figure 10. Experimental and numerical time-temperature curve of (Slab R1) model at fire resistance (4h) and different depths.

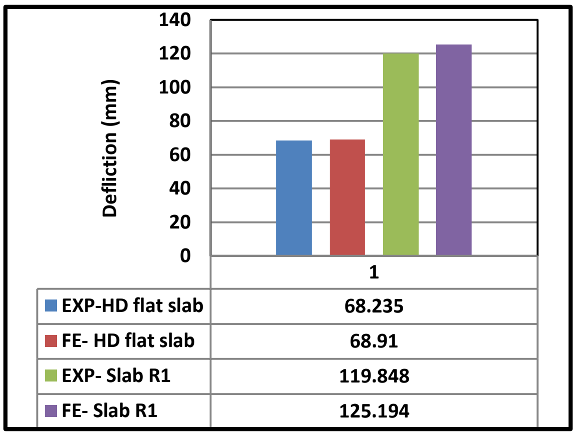

Figure 11. Experimental and numerical maximum deflection value of (HD12 flat slab) and ((Slab R1) models.

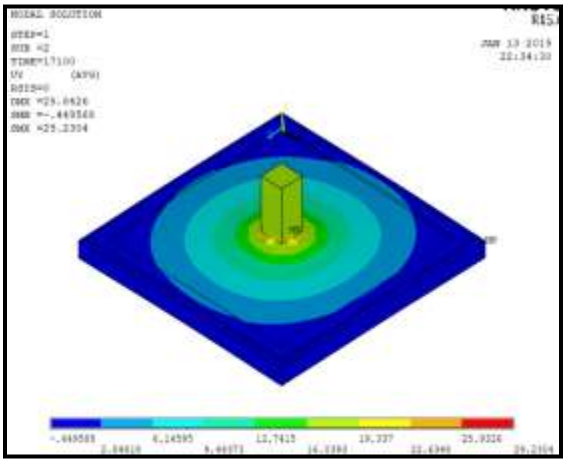

Figure 12. The variation deflection in the (TS-R1-C1-N) model.

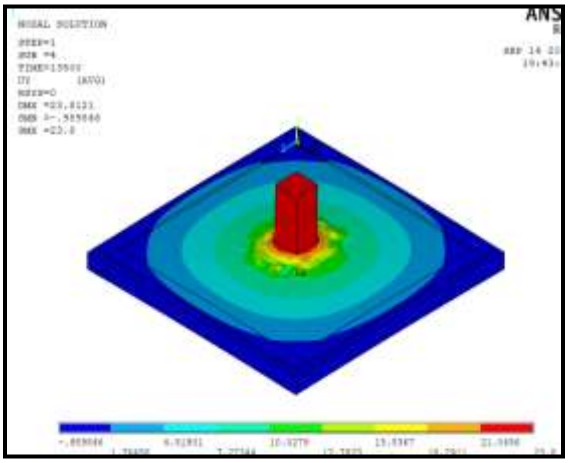

Figure 13. The variation deflection in the (TS-R2-C2-N) model.

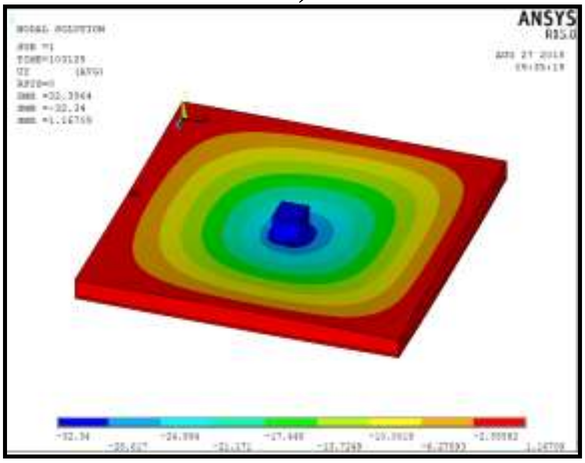

Figure 14. The variation deflection in the (AU75-0.8) model.

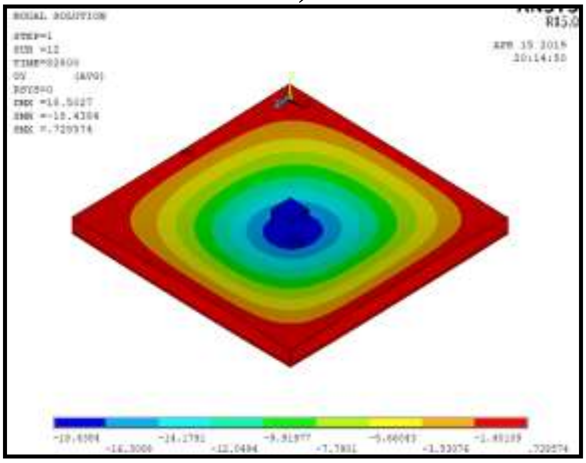

Figure 15. The variation deflection in the (HU75-0.8) model of ambient temperature test.

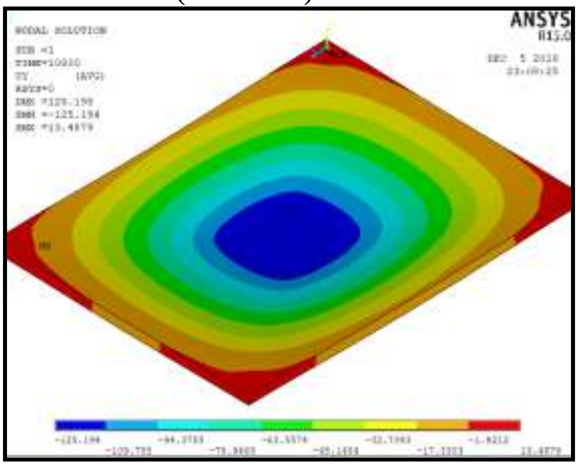

Figure 16. The variation deflection in the (HD12-flat slab) model.

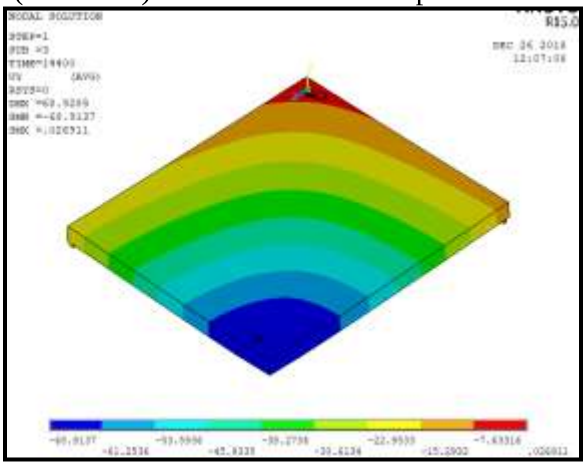

Figure 17. The variation deflection in the (Slab R1) model.

6. NUMERICAL MODEL

A numerical model is studied punching shear and behavior of reinforced concrete slabs subjected to fires using finite element analysis (ANSYSY package). The aim of the designing model is simulated the behavior of punching shear of the reinforced concrete slabs at fire conditions with some parameter. The parameters studied in this paper are part of parameters studied in the thesis's **(Gharbi, 2018)**. The main parameters were studied the effect of ratio of length to thickness (L/h), temperature (T), time exposed to fires(t), compressive strength of concrete (f'c), area exposed to fires (A) and varied support conditions parameter.

6.1. Characteristics of Slab Model

The slabs modeled have square cross section (2000*2000) mm and thickness varied according to length to thickness ratios. The slab model is connected with centroid column dimensions (200*200*100) mm. The flexure steel reinforcement that used in these specimen were (Ø8@200mm), positioned in two orthogonal directions, as shown in **Fig.18**. The yield strength of steel bar was (485MPa), elastic modules was (200GPa) and concrete cover (20mm).

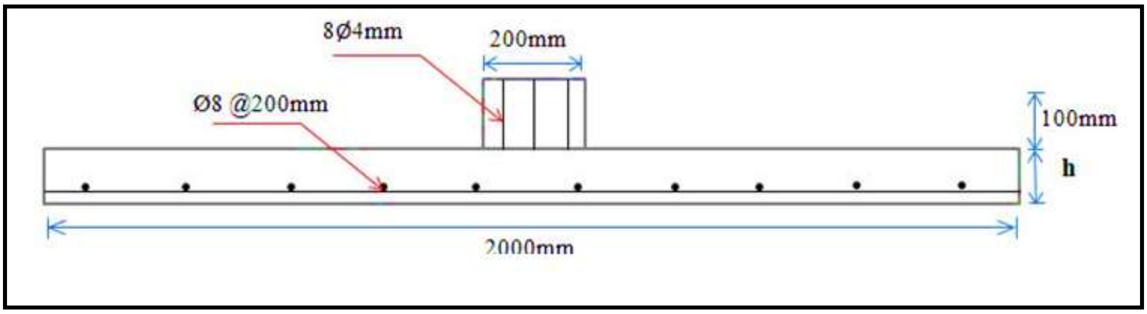

Figure 18. Details of specimens in parametric study.

In the finite element analysis use brick element to model concrete and line element to model steel reinforcement bar as discussion in this paper in section (2). The dimensions of element were chosen according to the volume of element that gives good results in the verified study of finite element models. The dimensions of elements were (25*25*20) mm for slabs models with thickness (100mm). The dimensions of the slabs element are modeled with thickness (130mm) with two sizes. The first volumes were (25*25*20) mm and second volumes were (25*25*22)mm as shown in **Fig.19**. These differences in element dimensions are to provide a constant concrete cover for all models. The dimensions of the element were similar in thermal and structural analysis. Four steel plates along the four edges of the slab with a width of (50 mm) and a thickness of (20 mm) were used to support the slab, these plates to prevent local failure at the support location. In addition to the use of steel plate dimensions (200* 200*20) mm to prevent local failure in the load location applied to the column. Thermal material properties that give good agreement were taken from results of verification models. The mechanical material properties of concrete utilized in thermal-structural analysis are compressive strength, elastic modulus and tensile strength as discussion in section (3). The coefficient of the open shear crack and close shear crack which represent to damage in the concrete are taken as (0.25) and (0.8) respectively. The mechanical material properties of steel bar utilized in structural and thermal-structural analysis are yield strength, elastic modulus and tangent model. The yield strength and elastic modulus reduction factored of steel reinforced are adopted form the (**EN 1992-1-2, 2004**) as shown in **APPENDIX A**. The models are dependent nonlinear thermal and mechanical material properties of concrete and reinforcement steel which was varying with temperature. Full bonding between concrete and steel is performed to prevent bond failure.

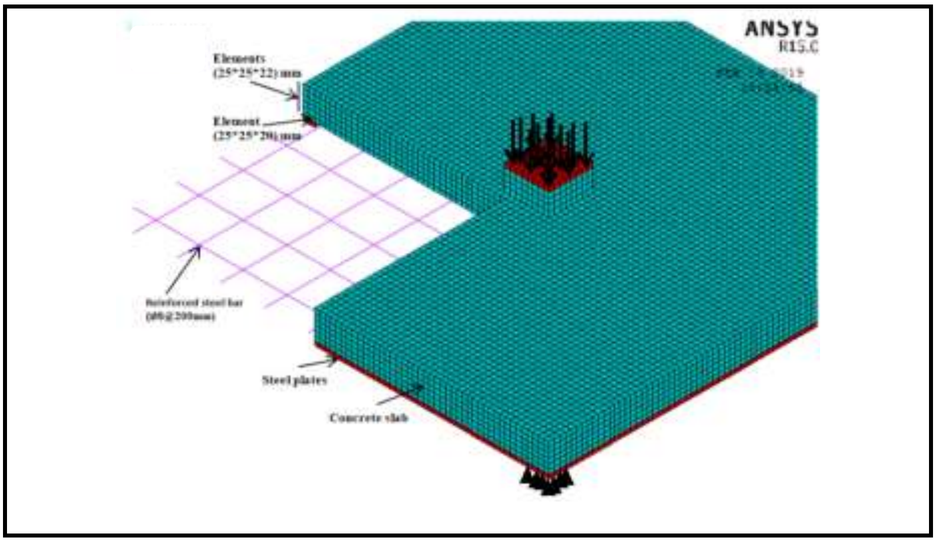

Figure 19. Detail of slab model with thickness (130mm).

6.2 Loading and Boundary Conditions

Thermal loading was applied as temperature distribution on the nodal point of bottom surface of the slabs. This temperature was applied on area (1800*1800) mm to represent the space occupied by the support is not exposed to fires. The value of temperature is varied according to the parametric study selected with time exposed to fires. The initial condition of temperature and reference temperature were (20°C) define of all models. Thermal analysis was transient ramped loading. In the structural analysis, the slab was loaded by structural load (100kN) distribution on (81nodes) on the steel plate of column and simply supported on the four corners of steel plates of slab as shown in **Fig.19**. The restrained was (Δx, Δy, Δz) on four nodes on one side, in the opposite direction was (Δy, Δz) restrained on four nodes, while other sides were restrained by four nodes in (Δy).

7. INFLUENCE OF VARIOUS PARAMETERS

This section studied various parameters on the models as length to thickness ratio (15, 20, 25 &30), temperature exposed (20, 600, 800 & 1000)°C, time exposed to fire (2, 3& 4) hours, compressive strength of concrete (30, 45 &60)MPa, area exposed to fire (A, 3/4A, 1/2A,1/4A &NA) and type of support condition (simply support & fixed support).

7.1. Effect of the Length to Thickness Ratio

The effect (L/h) ratio was studied in this paper at ambient temperature (20°C) and fire condition (600°C). The thickness of slabs were varied according to ratio of length to thickness (15, 20, 25 &30) confront thickness (130, 100, 80 & 60) mm, respectively. The time exposed to fires was two hours with compressive strength of concrete (30MPa). The **Fig.20** and **Fig.21** were observed that increasing of (L/h) ratio lead to increasing the deflection at same load, temperature, time and compressive strength. The increase (L/h) ratio from (15 to 20), (20 to 25) and (25 to 30) the deflection were increased by (2.23, 1.98 and 2.74) times respectively, at fire conditions. This means decrease thickness of slab from (130mm) to (60mm) lead to increasing the mid slabs deflection by (12.13) times as shown in **Fig.21**. The increasing deflection of slab with time was due to decreasing in thickness. The central deflection of the simply supported slab at ambient temperature is inversely proportional to the thickness cube of the slab (**Godzwon, 1960)**. The ratio of length to thickness affects the temperature of the unexposed surface as shown in **Fig.22**. The unexposed surface temperature increases with the increasing the ratio and that to reduce the thickness of the specimen and increasing the transfer of heat during the depth of slab. **Fig.22** illustrated increase unexposed surface temperature by (57.2%), (35.88%) and (34.5%) for increasing ratio of (L/h) from (15 to 20), (20 to 25) and (25 to 30), respectively. **Fig.23** shown there is no effect of the (L/h) ratio on the temperature of reinforcing steel because the value of concrete cover was equal for all specimens.

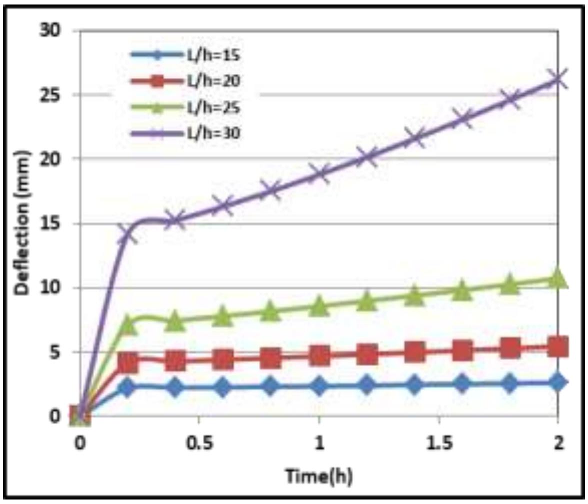

Figure 20. Time vs. deflection behavior of different slabs thickness at ambient temperature (20ºC).

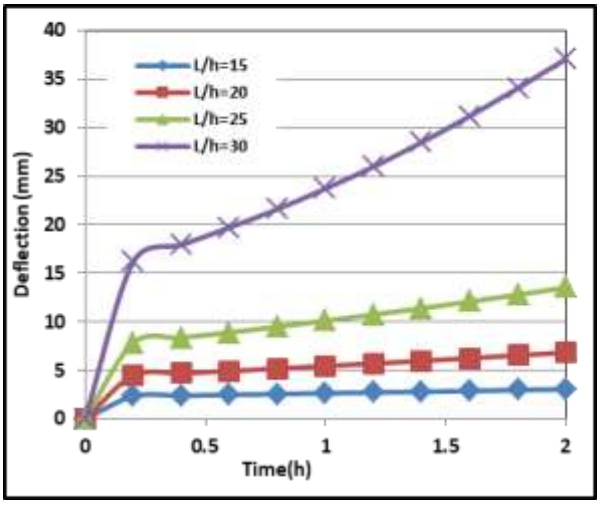

Figure 21. Time vs. deflection behavior of different slabs thickness at fire conditions (600ºc).

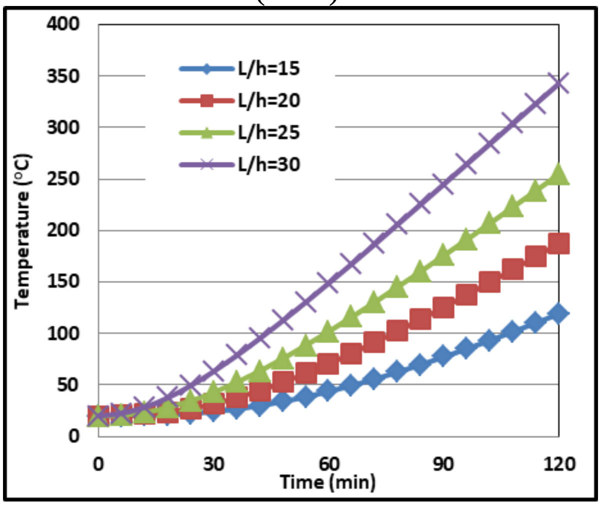

Figure 22. Time vs. temperature behavior of unexposed surface of different slabs thickness.

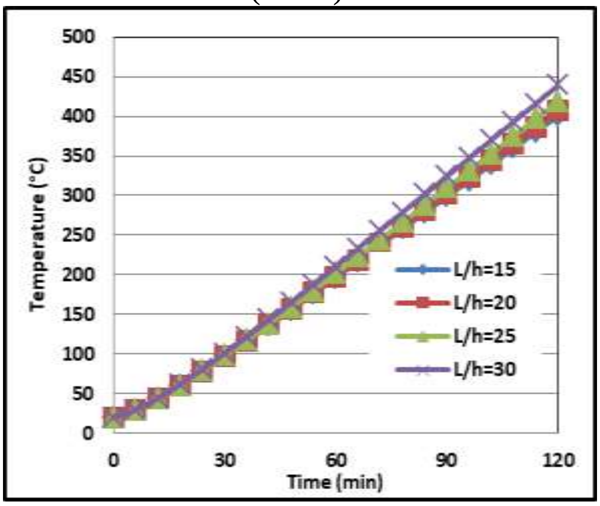

Figure 23. Time vs. temperature behavior of steel reinforced of different slabs thickness.

7.2. Effect of the Temperature Exposed to Surface

Four stages in temperature (20, 600, 800 and 1000)˚C are taken in this study to evaluate the concrete behavior under different degrees of temperature with the(30MPa) compressive strength, (2 hours) time exposed to fires and (L/h=20). **Fig.24** illustrated the increasing temperature for the specimen lead to increasing the deflection. This increase on the deflection was due to deterioration on material properties of reinforced concrete slabs. The mid deflection of slab was increase by (25.43%), (40.89%) and (65.65%) when temperature increasing from (20°C) to (600°C, 800°C and 1000°C) respectively. Temperature distribution on the exposed surface slabs to fire was offered in **Fig.25**. **Fig.26** showed that the reinforcing bar temperature increasing by (26% and 49.6%) when the temperature increasing from (600 to 800)° C and (600 to 1000)° C, respectively. **Fig.27** illustrated that increasing the unexposed surface temperature with increasing temperature on exposed surface. The unexposed surface temperature increase by (19.85%) and (36.35%) when temperature increasing from (600 to 800)°C and (600 to 1000)°C respectively. The risk of high temperature of the unexposed surface is the possibility of shear off in the column due to thermal expansion on the exposed surface to fire. The thermal load caused deterioration in the structure of the concrete, cracks and reduced load-capacity (**fib 38, 2007)**.

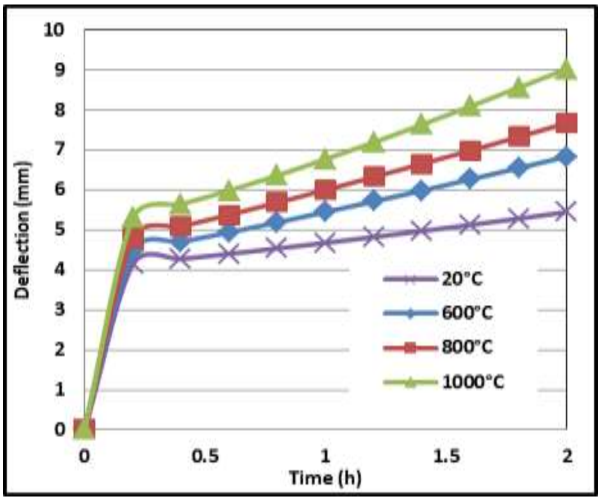

Figure 24. Time vs. deflection behavior of slabs with different temperature.

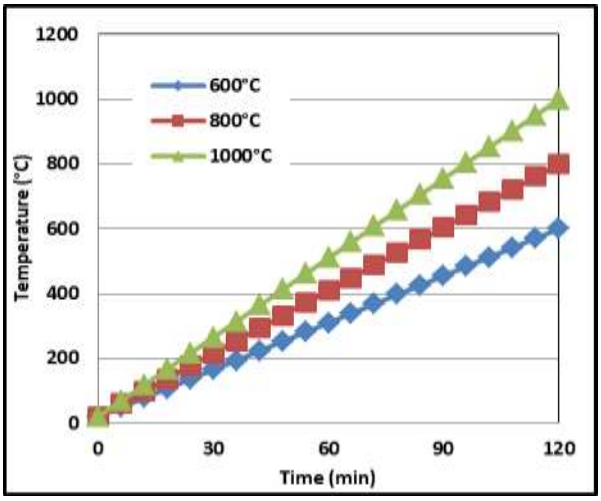

Figure 25. Time vs. temperature behavior of exposed surface of slabs with different temperature.

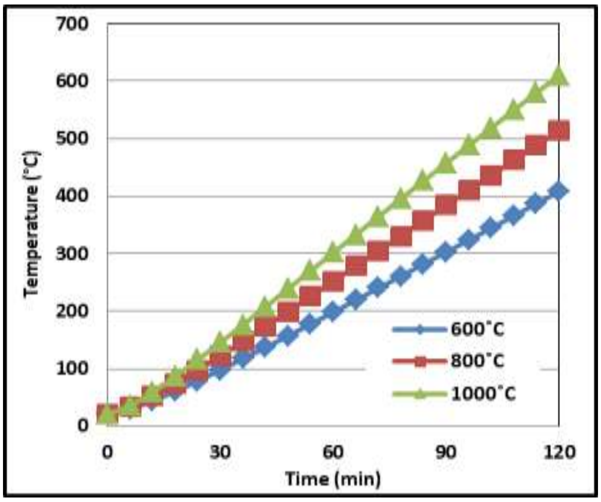

Figure 26. Time vs. temperature behavior of steel reinforced of slabs with different temperature.

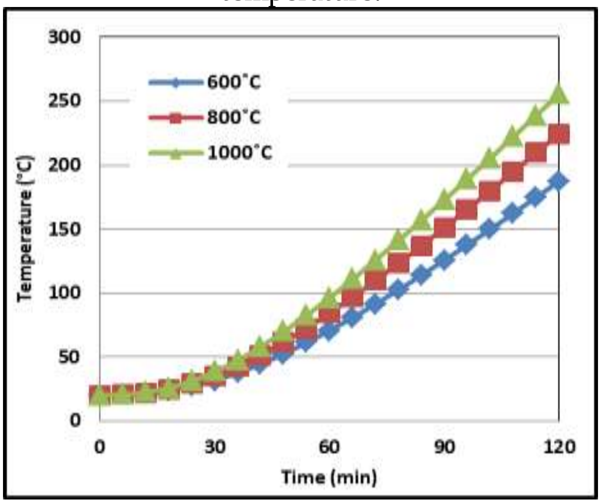

Figure 27. Time vs. temperature behavior of unexposed surface with different temperature.

7.3 Effect Time Exposed Slabs to Fires

The numerical analysis of effect time exposed of slabs to fires (2h, 3h and 4h) were performed on slabs have (600°C) temperature, (L/h=20) and (30MPa) concrete compressive strength. **Fig.28** showed that slightly effect on increasing the mid deflection of the slab by (5.74%) at increasing time exposed to fire by (50%). **Fig.29** showed the effect of increasing time exposed to fire on exposed surface of slabs. In **Fig.30** steel reinforced bar temperature increases with a very small amount with increasing the exposure time of the fire. The increasing temperature by (8.9%) and (14.57%) with increasing the time exposed to fire (50%) and (100%), respectively.

Fig.31 observed that increasing the exposure time of the model of the fire by (50%) and (100%) increases the unexposed surface temperature to fire about (31.5%) and (54.66%), respectively.

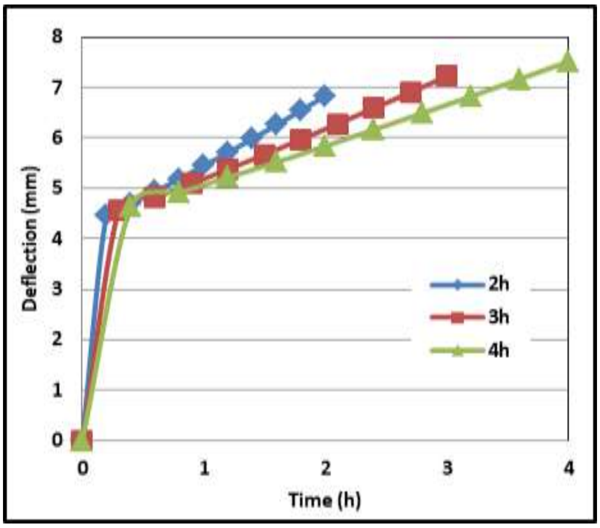

Figure 28. Time vs. deflection behavior of slabs with different time exposed.

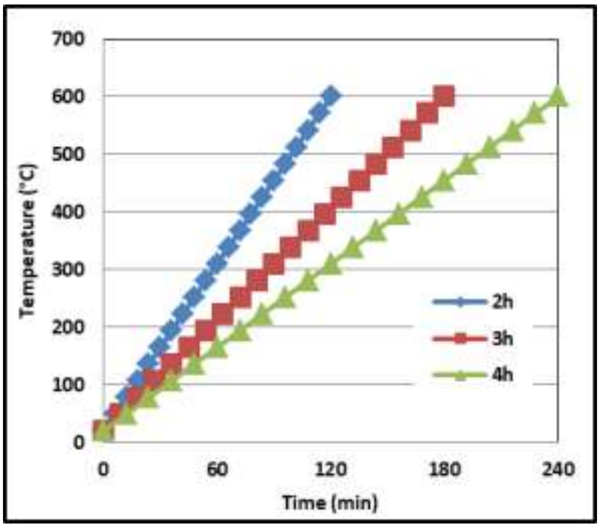

Figure 29. Time vs. temperature behavior of exposed surface with different time exposed.

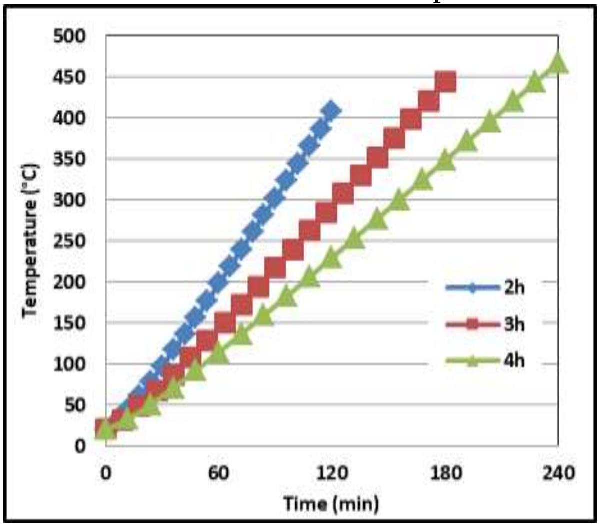

Figure 30. Time vs. temperature behavior of steel reinforced of slabs with different time exposed.

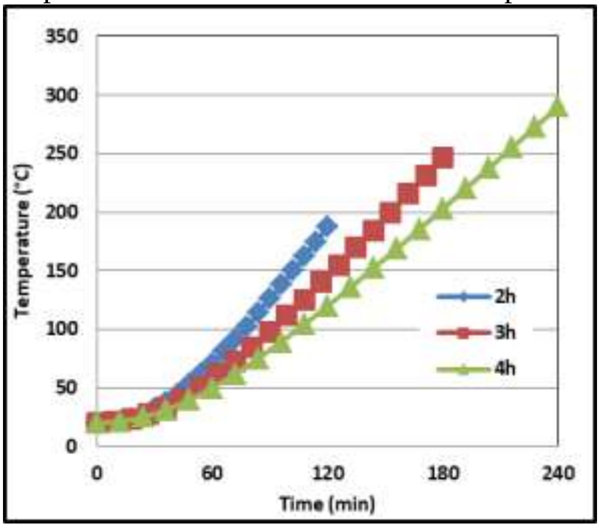

Figure 31. Time vs. temperature behavior of unexposed surface of slabs with different time exposed.

7.4 Effect of Concrete Compressive Strength

The compressive strength of concrete is the most important mechanical properties. It is main parameter to define the strength grade and goodness of the concrete, it is important to determine the value of elastic modulus, tensile strength, and strain-stress relationship. Similarly at high temperature, the compressive strength, elastic modulus, tensile strength and the stress–strain relationship, it is basis for studying the behavior of concrete structures. To study the effect of the compressive strength of the behavior of reinforced concrete slabs exposed to fire, three values of the concrete compressive strength (30, 45 and 60)MPa were selected. The models analysis have length to thickness ratio (20), exposed to temperature (20 & 600)°C for two hours. **Fig.32** and **Fig.33**, it was observed that increases of compressive strength by (50%) and (100%) lead to decrease the deflection in mid slab by (13.94%) and (30.46%), respectively at ambient temperature and by (17.155%) and (28.62%) respectively, at fire conditions. **(Chan et al., 1996)** were showed effecting temperature on compressive strength of concrete in experimental works. That increase temperature lead to gradually loss in mechanical strength at normal and high compressive strength of the concrete. The decrease in compressive strength of concrete was (15%) for normal strength concrete (NSC) and (10%) for high strength concrete (HSC) at increase temperature from (20 to 400)°C. At increase temperature from (400-800)°C the decrease in compressive strength of concrete be severe. This decrease in compressive strength of concrete due to hydration of harden cement paste (hydration of C-S-H gel and loses its cementing ability). At temperature above (800°C) was considered damaged at both normal strength concrete (NSC) and high strength concrete (HSC).

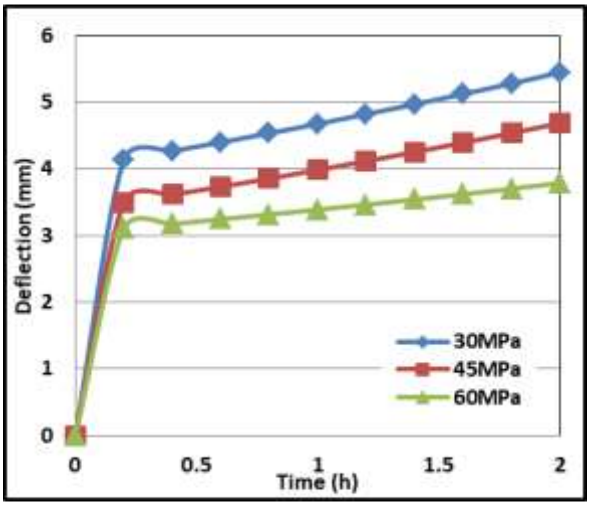

Figure 32. Time vs. deflection behavior of slabs with different compressive strength at ambient temperature (20°C).

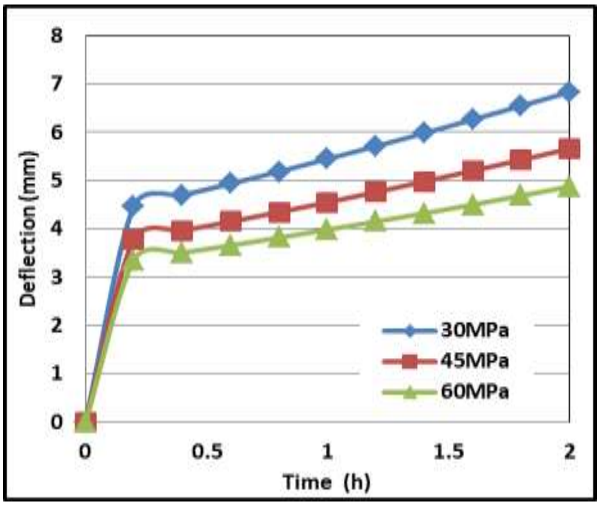

Figure 33. Time vs. deflection behavior of slabs with different compressive strength at fire conditions (600°C).

7.5 Effect of Area Exposed to Fires

To study the effect of the area exposed to fires of the behavior and punching shear of reinforced concrete slabs was analyzed. The models were selected to study the effect of the temperature on parts of the surfaces, and to study the effect of the type of support on the model have length to thickness ratio (20), exposed to (600°C) temperature for two hours. The distribution temperature was on all bottom surfaces (A), three-quarter bottom surface (3/4A), half bottom surface (1/2A) and quarter bottom surface (1/4A). In addition to analysis model ambient temperature to represent to area not exposed to temperature (NA). In **Fig.34** and **Fig.35**, it was observed that decrease area exposed to fire lead to decrease the deflection in mid slab. Whereas decrease area exposed from total exposed (A) to three quarter (3/4A) leads to decrease the deflection by (5.724%). Also decreasing area exposed from (A to 1/2A) and (A to 1/4A) leads to decrease the deflection by (10.362%) and (15.456%), respectively. At comparing between all surface exposed to area (A) and model analysis at ambient temperature, the deflection was decrease by (20.27%). These results illustrate that the effect of increasing thermal load on the exposed surface of the fire. Where analysis shows that the risk of exposure of all the surface of the slab to the fire is higher than the case of exposed parts of the slab surface to the fire. The behavior of the part slabs was exposed to fires better than the behavior of the all slabs exposed to fires. Generally one of the failure criteria is reaches the members to principal stress, or principal strain or principal shear-stain. So in this section principal strain can use to know or define failure criteria of sample instead of deflection, which is the maximum deflection on the surface exposed to fire on each surface. The enveloped principal strain of slabs was (0.001) of simply supported, and (0.000659) of fixed supported at slabs exposed to fire on half area, as shown in **Fig. 36** and **Fig37**. The principal strain is selected in mid slab in same location maximum point deflection for simply supported and fixed supported. This variation in principal strain is return to different temperature in surface area. Thermal load exposed on each surface area increased the principal strain by (102.5%) at ambient temperature. And that the difference in temperature on half of the surface and internal parts of the slab led to increased principal strain by (203.31%) at ambient temperature for simply supported. For fixed supported, Thermal load exposed on each surface area increased the principal strain by (17.85%) at ambient temperature, and that the difference in temperature on half of the surface and internal parts of the slab led to increased principal strain by (62.17%) at ambient temperature. From these results conclude that the strain due to thermal load, the distribution of different temperature on the exposed surface and the different temperatures of the inner layer act in the opposite direction of the strain resulting from fixed supported.

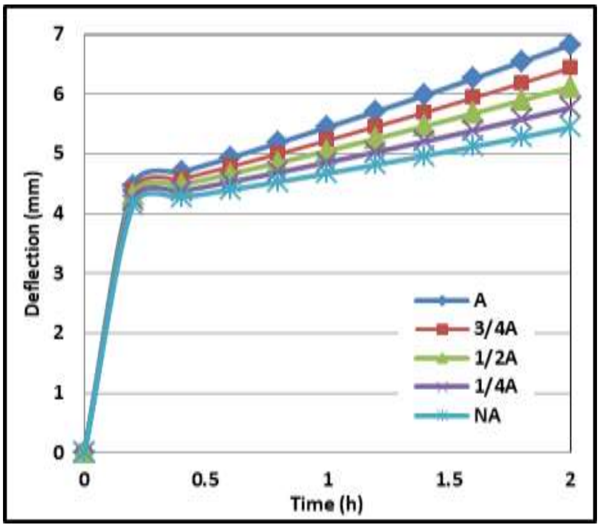

Figure 34. Time vs. deflection behavior of slabs with different area exposed and simply support.

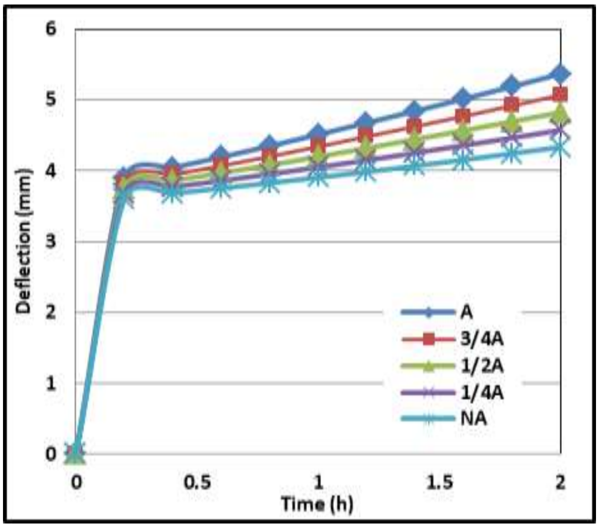

Figure 35. Time vs. deflection behavior of slabs with different area exposed and fixed support.

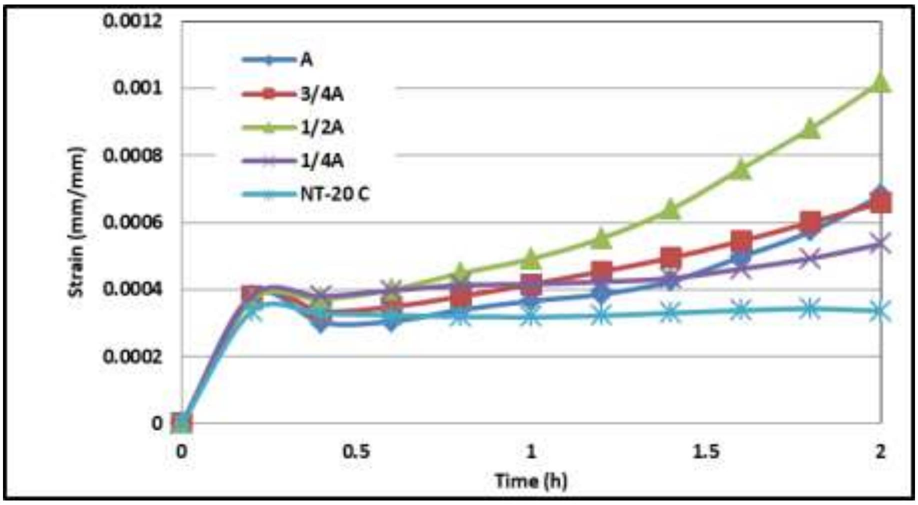

Figure 36. Time vs. strain behavior of slabs with different area exposed and simply support.

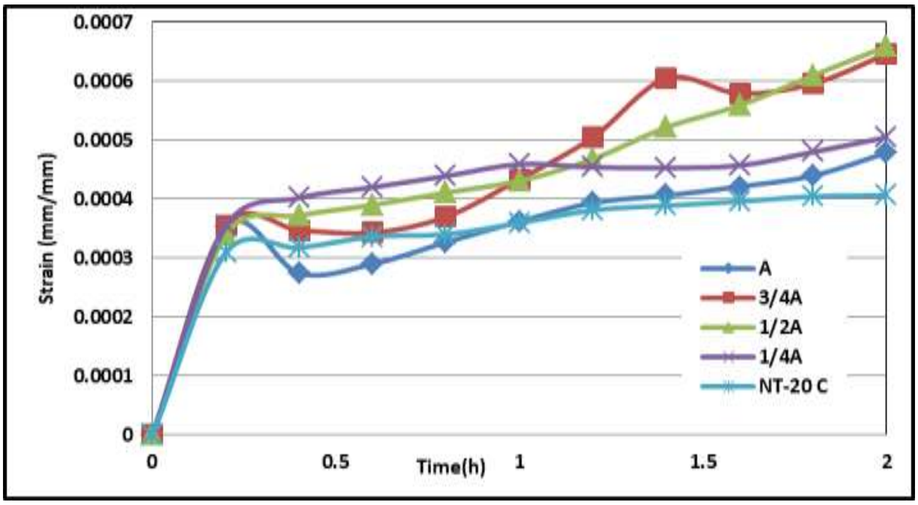

Figure 37. Time vs. strain behavior of slabs with different area exposed and fixed support.

7.6 Effect of Varied Support Conditions

For studies effect various support conditions on the punching shear of reinforced concrete two-way slab exposed to fire, two type of support (simply and fixed support) were selected. Most building is simply and fixed support so was studied effect these types of support. The fixed supported on the corner of slabs have beneficial effect of decreasing the deflections in the middle of the slab at ambient temperature and subjected to fire. In addition was causing restrained thermal expansion that leading to in-plane forces. The deflection of slabs was decreases by (21.37%) at use fixed supported as shown in **Fig.38.** However at ambient temperature, the end fixed supported of the member is reducing the deflection in the member because it generates in plane stress that led to increase strain.

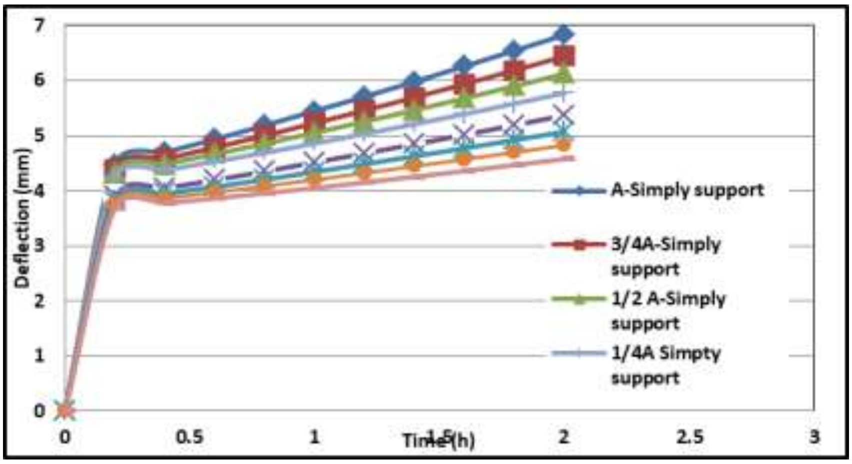

Figure 38. Time vs. deflection behavior of (S20-600-2h-30) model and different support.

8. SUMMARY AND CONCLUSION

The analysis presented in this paper studied the punching shear of reinforced concrete slabs subjected to fire. This study showed effect some parameter on punching shear of slabs exposed to fire. The parameters were studied the effect area exposed to fire, type of support condition, length to thickness ratio, the temperature exposed to the surface of the slab, concrete compressive strength and time exposed to fire. The following conclusions can be offer from the results of this studied:

- The modeling by finite element analysis is capable of predicting punching shear of reinforced concrete slabs subjected to fires, temperature distribution through slab thickness, deflections and fire resistance of reinforced concrete slabs subjected to fire.
- The ratio of the length to slabs thickness has a clear effect on the punching shear of reinforced concrete slabs exposed to fire. Increasing the ratio of length to thickness by (33.33%) increases the deflection of the slabs by (108.218%) and (123.256%) in ambient temperature and fire conditions, respectively. This increase is due to the inverse proportion of the slab thickness cube with the central deflection of the simply supported slab.
- The increasing temperature for the specimen leads to increase deflection. The mid deflection of slab was increase by (25.43%), (40.89%) and (65.65%) when temperature increase from (20°C) to (600°C, 800°C and 1000°C), respectively. This increasing on the deflection was due to deterioration on material properties of reinforced concrete of slabs. In addition to the concrete becomes unsafe when it reaches a temperature higher than (600°C).
- The increases of compressive strength of concrete lead to decrease the deflection in slab. Increasing compressive strength by (50%) and (100%) leads to decrease deflection by (13.94%) and (30.46%) respectively at ambient temperature and by (17.155%) and (28.62%) respectively, at fire conditions.
- The risk of exposure of all the surface of the slab to the fire is higher than the case of exposed parts of the slab surface to the fire. The behavior of the part slabs was exposed to fires better than the behavior of the all slabs exposed to fires.
- The fixed supported on the corner of slabs have beneficial effect of decreasing the deflections in the middle of the slab at ambient temperature and subjected to fire. In addition was causing restrained thermal expansion that leading to in-plane forces. The deflection of slabs was decreases by (21.37%) at use fixed supported than simply supported.

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APPENDIX A: THERMAL AND MECHANICAL MATERIAL PROPERTIES USED IN ANALYSIS

Thermal material properties	Concrete (EN 1992-1-2, 2004)	Steel reinforced bar (EN1993-1-2, 2005)
Conductivity (W/m.°C)	$\lambda_c = 2 - 0.2451 * \left(\frac{T_c}{100}\right) + 0.0107 * \left(\frac{T_c}{100}\right)^2$ (20°C ≤ T_c ≤1200 °C)	$\lambda_s = 54 - 33.3 * 10^{-3} * T_s$ (20°C ≤ T_s ≤ 800°C) $\lambda_s = 27.3$ (T_s ≥ 800°C)
Specific heat (J/kg. °C)	$c_{p\,(T_c)} = 900$ (20°C ≤ T_c ≤ 100°C) $c_{p\,(T_c)} = 900 + (T_c - 100)$ (100°C < T_c ≤ 200°C) $c_{p\,(T_c)} = 1000 + (T_c - 200)/2$ (200°C < T_c ≤ 400°C) $c_{p\,(T_c)} = 1100$ (400°C < T_c ≤ 1200°C)	$c_s = 425 + 0.77 * T_s - 1.69 * 10^{-3} * (T_s)^2 + 2.22 * 10^{-6} * (T_s)^3$ (20°C ≤ T_s ≤ 600°C) $c_s = 666 + \frac{13002}{738 - T_s}$ (600°C ≤ T_s ≤ 735°C) $c_s = 545 + \frac{17820}{T_s - 731}$ (735°C ≤ T_s ≤ 900°C) $c_s = 650$ (900°C ≤ T_s ≤ 1200°C)
Density (kg/m^3)	$\rho_c = \rho_{(20°C)}$ (20°C ≤ T_c ≤ 115°C) $\rho_c = \rho_{(20°C)} * (1 - 0.02 * \frac{T_c - 115}{85})$ (115°C < T_c ≤ 200°C) $\rho_c = \rho_{(20°C)} * (0.98 - 0.03 * \frac{T_c - 200}{200})$ (200°C < T_c ≤ 400°C) $\rho_c = \rho_{(20°C)} * (0.95 - 0.07 * \frac{T_c - 400}{800})$ (400°C < T_c ≤ 1200°C)	$\rho_s = 7850$ (20°C ≤ T_c ≤1200 °C)
Mechanical material properties	*Concrete* (EN 1992-1-2, 2004)	Steel reinforced bar (EN 1992-1-2, 2004)
Elastic modulus (MPa)	$E_c = 4730\sqrt{f'_c}$ (20°C ≤ T_c ≤1200 °C) **(ACI Committee 318, 2011)**	$E_{s,T} = 1$ Es (20°C ≤ T_s ≤ 100 °C) $E_{s,T} = 0.9$ Es (T_s=200 °C) $E_{s,T} = 0.8$ Es (T_s=300 °C) $E_{s,T} = 0.7$ Es (T_s=400 °C) $E_{s,T} = 0.6$ Es (T_s=500 °C) $E_{s,T} = 0.31$ Es (T_s=600 °C) $E_{s,T} = 0.13$ Es (T_s=700 °C) $E_{s,T} = 0.09$ Es (T_s=800 °C) $E_{s,T} = 0.07$ Es (T_s=900 °C) $E_{s,T} = 0.04$ Es (T_s=1000 °C) $E_{s,T} = 0.02$ Es (T_s=1100 °C)
Tensile strength of concrete	$f_t = 0.62\sqrt{f'_c}$ (T=20°C) **(ACI Committee 318, 2011)** $f_{t,T} = \left(1 - 1 * \frac{T - 100}{500}\right) f_t$ (100 °C ≤ T ≤ 600°C) **EN 1992-1-2, 2004**	
Uniaxial stress-strain relationship	$\sigma(T) = [3 * \varepsilon * f_{c,T}]/[\varepsilon_{c1,T}\ (2 + (\frac{\varepsilon}{\varepsilon_{c1,T}})\wedge 3)]$	
Reduction factor of concrete compressive strength and the yield strength of steel	$f'_{c,T}/f'_c = 1$ (20°C ≤ T_s ≤ 100 °C) $f'_{c,T}/f'_c = 0.95$ (T_s=200 °C) $f'_{c,T}/f'_c = 0.85$ (T_s=300 °C) $f'_{c,T}/f'_c = 0.75$ (T_s=400 °C) $f'_{c,T}/f'_c = 0.6$ (T_s=500 °C)	$f_{y,T}/f_y = 1$ (20°C ≤ T_s ≤ 400 °C) $f_{y,T}/f_y = 0.78$ (T_s=500 °C) $f_{y,T}/f_y = 0.47$ (T_s=600 °C) $f_{y,T}/f_y = 0.230$ (T_s=700 °C) $f_{y,T}/f_y = 0.110$ (T_s=800 °C)

	$f'c_{,T} / f'c = 0.45$ (T_s=600 °C) $f'c_{,T} / f'c = 0.30$ (T_s=700 °C) $f'c_{,T} / f'c = 0.15$ (T_s=800 °C) $f'c_{,T} / f'c = 0.08$ (T_s=900 °C) $f'c_{,T} / f'c = 0.04$ (T_s=1000 °C) $Es_{,T} / Es = 0.01$ (T_s=1100 °C)	$fy_{,T} / fy = 0.06$ (T_s=900 °C) $fy_{,T} / fy = 0.04$ (T_s=1000 °C) $fy_{,T} / fy = 0.02$ (T_s=1100 °C) $fy_{,T} / fy = 0$ (T_s=1200 °C)