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Behavior of Composite Steel Deck-Concrete Slabs Subjected to Elevated Temperature

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Abstract

The present study is concerned with behavior of composite steel deck-concrete slabs under the effect of high temperature. Twenty-eight slabs were prepared for this matter and divided into eight groups. The parameters included in this study are, slab depth, additional reinforcement (reinforcing bars, welded wire mesh, transverse spot-welded wires), and temperature level.

The specimens were tested as simply supported slabs with two-point load. The ultimate load capacity, mid-span deflection, slip at ends of slab, failure mechanism and crack pattern are studied.

The ultimate load capacity for composite slabs decreases significantly when subjected to high temperature. The decrease in the ultimate load capacity becomes less when additional reinforcement is used in composite slabs.

الخلاصــــــة

تتعلق الدراسة الحالية بسلوك البلاطات المركبة من صفائح الحديد و الخرسانة تحت تأثير الحرارة العالية. ثمان و عشرون بلاطة صبت لهذا الغرض مقسمة إلى ثمان مجاميع. المتغيرات المتضمنة في هذه الدراسة هي: سمك البلاطة، التسليح الإضافي (التسليح الطولي، مشبك الانكماش و الحرارة، التسليح العرضي بواسطة الأسلاك ذات اللحام النقطي)، ودرجة الحرارة.

تم فحص النماذج كبلاطات بسيطة الإسناد و بخطي حمل، وتمت دراسة الحمل الأقصى، الانحراف في وسط النموذج، الانزلاق عند حافتي السقف و أيضاً تمت دراسة طبيعة الفشل و نموذج التشقق.

مقدار الحمل الأقصى للبلاطات المركبة يقل عند تعرضها للحرارة العالية وان هذا النقصان يقل باستخدام التسليح الإضافي في السقوف المركبة.

1. Introduction

One of the major dangers confronting buildings is the exposure to fire. Nowadays, and in spite of civilization progress in the world in many fields of life especially in the field of building and construction and in spite of the high technology in the design of buildings and the full control of the progress, fire (caused by wars or uncontrolled conditions or simply because of errors that would happen) endangers people's life and makes the building unsafe. It is necessary that the building should be provided with sufficient structural fire resistance to give occupants time to escape before strength and, or stability failure ensure.

Both concrete and steel reinforcement are not able to burn, but if they are exposed to fire the behavior of both will not be acceptable (negative behavior, i.e. physical and geometric properties) and the degree of acceptability depends on the degree of temperature during the fire, the period of the fire and the distribution of the temperature in the internal section of the concrete and the steel [1].

In composite construction, the degree of fire protection that must be provided is one of the factors that influence the choice between concrete, composite, and steel structure, and here concrete has an advantage [2].

Composite construction has been widely used for building structures over the past (50 years). Initially, developed for beams and girders in building and bridges, composite elements now also include columns and shear walls, and are frequently utilized in high-rise structures, owing to their high axial load capacity and stiffness. The last few years have seen the development of the complete composite frame, where the advantages of steel and concrete are combined to provide structural systems of great strength and stiffness [3].

The composite slabs formed using by profiled sheeting as a permanent formwork and as a tensile reinforcement to a concrete slab, have now become a common form of construction of floor decks in steel framed buildings. Nowadays approximately (40%) of all new multi-storey buildings, **Fig.(1)**, in the UK use composite floor construction [4]. This type of construction is structurally efficient because it exploits the tensile resistance of the steel and the compressive resistance of the concrete.

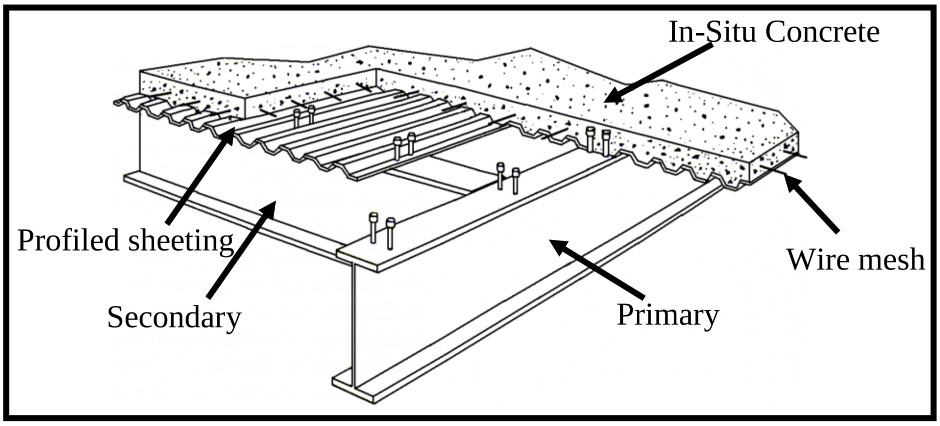

Figure (1) Composite slab [4]

The decking performs a number of roles and is an important part of structural system; some of the benefits are [4]:

1. Profiled sheeting acts as stay-in place formwork.
2. It offers an immediate working platform.
3. It acts as slab reinforcement.
4. It saves up till (30%) concrete material.
5. It makes easy transportation and installation.
6. It accommodates service ducts.

2. Experimental Investigation

2-1 Test Slabs

Twenty-eight slab specimens are divided into eight groups, two groups consist of five slabs for each one and the other remaining groups consist of three slabs for each. The specimens of the two groups were heated to four levels of temperature of (150, 300, 600 and 800^{0}C), and the other groups were heated to two stages of temperature of (300 and 600^{0}C) with exposure time was (90 minutes), and a slab was tested at room temperature (30^{o}C) for each series to compare the result of the heated specimens with the unheated ones. **Table (1)** describes the eight groups of slabs included in this study.

The test slab specimens are divided into eight groups, these groups are described below:

Group (CSA): It is cast without any additional reinforcement, and has slab depth of (80 mm).

Group (CSMA): It is cast with welded wire mesh of ϕ5 mm placed at the middle of concrete thickness, and has slab depth of (80 mm).

Group (CSTA): It is cast with transverse spot-welded wires of ϕ6 mm welded @ 140 mm on the top of steel deck, and has slab depth of (80 mm).

Group (SCLA): It is cast with additional longitudinal tensile reinforcement by using bars of 10 mm in diameter and has slab depth of (80 mm).

Group (CSB): It is cast without additional reinforcement, and has slab depth of (110 mm).

Group (CSMB): It is cast with welded wire mesh of ϕ5 mm placed at the middle of concrete thickness, and has slab depth of (110mm).

Group (CSWB): It is cast with welded wire mesh of ϕ5 mm welded on the top of steel deck, and has slab depth of (110 mm).

Group (CSLB): It is cast with additional longitudinal tensile reinforcement by using steel bars of ϕ(10 mm), and has slab depth of (110 mm).

Figure (2) shows the details of test specimens.

Table (1) Slab specimens properties

Group No.	Type	Slab No.	Length (mm)	Width (mm)	Depth (mm)	f_{cu} (MPa)	Additional reinforcement	Temp. stage (°C)
1	CSA	S_1	700	300	80	31.55	Without	Room temp.
		S_2				29.65		150
		S_3				32.22		300
		S_4				34.78		600
		S_5				32.44		800
2	CSMA	S_6	700	300	80	33.87	Welded wire mesh ϕ5 mm placed at middle of concrete thickness	Room temp.
		S_7				30.15		300
		S_8				33.42		600
3	CSTA	S_9	700	300	80	29.88	Spot welded wires ϕ6 mm @ 140 mm along steel deck	Room temp.
		S_{10}				30.32		300
		S_{11}				31.33		600
4	CSLA	S_{12}	700	300	80	33.82	Tensile reinforcement by using steel bars ϕ10 mm	Room temp.
		S_{13}				32.91		300
		S_{14}				33.23		600
5	CSB	S_{15}	700	300	110	31.55	Without	Room temp.
		S_{16}				29.65		150
		S_{17}				32.22		300
		S_{18}				34.78		600
		S_{19}				32.44		800
6	CSMB	S_{20}	700	300	110	33.87	Welded wire mesh ϕ5 mm placed at middle of concrete thickness	Room temp.
		S_{21}				30.15		300
		S_{22}				33.42		600
7	CSWB	S_{23}	700	300	110	29.88	Welded wire mesh welded on top of steel deck	Room temp.
		S_{24}				30.32		300
		S_{25}				31.33		600
8	CSLB	S_{26}	700	300	110	33.82	Tensile reinforcement by using steel bars ϕ10 mm	Room temp.
		S_{27}				32.91		300
		S_{28}				33.23		600

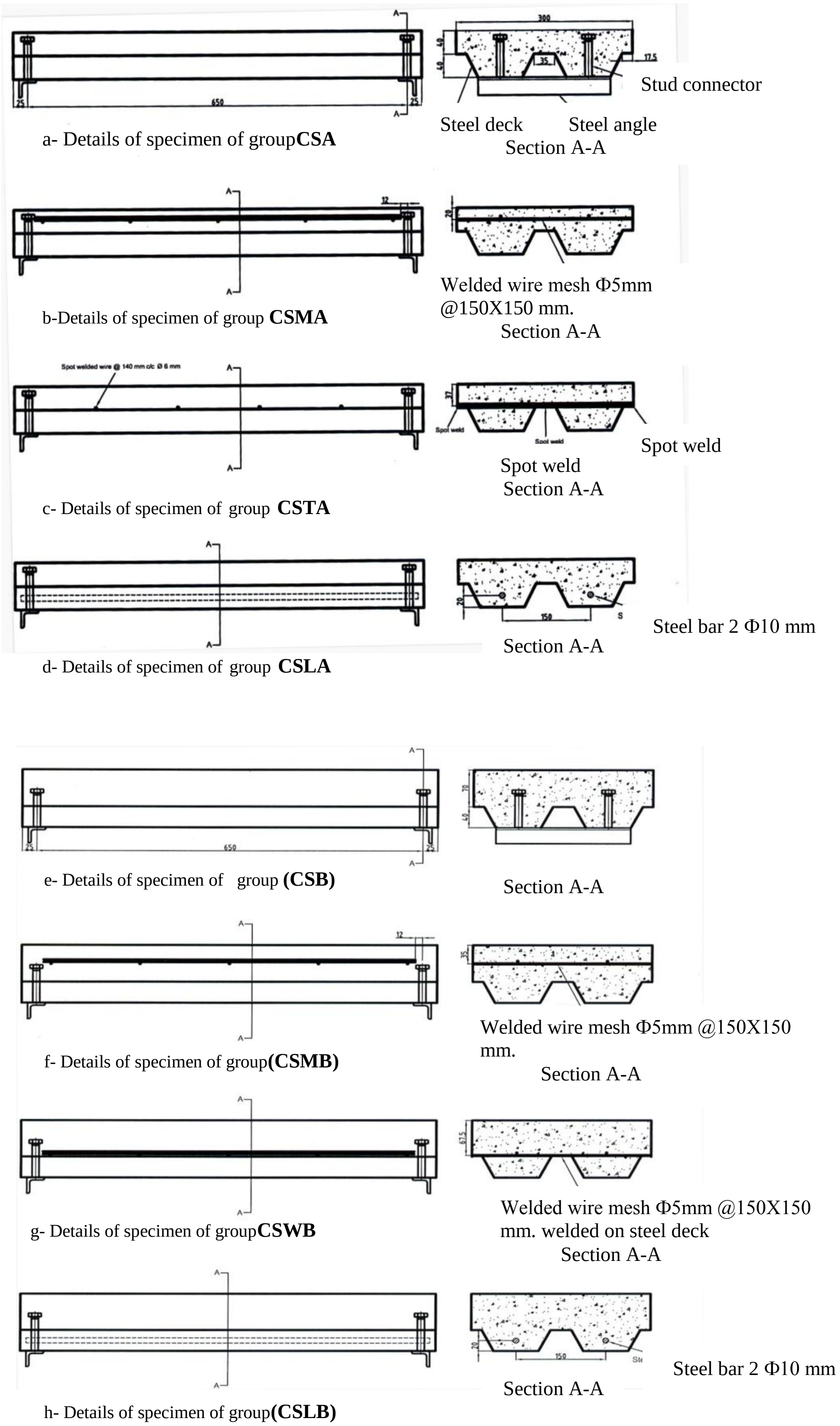

Figure (2) Details of test specimens (all dimensions in mm)

2-2 Loading Setup and Measuring Devices

The specimens were tested over a simply supported span of (650mm), where two-line loads were applied at L/3 from supports. The slabs were tested by using (3000kN) capacity universal testing machine, as shown in **Fig.(3)**.

Dial gauge of (0.01mm) was placed on its position by using a magnetic holder to determine the deflection at mid-span. End slip was measured at each end of specimens by using dial gauges of (0.01mm) fixed over small steel plates which were bonded to each end by adhesive material. **Figure (3)** shows the loading arrangement.

At zero loading, initial readings of dial gauges were recorded, then load was increased gradually in steps while deflection and end slip measurements were recorded simultaneously until failure occurred (defined as the highest capacity beyond which loading drops).

Figure (3) Loading set up and measuring devices

3. Experimental Results

3-1 Ultimate Load Capacity

The failure loads measured for each group in tests are given in **Tables (2)** to **(9)**. It is shown that the ultimate load (defined as the highest capacity beyond which loading drops) for all specimens decreases with the increase of temperature, but this decrease is less at temperature of (150 °C) for (S_2) group (CSA) and (S_{16}) group (CSB) where the percentages decrease for (S_2) and (S_{16}) are (4.5%) and (6.25%) respectively from their ambient temperature load.

Table (2) Test results of slab specimens of group (CSA)

Specimen identification	Temp. stage (°C)	Residual ultimate load (kN)	Percentage residual load (%)	Maximum deflection (mm)	Maximum slip (mm)
S_1	Room temp.	40	100	24.46	5.915
S_2	150	38.2	95.5	24.92	6.45
S_3	300	28	70	25.27	6.94
S_4	600	21	52.5	26.83	7.93
S_5	800	15.2	38	28.67	8.81

Table (3) Test results of slabs specimens of group (CSMA)

Specimen identification	Temp. Stage (°C)	Residual ultimate load (kN)	Percentage residual load (%)	Maximum deflection (mm)	Maximum slip (mm)
S_6	Room temp.	52	100	24.43	5.40
S_7	300	39	75	25.37	6.11
S_8	600	31.1	59.81	27.02	7.82

Table (4) Test results of slabs specimens of group (CSTA)

Specimen identification	Temp. Stage (°C)	Residual ultimate Load (kN)	Percentage residual Load (%)	Maximum deflection (mm)	Maximum slip (mm)
S_9	Room temp.	56	100	9.87	2.05
S_{10}	300	48	85.71	10.79	3.13
S_{11}	600	34.3	61.25	12.13	4.53

Table (5) Test results of slabs specimens of group (CSLA)

Specimen identification	Temp. Stage (°C)	Residual ultimate Load (kN)	Percentage residual Load (%)	Maximum deflection (mm)	Maximum slip (mm)
S_{12}	Room temp.	74.1	100	10.86	3.01
S_{13}	300	67	90.42	11.51	4.03
S_{14}	600	48	64.77	13.02	5.73

Table (6) Test results of slabs specimens of group (CSB)

Specimen identification	Temp. Stage (°C)	Residual ultimate load (kN)	Percentage residual Load (%)	Maximum deflection (mm)	Maximum slip (mm)
S_{15}	Room temp.	64	100	12.85	6.58
S_{16}	150	60	93.75	13.24	6.86
S_{17}	300	44	68.75	14.20	7.25
S_{18}	600	29.3	45.78	15.33	8.32
S_{19}	800	21	32.81	16.09	9.02

Table (7) Test results of slabs specimens of group (CSMB)

Specimen identification	Temp. Stage (°C)	Residual ultimate load (kN)	Percentage residual Load (%)	Maximum deflection (mm)	Maximum slip (mm)
S_{20}	Room temp.	76	100	19.85	6.425
S_{21}	300	70	92.11	20.51	7.15
S_{22}	600	55.2	72.63	22.15	8.11

Table (8) Test results of slabs specimens of group (CSWB)

Specimen identification	Temp. Stage (°C)	Residual ultimate Load (kN)	Percentage residual Load (%)	Maximum deflection (mm)	Maximum slip (mm)
S_{23}	Room temp.	92	100	6.57	2.21
S_{24}	300	86	93.48	8.55	3.82
S_{25}	600	67.6	73.48	10.32	4.24

Table (9) Test results of slabs specimens of group (CSLB)

Specimen identification	Temp. Stage (°C)	Residual ultimate Load (kN)	Percentage residual Load (%)	Maximum deflection (mm)	Maximum slip (mm)
S_{26}	Room temp.	108.5	100	7.82	3.32
S_{27}	300	101	93.1	9.96	4.82
S_{28}	600	78.3	72.17	12.05	6.03

The decrease in the ultimate load of group (CSMB) (having welded wire mesh at middle of concrete thickness) is less than that of specimens of group (CSB) (without additional reinforcement) at the same elevated temperature because the welded wire mesh leads to increase the stiffness of compressive zone (by acting as the compression reinforcement in ordinary reinforced concrete structure) where Bresler [5] and during his study about the causes of collapse of the slab in buildings subjected to fire, found that one of the main causes of collapse, is that the slab had no compression reinforcement in positive moment region although it is unnecessary in the normal condition. The effect of welded wire mesh in group (CSMA) has a little effect on residual load capacity where the percentage decrease of (S_7) at (300 °C) and (S_8) at (600 °C) are (25% and 40.19%) respectively if compared with percentage decrease of specimens of group (CSA) at the same temperature.

The decrease in the ultimate load of group (CSTA) (having spot-welded wire welded on top of steel deck) is less than that of group (CSA) at the same temperature where the residual loads of (S_{10}) at (300 °C) and (S_{11}) at (600 °C) are (85.7%) and (61.25%) respectively from the ambient temperature load, as shown in **Table (4)**. But the residual loads of (S_3) and (S_4) are (70% and 52.5%) respectively from ambient temperature load, as shown in **Table (2)**. The ultimate load of group (CSWB) (having welded wire mesh on the top of steel deck) decreases with a little percentage if compared with the decrease of ultimate load of group (CSB) at the same temperature where the percentage decreases in the ultimate load of (S_{24}) at (300 °C) and (S_{25}) at (600 °C) are (6.52% and 26.52%) respectively if compared with the ultimate load of unheated specimen.

Reinforcing bars are often provided in the composite slab ribs to enhance the fire response where the decrease of ultimate load of group (CSLA) (having reinforcing bars) is less than that of group (CSA) at the same temperature where the residual ultimate loads at (300 °C) and (600 °C) are (90.42% and 64.77%) respectively from the ambient temperature load, as shown in **Table (5)**. From these results, it can be concluded that the effect of bars is obvious on residual ultimate load if compared with results of other groups (CSMA and CSTA) but the effect of bars in group (CSLB) on residual ultimate load is close to the results of groups (CSMB and CSWB) at the same temperature.

3-2 Load-Deflection Relationship

The load-deflection curves for the different temperatures for each group are shown in **Figs.(4)** to **(11)**. The eight groups, show that for each value of load, the deflection increases clearly with the increase in temperature and for specimens at temperature of (150 °C) and (300 °C), the load-deflection curves have only minor deviation from the ambient temperature curves (i.e., the effect of temperature upon the load-deflection curves at first is quite small). This effect is more obvious in the specimens which have the additional reinforcement. Above this range, the shape of the load-deflection curves changes, because both concrete strength and steel strength are reduced at high temperature and the specimen becomes very weak to resist even a small applied load. Therefore, only a very small load can destroy the specimen and causes high value of deflection.

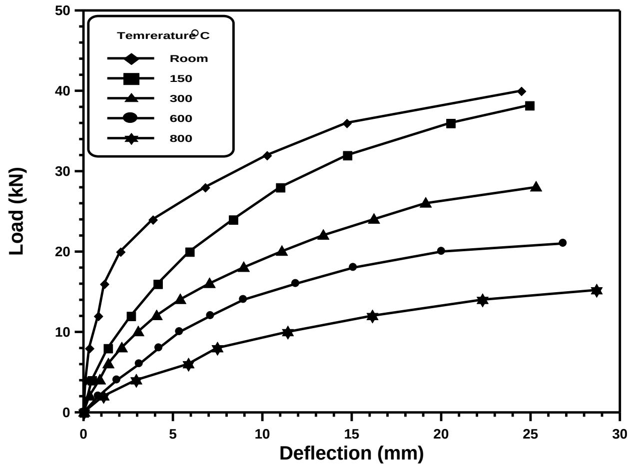

Figure (4) Load-deflection relationship for various temperatures for group (CSA)

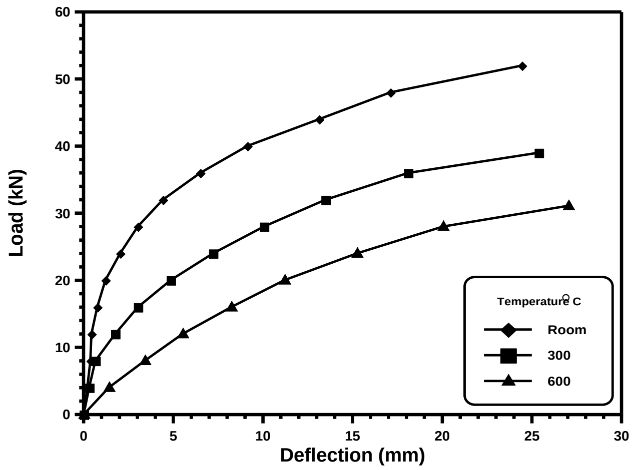

Figure (5) Load-deflection relationship for various temperatures for group (CSMA)

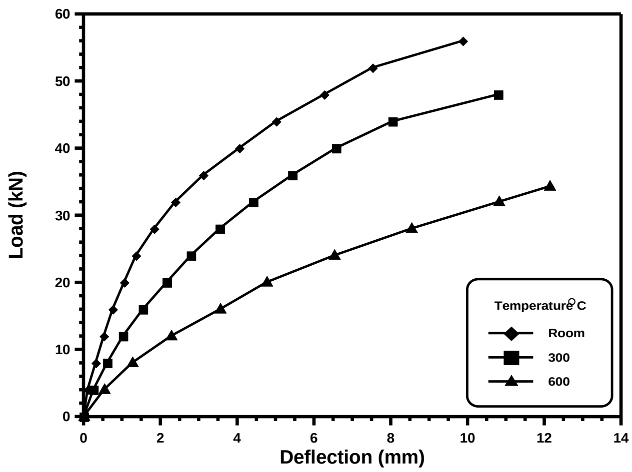

Figure (6) Load-deflection relationship for various temperatures for group (CSTA)

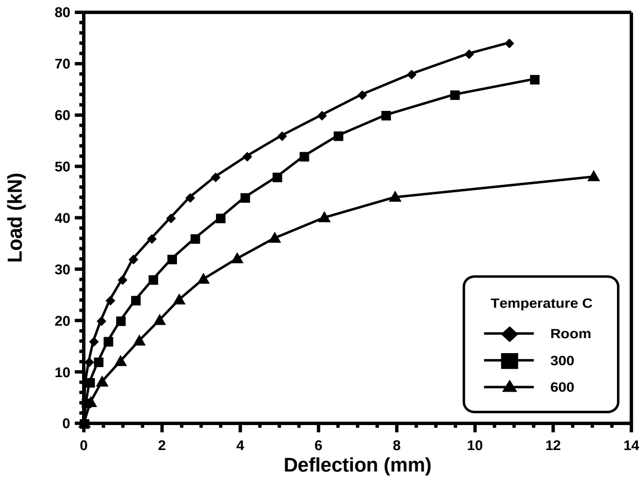

Figure (7) Load-deflection relationship for various temperatures for group (CSLA)

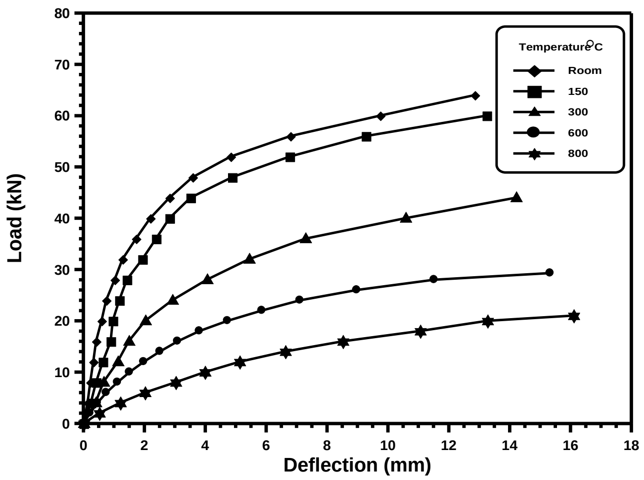

Figure (8) Load-deflection relationship for various temperatures for group (CSB)

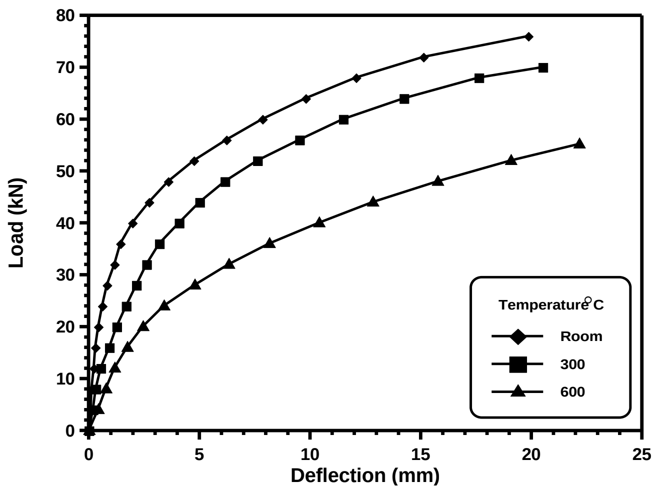

Figure (9) Load-deflection relationship for various temperatures for group (CSMB)

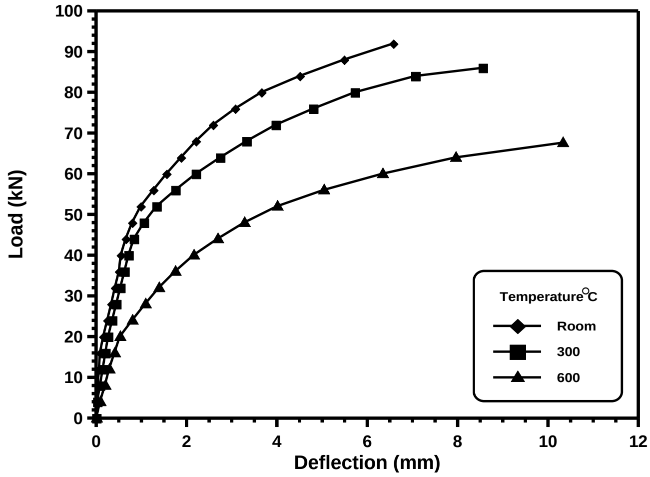

Figure (10) Load-deflection relationship for various temperatures for group (CSWB)

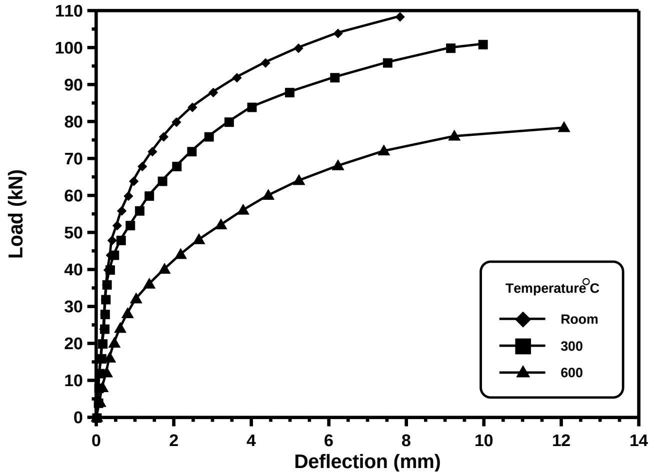

Figure (11) Load-deflection relationship for various temperatures for group (CSLB)

3-3 Load-Slip Relationship

The slip between the concrete and the steel deck is measured at ends of specimens. The load-slip curves for the different temperatures for each group are shown in **Figs.(12)** to **(19)**. The slip is measured at each gradual increment in the load level. At first, these curves show that for each value of load, the slip increases with the increase in temperature. The load-slip curves for unheated specimens display two stages of slip. Initially, load increases insignificantly with no or relatively very small slip up to a critical value of applied load, and with the increase of load beyond this point, the rate of slip increases significantly with load increase until failure occurs. The first stage becomes more obvious at the specimens having interlocking devices and specimens having tensile bars. The appearance of two stages is due to the fact that the bond (chemical bond and mechanical interlocking) between concrete and steel deck prevents the slip occurrence in first loading stages and whenever the bond is more secured, the occurrence of slip will be delayed. Hence, it can be considered that, initial slip is due to the loss in bond and crushing the concrete surrounding the interlocking devices. Also, the concrete strength is reduced to such an extent that the concrete beneath the ribs of bars fails during the heating cycle [6] and that becomes clear at high temperature. It is observed that at temperature of (150 °C and 300 °C), the load-slip curves have only little deviation from ambient temperature curves and this is more obvious for specimens that have the depth of (110 mm). Above this range, the shape of the load-slip curves changes at higher temperature (600 °C and above), the bond nearly diminishes and the observed two stages in the curve at lower temperature, cannot be recognized.

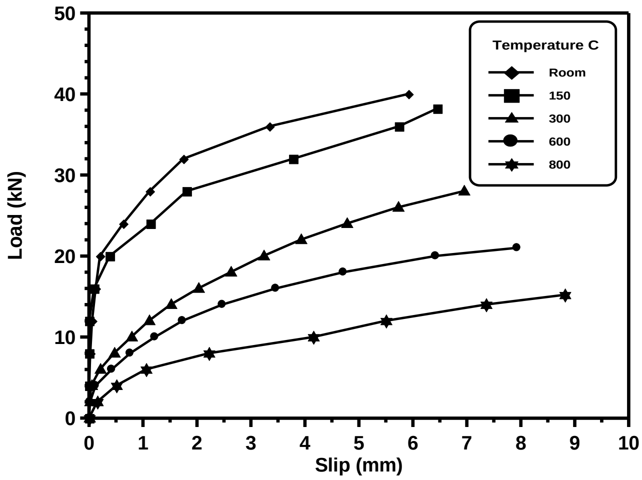

Figure (12) Load-slip relationship for various temperatures for group (CSA)

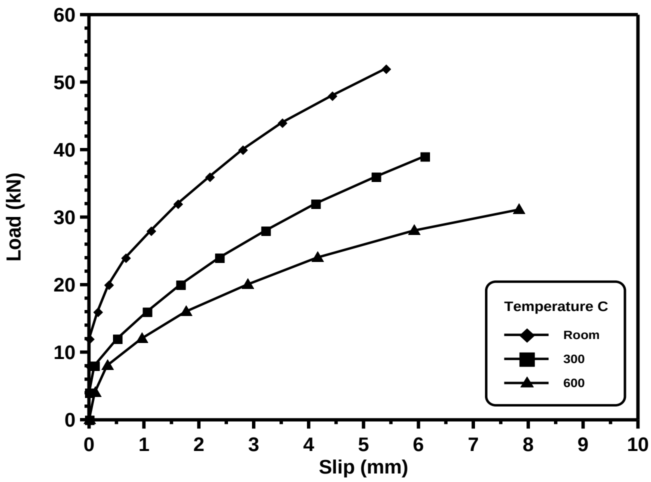

Figure (13) Load-slip relationship for various temperatures for group (CSMA)

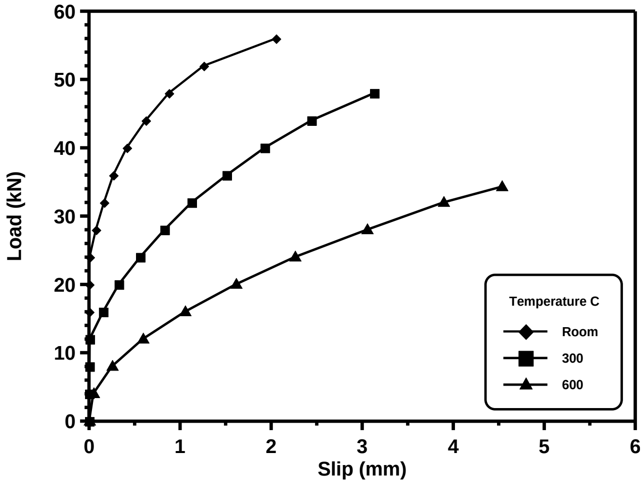

Figure (14) Load-slip relationship for various temperatures for group (CSTA)

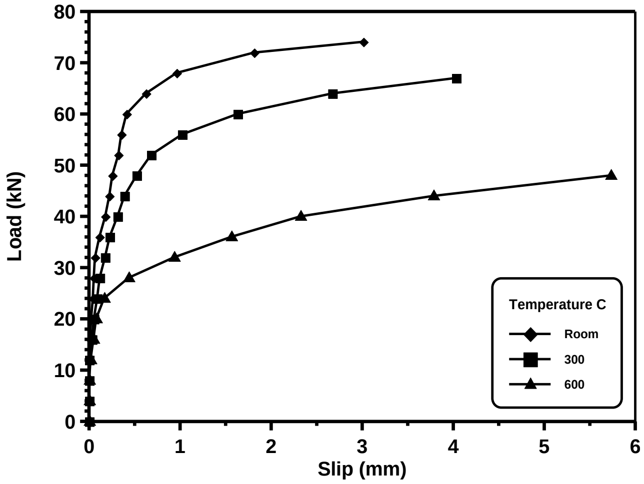

Figure (15) Load-slip relationship for various temperatures for group (CSLA)

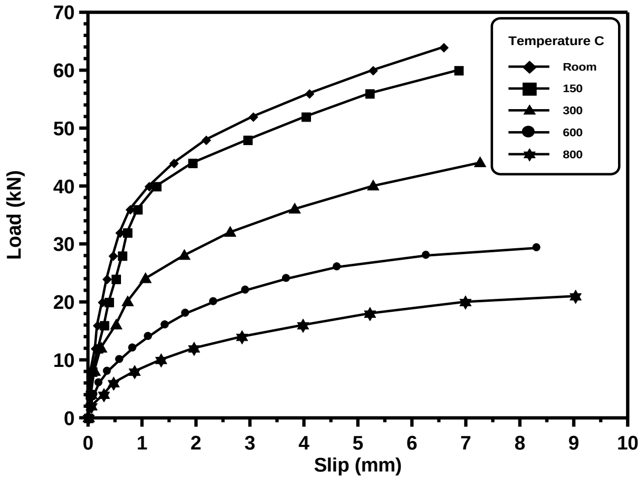

Figure (16) Load-slip relationship for various temperatures for group (CSB)

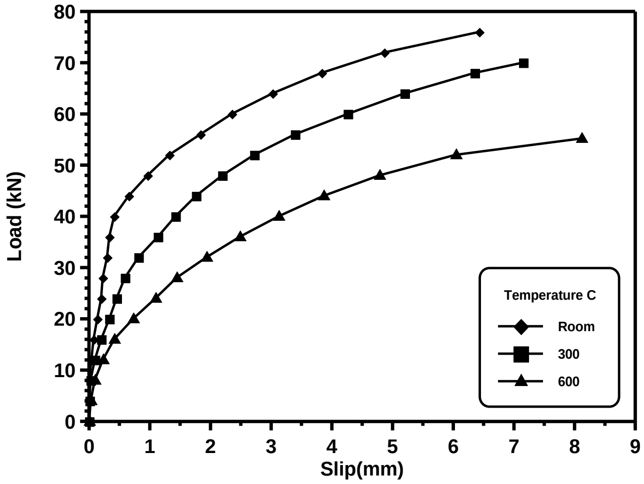

Figure (17) Load-slip relationship for various temperatures for group (CSMB)

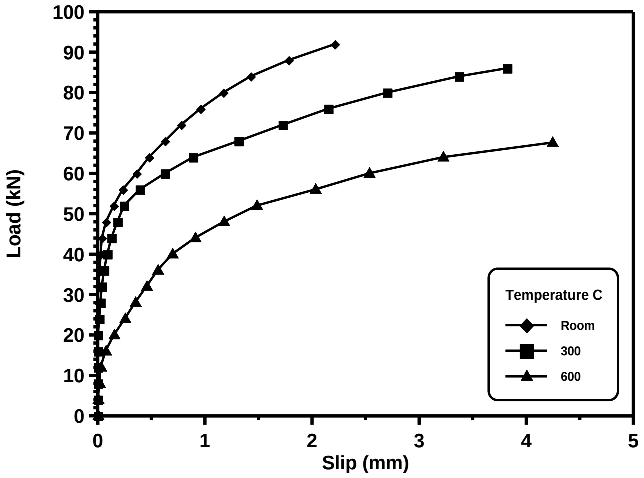

Figure (18) Load-slip relationship for various temperatures for group (CSWB)

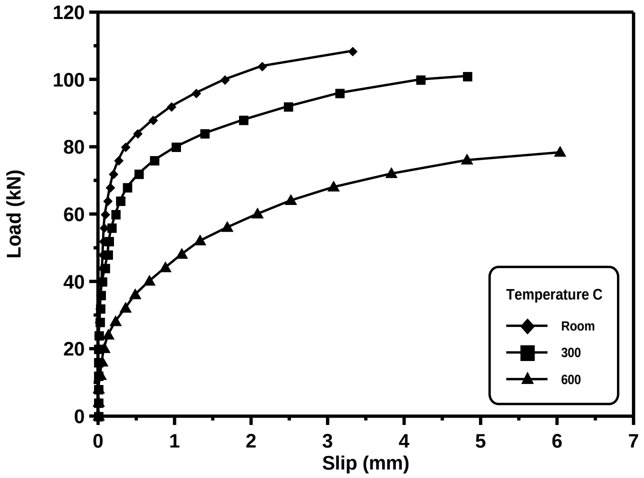

Figure (19) Load-slip relationship for various temperatures for group (CSLB)

3-4 Ultimate Load-Temperature Curves

The residual load-temperature curves for both groups which have slab depth of (80 mm) and (110 mm) are shown in **Figs.(20)** to **(21)** respectively. It is observed that for all groups, the residual load is less than the load of unheated specimens and for all the studied range of temperature. For both groups (CSA) and (CSB) (without additional reinforcement), which are subjected to temperature levels of (150, 300, 600 and 800 °C), three stages for load behavior under elevated temperature are recognized. Initially, a little decrease in the ultimate load takes place when the temperature increases from room temperature to (150 °C). Then, and at interval from (150 to 300 °C), a clear decrease in the ultimate load is observed. Then, the third stage shows a continuous decrease in the ultimate load with temperature increase. With respect to other groups, which are subjected to temperature levels of (300 and 600 °C), two stages appear in load-temperature curves where a clear decrease in the ultimate load takes place when temperature increases from the ambient to (300 °C). Then, a continuous decrease in the ultimate load with temperature increase is observed.

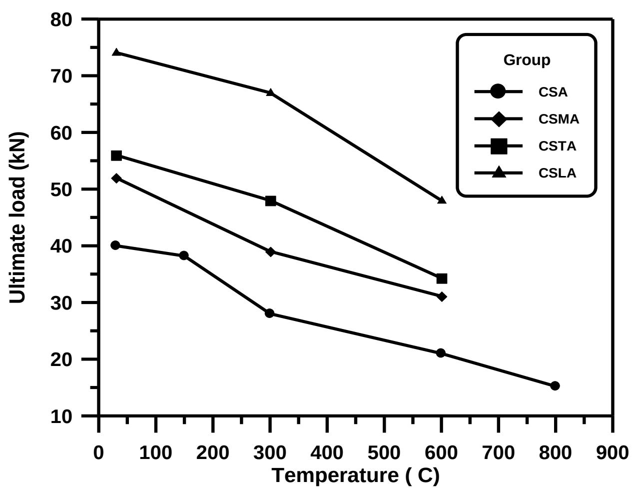

Figure (20) Variation of ultimate load with temperature for different additional reinforcement for slabs of depth of (80 mm)

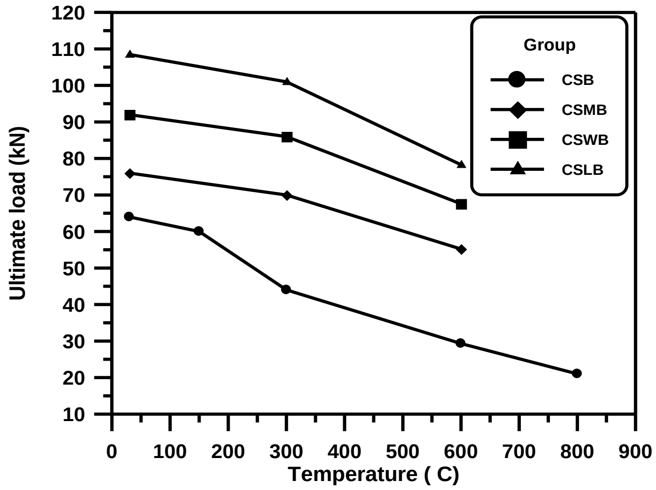

Figure (21) Variation of ultimate load with temperature for different additional reinforcement for slabs of depth of (110 mm)

3-5 Failure Mechanism and Crack Patterns

Most of slab specimens fail by shear-bond failure and this becomes more obvious gradually with the increase in exposure temperature in comparison with room temperature.

Shear-bond failure arises from the loss of composite action because of inadequate shear transfer at the interface between the concrete slab and the steel deck. It is characterized by the formation of a major diagonal crack in the concrete at or near one point load and by horizontal slip and vertical separation between the concrete and the steel [7].

For the specimens without additional reinforcement, the major crack occurs in shear span towards the line load, and vertical separation occurs under line load near the major crack.

With respect to the specimens which have welded wire mesh placed at the middle of concrete thickness, flexural cracks appear near line load and a vertical separation occurs at mid-span of specimen and this separation increases with the increase of temperature. Yielding of steel plate occurs at this separation. Cracks in top surface of concrete continue from major cracks to other side and no additional cracks appear at the top surface of concrete, because the welded wire mesh leads to increase the concrete stiffness and prevents the occurrence of additional cracks.

For specimens which have spot-welded wires, flexural cracks appear near line load and no separation is observed at room temperature but when the exposure temperature increases, the diagonal tension crack with a vertical separation near the support is observed because the bond becomes less when the temperature increases. With respect to the specimens having the welded wire mesh welded at top of steel deck, no separation between concrete and steel deck is observed at the studied ranges of exposure temperature and a diagonal tension crack appears and extends toward the line load in all specimens and for all temperature levels.

For specimens having additional tensile reinforcement, number of flexural cracks at the region of constant moment is observed and before the ultimate load is reached, the diagonal tension crack appears and grows towards the line load. This is observed for a specimen subjected to temperature (300 °C and below), but at high temperature, the major crack will appear near the support extends towards the line load. No yielding of steel deck was observed at room temperature, but with the increase of temperature, the yielding of steel deck was observed under line load and especially at separation region. Number of cracks, in top surface of concrete initiated from transverse major crack and extended towards the end of specimens and this crack increased gradually with the increase in temperature. The failure mechanism is shown in **Fig.(22)**.

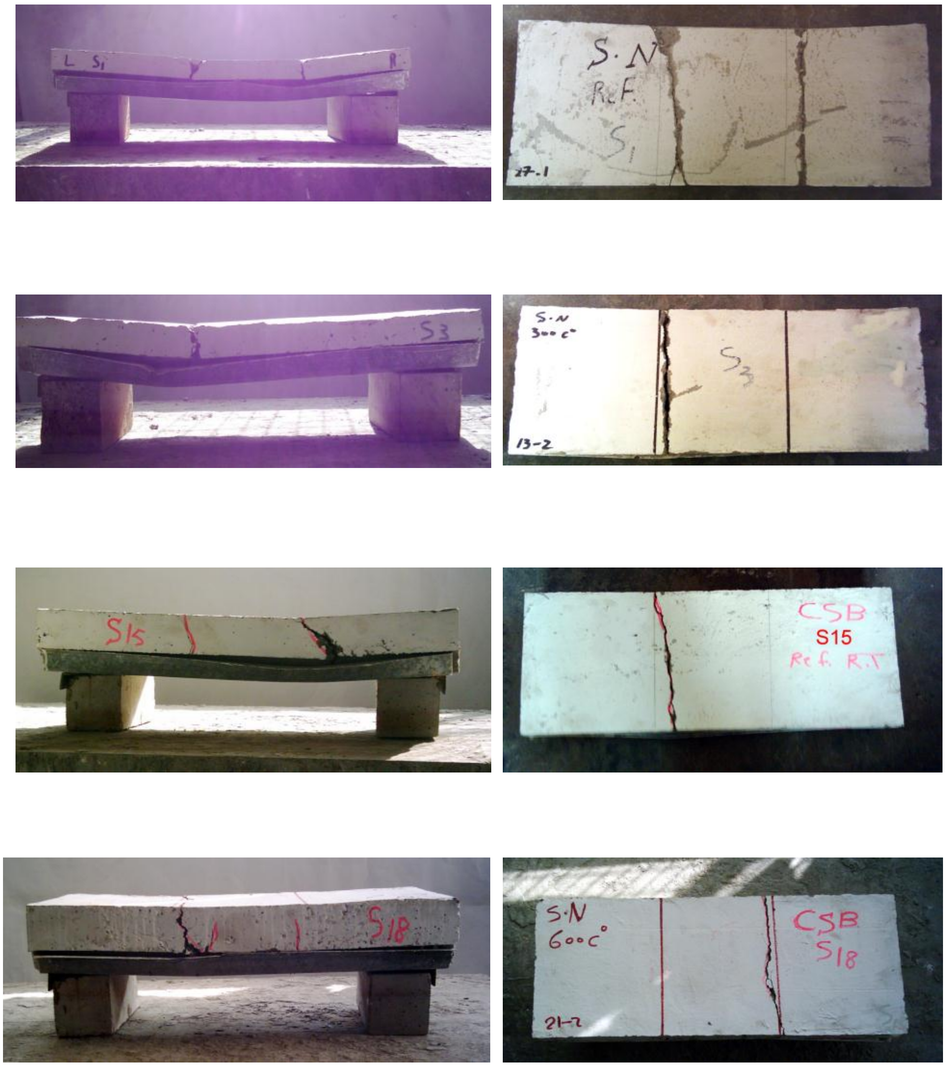

Figure (22) Failure mechanism

4. Conclusions

1. The ultimate load capacity for composite slabs decreases significantly when subjected to high temperature and the percentage reduction depends mainly on the temperature level and time of exposure.
2. The decrease in the ultimate load capacity becomes less when additional reinforcement is used in composite slabs where:

   (i) The slabs with welded wire mesh placed at the middle of concrete thickness have a significant resistance to high temperature if compared with slabs without additional reinforcement. This is more obvious for slabs which have the large depth, where an increase in the ultimate load of about (18 %, 59 % and 88%) at (room temperature, 300 ^{O}C and 600 ^{O}C) respectively takes place when welded wire mesh is used.

   (ii) The composite slabs with interlocking devices (spot-welded wires and welded wire mesh welded on top of steel deck) show ultimate load greater than slabs without additional reinforcement and for all studied ranges of exposure temperature. An increase in ultimate load is about (40%,71% and 63%) at (room temperature, 300 ^{O}C and 600 ^{O}C) respectively when spot-welded wires are used and it is about (43 %, 95% and 130 %) at (room temperature, 300 ^{O}C and 600 ^{O}C) respectively when welded wire mesh welded on steel deck is used.

(iii) The decrease in the ultimate load capacity become less when tensile bars are used in composite slabs and these slabs show higher load capacity if compared with other slabs with other additional reinforcement. An increase in ultimate load takes place at (room temperature, 300 ^{O}C and 600 ^{O}C) which is about (85%, 139% and 129%) for slabs having the small depth and it is about (70%, 130% and 167%) for slabs having the large depth.

3. The load-deflection curve is affected by the increase in exposure temperature, where at high temperature the relationship becomes with flatter slop than those at room temperature. This is noticeable at temperature above (300 ^{O}C) and the load-deflection curves at temperature (300 ^{O}C and below) have only a minor deviation from the ambient temperature curve.
4. The load-slip curves for unheated specimens display two stages of slip. Initially, load increases with no or relatively very small slip then the rate of slip increases with the increase of load until failure occurs and when the exposure temperature increases, specially at temperature on (600 ^{O}C), the two stages cannot be recognized and the curve becomes with flatter slop than those at room temperature.
5. At normal and high temperature, shear-bond failure is a common failure mechanism which takes place in most of slab specimens characterized by the formation of a major crack at or near one line load and by a horizontal slip and vertical separation between the concrete and the steel. This mode of failure becomes more obvious gradually with the increase in the exposure temperature.

5. References

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