

Performance Evaluation of Mero Jointed Composite Uhpc Slab Steel Space Frame Structures

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Abstract

Spaceframe is a structural roofing system that generally made of steel tubes connected together by ball joints which is called MERO jointed space frame structures and mostly used for covering large space area. In addition, some advantages could be obtained with such a system, for example the lower weight, the high strength-to-weight ratio and the low cost. This study aims to use such a known roof structure for a composite structure in which Ultra-High-Performance Concrete slabs are used to withstand various loads as a structural floor system. Various inclinations of the main elements for space frames were tested, namely 30 °, 45 ° and 60 °. The composite effect is accounted for by testing composite and non-composite samples. The test results were evaluated and compared against several performance indices, such as: Ultimate load, stiffness, hardness, ductility, ductility index and absorbed energy. Test results have shown that spaceframe models with an angle of 60 ° have the highest load-carrying capacity compared to other angles and the highest toughness compared to various techniques. Putting together 40mm UHPC slab panels reflected a slight increase in the models used in this study than traditional spaceframe samples.

Keywords: Spaceframes, Composite Spaceframes, Mero joints, ultra-high-performance concrete

1. Introduction

Space trusses are three-dimensional (3D) reticulate systems used worldwide because of their advantages in covering large, free spaces. In addition to having low cost, the weight itself is relatively reduced. Space trusses are versatile in a range of applications, from small ornamental marquees, the cover of warehouses, gymnasiums, hangars, and shopping centers, to helipads, ...etc. [1-3]. Space trusses were copied from nature. The natural elements always seek to minimize stress and maximize strength in an efficient way, taking advantage of the load capacity of all members of the body [4, 5]. The natural shapes have exceptional stiffness and use minimum materials to obtain the maximum structural advantage. The natural forms act in the direction of the least force.

Despite of the advantages of the space truss roof systems mentioned above, these types of systems are very prone to progressive collapse initiated by buckling of the truss bars [6, 7]. In particular, a space truss roof in which the buckled truss bars gather in a certain region across the roof may entirely collapse suddenly without any indication and warning [8]. A numerical investigation on the progressive collapse behavior of double-layer space truss roofs has

proved the sensitivity of these structures against the progressive collapse when subjected to increasing applied load [9-11]. However, even though some truss bars buckle in flexural mode, the entire roof may not collapse because the system is capable of allowing redistribution of axial forces to a small extent [12].

For instance, if a space truss roof is subjected to soil-induced settlements resulting in disproportionately relative displacements among the roof supports in time, axial force redistribution in truss bars is likely, but still limited to negligible level [13].

Many researches have been conducted on the investigation of collapse behaviors of the space truss roof systems. One has reported the investigation of a space truss roof designed to cover a reinforced building that has totally collapsed due to snow load initially underestimated and some mistakes made in the design [14]. Additionally, the partial collapse of a space truss roof of an industrial plant has been investigated in [4] and it has been reported that the collapse was because of ice ponds after exceptional snow load. Another research [9] has addressed a long-span steel roof structure collapsed during construction as a result of an out-of-plane

buckling phenomenon caused by a gust of wind. The partial collapse of a space truss roof structure that occurred during strong winds and heavy rains was investigated based on the site observation and experimental study on the bolts [8, 13, 15]. An experimental study was performed to investigate the ductility behavior of a space truss roof system that consists of cold-formed hollow square sections attached to a joint through the special welded joint plates and bolted connections [16]. A methodology allowing to perform nonlinear post buckling analysis of space truss systems was developed and applied to double-layer space truss to obtain the vertical load displacement response [17]. A numerical research based on a nonlinear stepwise linearization analysis method was carried out to investigate the nonlinear behavior of space truss members [18].

This study aims to evaluate and identify capacity performance of composite and non-composite spaceframes systems, under different vertical diagonal members angle, and check their ability to withstand floor loads through connecting a concrete slab layer made with plain ultra-high-performance concrete (UHPC) to the conventional space frame.

2. Description of the models

Three different type of conventional SF models were prepared for testing. Each specimen has different angle of the diagonal hallow circular members, they are; angle 30, 45, and 60. Each specimen was designated with a code as listened in Table (1). As a result of variable angle of connected members, the spaceframes heights were also varied, started from 302 mm for SF-A30, to 514 mm for SF-A60. As illustrated in Figure (1), All SFs chord members dimensions are the same in top view direction. While the diagonal members were varied based on SF height. In general, the joint steel ball has 90 mm diameter, the thickness of used hallow circular section is about 2.2 mm, and outer diameter of 42 mm. The total length of SFs c/c of balls is about 1200 mm, and 600 mm c/c of ball for width. The same dimensions were adopted for Composite spaceframes models, (as clarified in Figure (2) which also have

designated with codes as mentioned in Table (1). The CSFs have the same dimensions mentioned previously in SFs models, except three mains different; The first, is the concrete deck slab of dimensions 700 mm width, 1300 mm length, and 40 mm thickness. The second, is the steel supporting plate placed under deck slab and above ball joints. The corner square plates were with dimensions of 100 mm and 10 mm, for length and thickness, respectively. While the mid support plates were with dimensions of 200x100x10 mm, for length, width, and thickness, respectively. the third, a threaded bolt of 25 mm diameter connected mechanically to the ball joints through the whole slab thickness. It worth to be mentioned that all parts dimensions were selected according to what available locally. Figures 1 and 2 illustrate specimen's configurations for SFs and CSFs, respectively.

Table1: Specimens' designation of B-SFs and B-CSFs

SPECIMENS DESIGNATION	TYPE OF SPACE FRAME	ANGLE OF SHAFT DIAGONAL MEMBERS	SPECIMENS HEIGHT, MM
B-SF-A30	Non composite	30	325
B-SF-A45	Non composite	45	511
B-SF-A60	Non composite	60	816
B-CSF-A30	Composite	30	375
B-CSF-A45	Composite	45	561.
B-CSF-A60	Composite	60	866

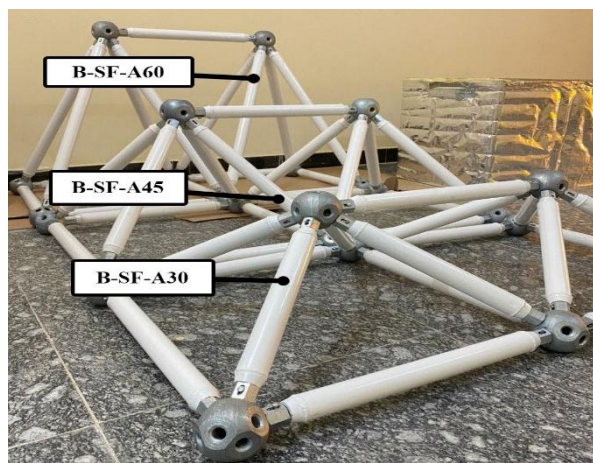


Figure (1): Typical configurations of SFs specimens

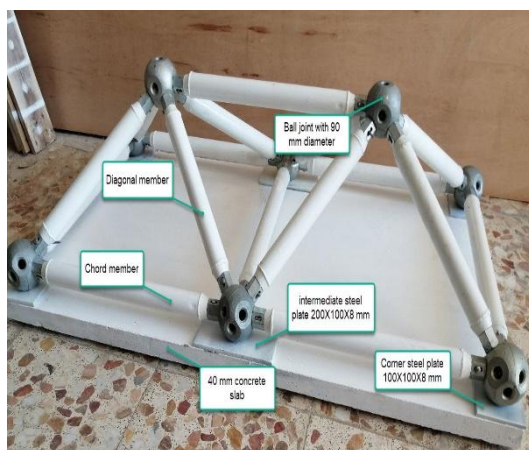


Figure (2): Typical configurations of CSFs specimens

3. Materials Characteristics

3.1 UHPC Mixing component's ratios

The main components of the prepared UHPC mixture including the following materials, ordinary Portland cement (CEM II/A-LL type according to BS EN 197-1, 2011), local limestone aggregate (the coarse aggregate was eliminated since the prepared layer was relatively thin) with 0.6 mm maximum aggregate size), graded according to ASTM C33

limitations. the other supplementary components included silica fume (SF), incorporated according to ASTM C1240 using hybrid method (addition and replacing), poly carboxylic polymer (3rd-generation type) superplasticizer, Micro steel fiber (MSF) having aspect ratio of 60, and tensile strength of about 2850 MPa, and finally portable water. this study, the adopted optimum mixing ratios of the components per one cubic meter were listed in Table (2). It worth to be mentioned that the final ratios have been achieved after many trails obtained from previous researches [19]. The prepared mixture then plastid in a steel model having thickens of 40 mm to produce UHPC slab, as shown in Figure (3)

Table2 : Final mix design weights of UHPC components per one cubic meter [19]

Cement	Fine sand, kg	Supplementary S.F., kg	Added S.F., kg	w/cm	Super., kg	Steel Fiber, kg
900	1100	100	100	0.20	40	353.25(4.5%)



Figure (3): concrete slab after casting

3.2 Steel properties

For the steel tube bar properties, a representation sample has been tested locally in S.I.E.R general company- engineering laboratories, in term of yield stress, rapture strength, and maximum elongation. Tests results can be seen in Table (3).

Table(3): Steel properties of tube members

sample	Yield stress, MPa	Rapture strength, MPa	Elongation, %
Tube shaft	389	487	20

4. Testing configuration

The B-SF-AX specimens were tested with upside down configuration to facilitate testing and avoid any unexpected errors during loading increment. load application was a two-point load applied quasi statically on a thick steel plates aligned at the centers of the specimen. All bottom 6 joints were interlaced with the support plate using 25 mm diameter bolt connected by two nets as illustrated in Figure (4). The applied load was achieved using hydraulic press machine. A load cell of capacity bout 2000 KN has been installed under the hydraulic jack to measure applied load. Additionally, displacement readings were achieved using two LVDT sensors installed underneath the top two joints of the specimens.

While for B-CSFs, displacement devices measurement arrangement was as illustrated in Figure (5). The loading steel plate of width 100 mm and length 700 mm was installed at top mid slab and under load cell. Two LVDT sensors were installed under the top mid joints to measure displacement during the test. Moreover, a very high accuracy Laser sensor were installed under concrete midspan to measure slab deflection. The data acquisition system connected to a computer device which is consider as the final station for processing and storing received data. It worth to be mentioned that the computer system was equipped with LabView version 2018 to manage all data received from the sensors, as can be seen in Figure (6).

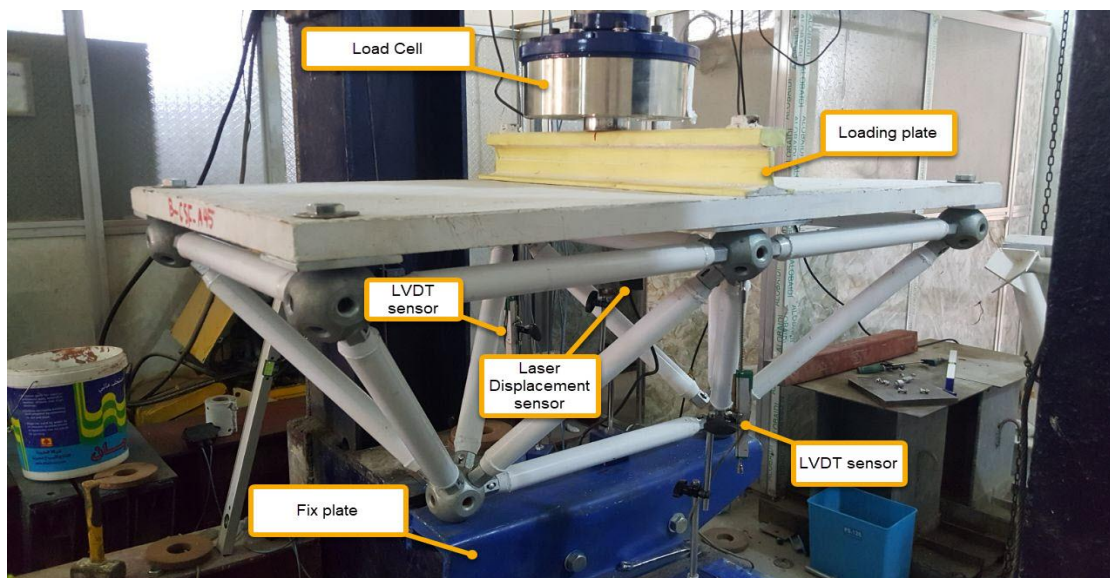


Figure (4): Typical test configuration of CSFs specimens

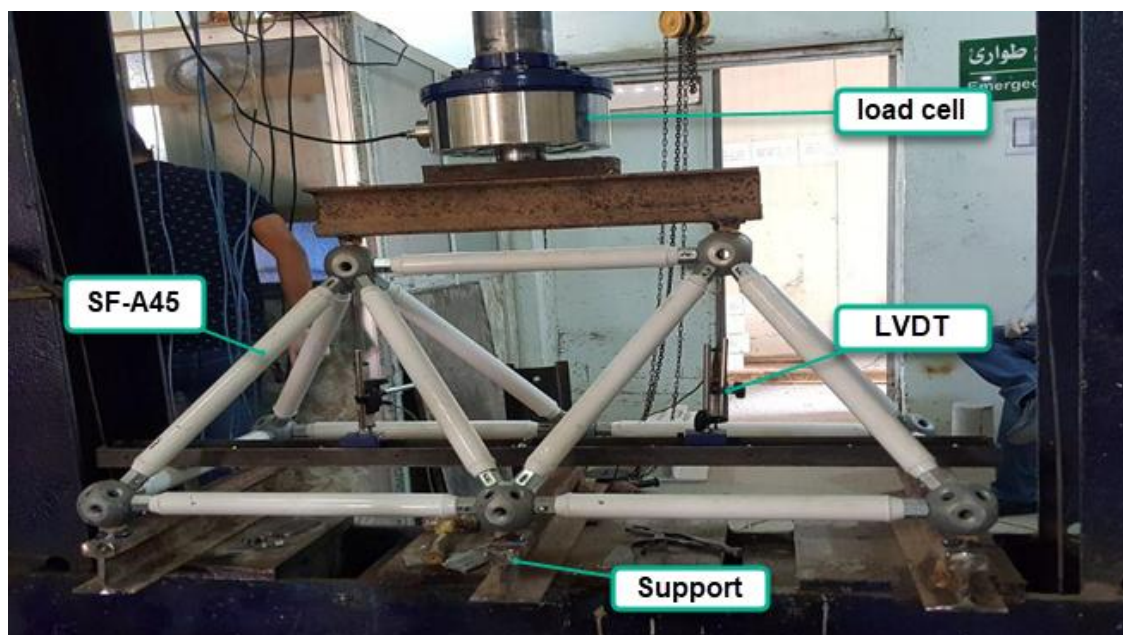


Figure (5): Typical test configuration of SFs specimens

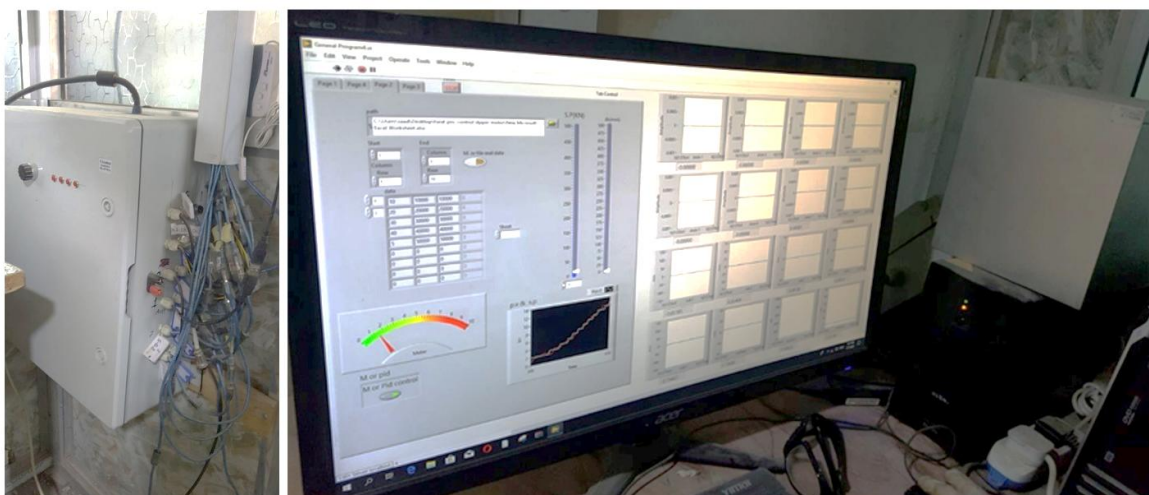


Figure (6): Data acquisition and load control monitor using LabView software

5. Results and Discussion

5.1 Mechanical properties and failure modes for non-composite spaceframes specimens

Three different common types of failure could be appeared in spaceframes, they are: buckling of the compressive diagonals, brittle failure of the connecting joints, and ductile tension failure of the chords. While for the composite spaceframe's specimens, additional failure mode possibly occurred, which is the concrete slab failure before the spaceframe's elements reaching yielding limit. Many common types of concrete failure could be happened, such as; bending failure, or shear failure. the failure by reaching maximum allowable compressive stresses may be eliminated due to the nature of slab support. Shear and tension failure could be highly appeared during the test, since no steel bar reinforcement existed in the slab.

For non- composite spaceframes specimens, as illustrated in Figure (7) the failure was occurred in the diagonal's members, by reaching the yield limit compressive stresses at first, followed by members buckling and hardening till reach failure load. A general bulking type of the diagonal members were appeared with load increasing on the top two joints. The global buckling failure started to be more clearly with load increment after yield point. No rule could be mentioned here for the main chord's members, since they were tied at the six bottom joints interlocked by steel bolt with 25 mm diameter at the base plate strips. Also, no local buckling in the tube members were appeared during the test, even when with reaching failure load. Further, a noticeable rotation was appeared in the top loaded joints relatively in some tested specimens.

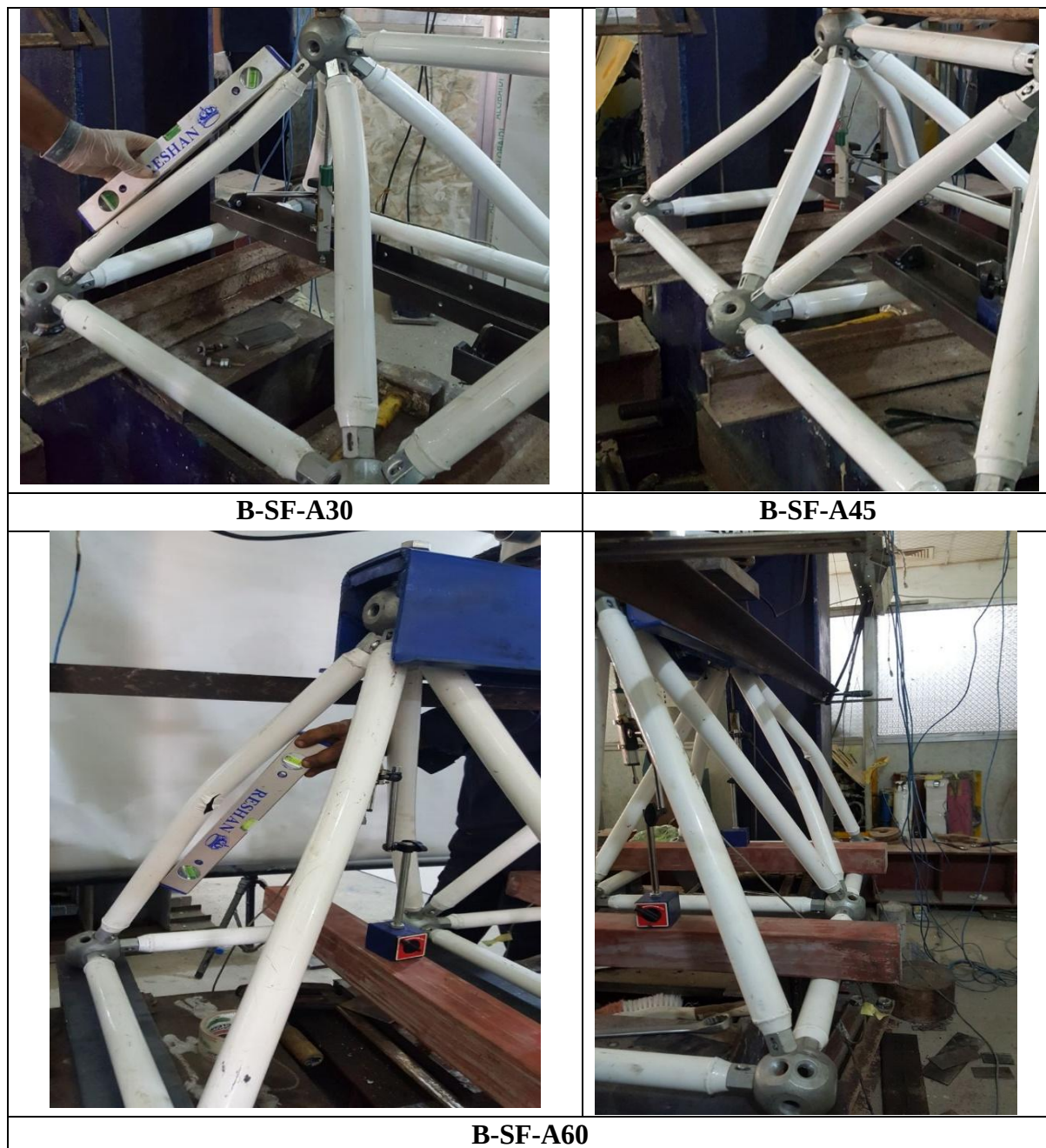


Figure (7): Failure modes of SFs specimens having variable diagonal vertical member angles

By comparison of these three mentioned specimens, and as illustrated in Figure (11), B-SF-A60 specimen showed the highest load resistance, which was about 1.27 times the load resistance of B-SF-A45, and about 1.66 times the load resistance of B-SF-A30. In the contrary to displacement at ultimate load of B-SF-A60, which showed the lowest value among other specimen's values.

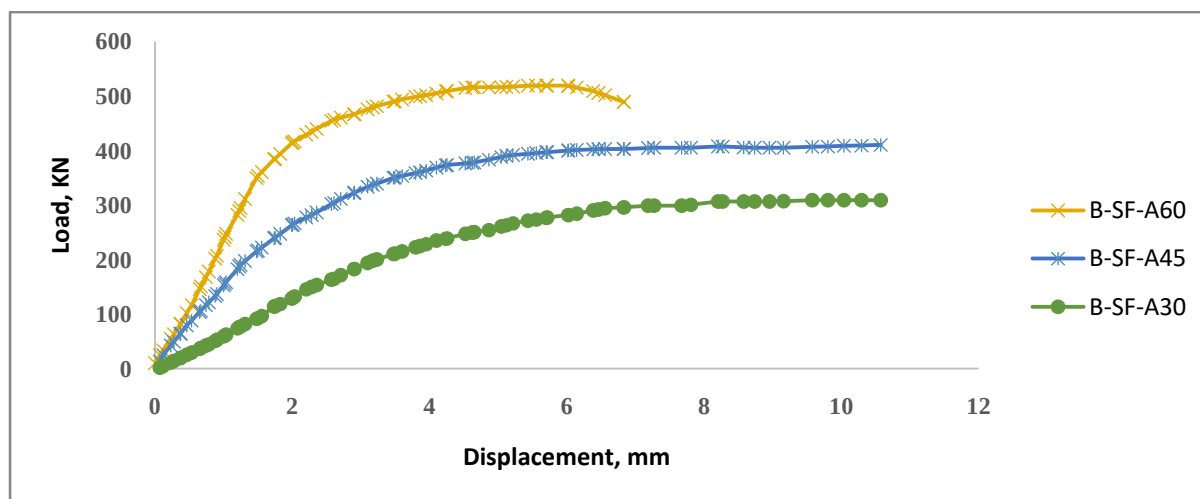


Figure 8: Load- displacement relationship of SFs specimens having variable diagonal member angles

5.2 Performance and Failure Modes results in Composite Spaceframes specimens

Three common possible failure sequence of composite spaceframes, they are, the first failure of steel members by yielding or buckling (global or local) before the slab concrete, the second, failure of concrete deck slab by flexural or shearing stresses before reaching yield limit of steel members, and the third, failure of the steel members and the concrete slab together. It is too important to compatible such elements to gather, to make sure that all structure elements reached their full capacity together. The compatibility could be achieved either with spaceframe angle type (height), thickness of tube members, or steel strength. from the other side, the concrete compressive strength, slab thickness, and loading nature play major rule to get compatible failure. tests results have shown that all composite spaceframes specimens collapsed by reaching the concrete slab to the maximum tensile stresses limit. Some of the concrete slab specimens were failed near the joints by excessive shearing stresses, as notified by the crack's patterns. The system was collapsed due to reaching the top mid chord member to the maximum allowed stresses near the necking cone at the left end. The failure was in lactation where the cone steel part and the bar welded together. Such location is weak point to start failure, and that what happened, the stresses were concentrated at this point resulted in a sudden failure. additionally, a local buckling has been observed in the mid diagonal members.

Moreover, concrete tensile cracking under the line load in short direction has been observed at the ultimate load, after the collapse. Further, concrete spalling out from the top mid right location near the connected bolt has been observed duce to the loss of top mid tube

member suddenly, which reflected an impact load to the connected joints between the slab and the ball. It worth to be mentioned that no failure has been observed in the joints, no shear failure was occurred in the bolts those connects the concrete and the ball joints. Also, no local buckling was observed in the diagonal steel tube members.

In general, as clarified in Figures (10 and 11), The behavior of B-CSFs specimens with angle 30, 45, and 60 were compared in term of many performance indexes. The load displacement relationship as illustrated in Figure (10), has shown that B-CSF-A60 had maximum load carrying capacity which was higher than B-CSF-A45 and B-CSF-A30 by about 1.36 and 2.75, respectively. followed by B-CSF-A45, which was higher load capacity then B-CSF-A30 by about 2 times. B-CSF-A30 has the lowest load capacity among other Ball jointed composite specimens. while B-CSF-A60 has the highest load capacity among other specimens.



Figure (9): typical failure mode of B-CSF specimens

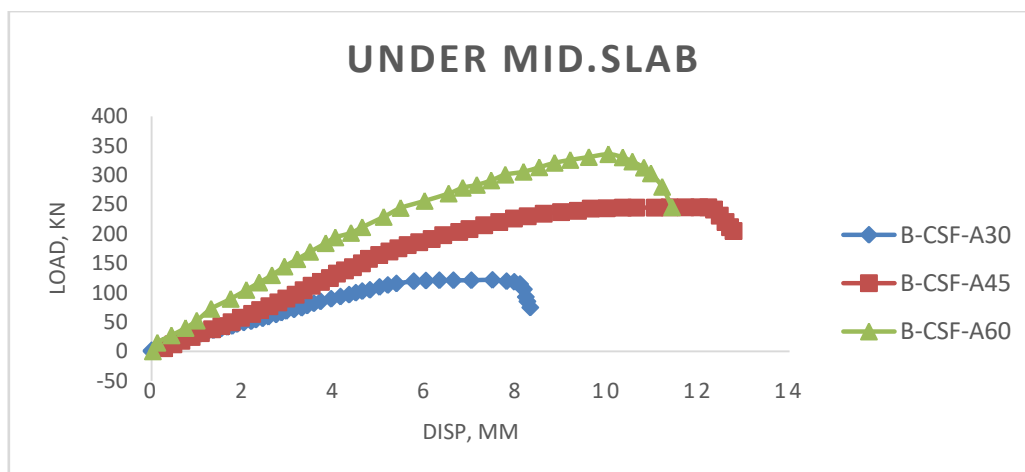


Figure (10): Load- displacement (under slab midspan) curves behavior comparison between various B-CSFs specimens

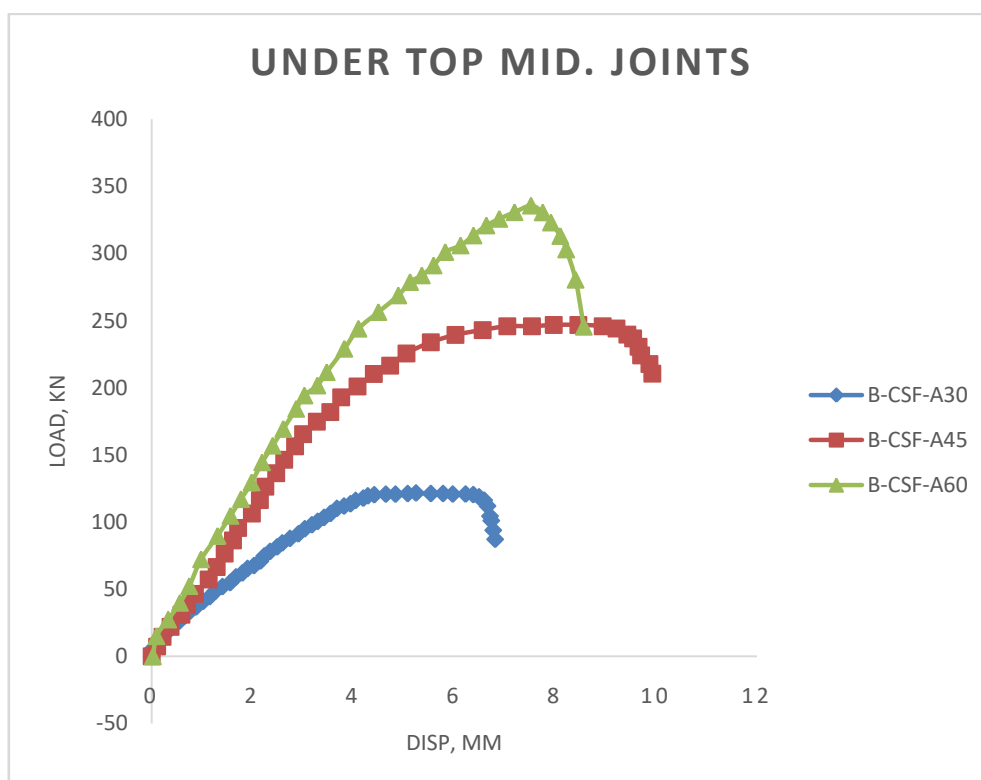


Figure (11): Load- displacement (under top joints) curves behavior comparison between various B-CSFs specimens

5.3 Performance indexes comparison between SFs and CSFs

Due to the nature of the support and loading conditions, it may be inappropriate to compare the two systems mentioned. Even so, a general comparison was made to check performance variations in and for each individual system and to illustrate SFs and CSFs in relation to various performance indices such as load-bearing capacity, hardness, stiffness, ductility, ductility index and absorbed energy, as can be seen in the following figures. As can be seen in Figure (12), B-SF-A60 specimens had the highest load carrying capacity among

other specimens, followed by B-SF-A45 and then B-SF-A30. For composite spaceframe specimens, the same arrangement was observed.

In case of hardness property, (which means the ultimate load resistance in KN over its corresponding displacement in mm), for B-SFs specimens, B-SF-A60 specimens have better performance than B-SF-A45 and B-SF-A30 specimens by about 2.4 and 3.69 times, respectively. While B-SF-A45 have better performance by about 1.53 times B-SF-A30 specimens. while for B-CSFs specimens, B-CSF-A30 had the highest value of hardness property among other specimens, while B-CSF-A30 possessed the lowest value. hardness was increased about 1.66 times and 1.26 times when compare B-CSF-A60 with B-CSF-A45 and B-CSF-A30, respectively.

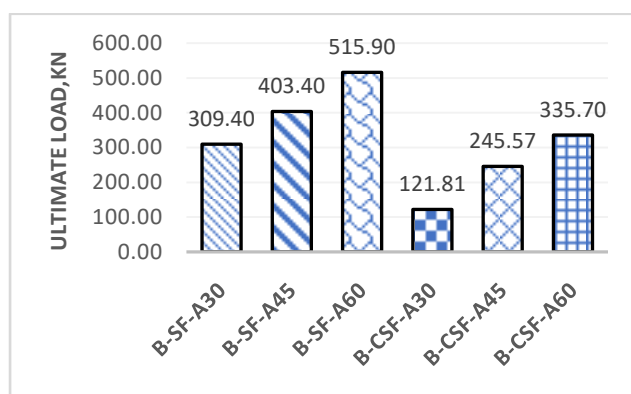


Figure (12): Ultimate load comparison of B-SFs and B-CSFs

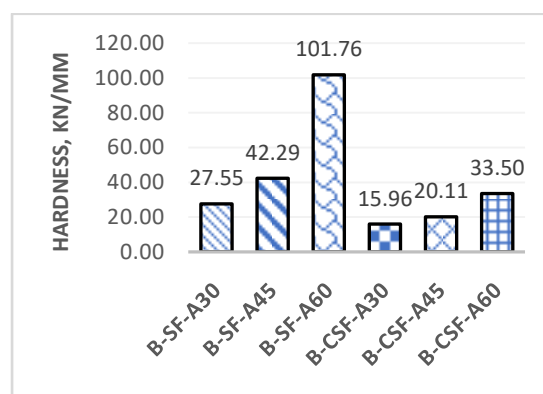


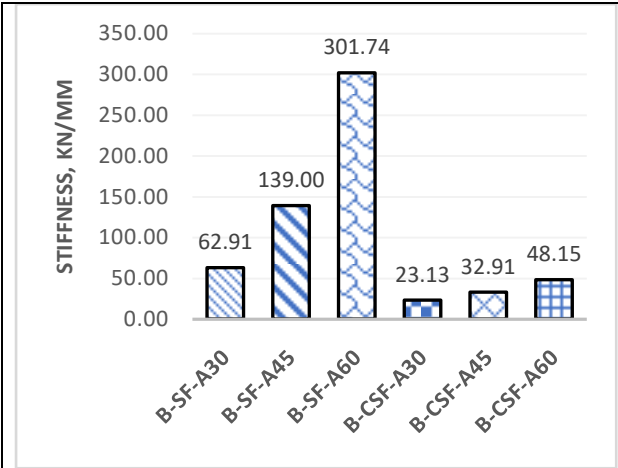
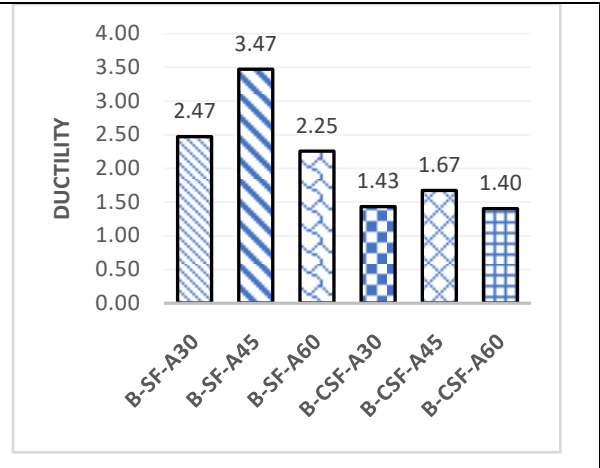
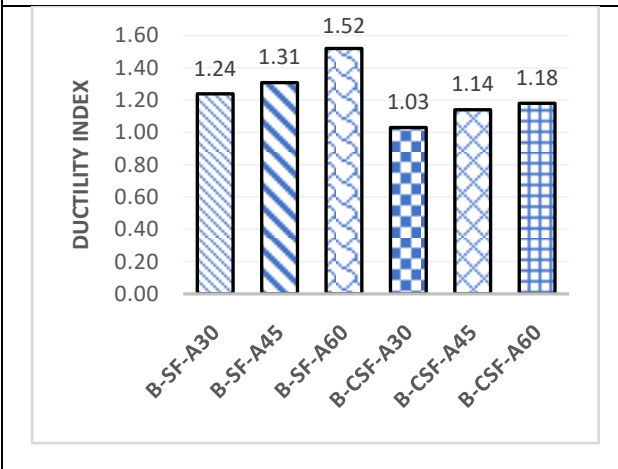
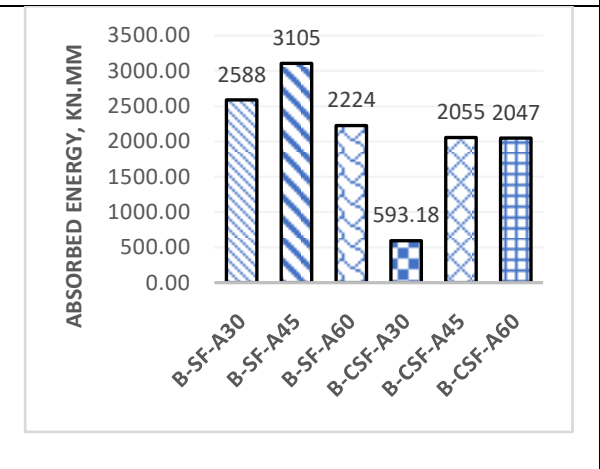
Figure (13): Hardness property comparison of B-SFs and B-CSFs

In term of initial stiffness, (which is mean the ultimate load at which the load- displacement curve behavior converts form linear to nonlinear, over corresponding displacement), for B-SFs specimens, results have shown that B-SF-A60 have the highest value among other specimens, which was about 301.74 KN/mm. B-SF-A60 specimens have better performance than B-SF-A45 and B-SF-A30 specimens by about 2.16 and 4.78 times, respectively. While B-SF-A45 have better performance by about 2.2 times B-SF-A30 specimen. While for B-CSFs specimens, the highest value was noticed for B-CSF-A60, was about 48.15 KN/mm, which was higher than CSF-A30 and CSF-A45 by about 2.08 times and 1.46 times, respectively.

Another criterion which can be used as a performance index to a structural system is the absorbed energy. In simple way, absorbed energy can be calculated using the area under load displacement curve till failure load. For B-SFs specimens, B-SF-A45 specimens has higher absorbed energy value than B-SF-A30 and B-SF-A60 by about 1.2 times and 1.4 times, respectively. while for B-CSFs specimens, the absorbed energy property of the ball jointed composite specimens with angle 45 and 60 were approximately the same, was about 2050 KN.mm, which was higher than B-CSF-A45 value by about 3.46 times.

Ductility can be classified as one of the most important parameters in structures performance evaluations. Tests results have shown different behaviors in term of ductility and ductility index. For SFs specimens, by comparison between the three previously mentioned specimens, B-SF-A45 have the highest ductility and ductility index values among other specimens' values. The ductility of SP-A45 was higher about 1.4 and 1.54 times the ductility of B-SF-A30 and B-SF-A60, respectively. While the ductility index of the three specimens were slightly varied and were approximately the same. While for CSFs specimens, increasing angle of the composite specimens resulted in an enhancement in the ductility index. The highest value was noticed for B-CSF-A60, was about 1.18, which was higher than CSF-A30 and CSF-A45 by about 1.035 times and 1.145 times, respectively. the performance in term of ductility of B-CSF-A45 specimen was the best among other specimens, which was about 1.2 and 1.17, respectively.

Table (4) clarifies Performance summery results of B-SFs and B-CSFs specimens.

 <table><thead><tr><th>Specimen</th><th>Stiffness (KN/MM)</th></tr></thead><tbody><tr><td>B-SF-A30</td><td>62.91</td></tr><tr><td>B-SF-A45</td><td>139.00</td></tr><tr><td>B-SF-A60</td><td>301.74</td></tr><tr><td>B-CSF-A30</td><td>23.13</td></tr><tr><td>B-CSF-A45</td><td>32.91</td></tr><tr><td>B-CSF-A60</td><td>48.15</td></tr></tbody></table>	Specimen	Stiffness (KN/MM)	B-SF-A30	62.91	B-SF-A45	139.00	B-SF-A60	301.74	B-CSF-A30	23.13	B-CSF-A45	32.91	B-CSF-A60	48.15	 <table><thead><tr><th>Specimen</th><th>Ductility</th></tr></thead><tbody><tr><td>B-SF-A30</td><td>2.47</td></tr><tr><td>B-SF-A45</td><td>3.47</td></tr><tr><td>B-SF-A60</td><td>2.25</td></tr><tr><td>B-CSF-A30</td><td>1.43</td></tr><tr><td>B-CSF-A45</td><td>1.67</td></tr><tr><td>B-CSF-A60</td><td>1.40</td></tr></tbody></table>	Specimen	Ductility	B-SF-A30	2.47	B-SF-A45	3.47	B-SF-A60	2.25	B-CSF-A30	1.43	B-CSF-A45	1.67	B-CSF-A60	1.40
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B-SFs and B-CSFs	comparison of B-SFs and B-CSFs
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Table (4): Performance summery results of B-SFs and B-CSFs specimens.

Specimen	B-SF-A30	B-SF-A45	B-SF-A60	B-CSF-A30	B-CSF-A45	B-CSF-A60
Ultimate load, kN	309.40	403.40	515.90	121.81	245.57	335.70
disp. At ultimate load, mm	11.23	9.54	5.07	7.63	12.21	10.02
final load in elastic region	173.00	236.30	422.44	90.20	164.24	194.27
hardness, kN/mm	27.55	42.29	101.76	15.96	20.11	33.50
yield load, kN	249.80	308.40	339.50	117.89	214.87	283.72
yield displacement, mm	4.55	2.75	2.25	5.35	7.30	7.14
stiffness, kN/mm	62.91	139.00	301.74	23.13	32.91	48.15
Ductility	2.47	3.47	2.25	1.43	1.67	1.40
Ductility index	1.24	1.31	1.52	1.03	1.14	1.18
Absorbed energy, kN.mm	2588.00	3105.00	2224.00	593.18	2055.00	2047.00

6. CONCLUSIONS

based on the previously mentioned results and discussions, the following conclusions can be drawn:

- 1- Spaceframe model (conventional and composite) with angle 60 between members had the highest maximum load carrying capacity, and lowest ductility. In the contract to the Spaceframe with angle 30 which had the lowest maximum load can withstand, and highest ductility among other types.
- 2- It was observed that most of non-composite spaceframes specimens of various connection type and height were failed by global buckling of diagonal members, because of nature of specimens supporting. No local buckling was observed in the diagonal tubular members. Also, no failure was observed in the connections.
- 3- Increasing angle between members (or increasing spaceframes height) resulted in higher load can be carried by the system. On the other hand, such increase will not be safe as observed in models with angle 60 duce to reduction in ductility and absorbed energy. balancing between load capacity and ductility may be necessary for structural systems to avoid any unexpected sudden failure. the systems with higher angle between member mean longer diagonal members, which is largely exposed to bucking failure rather than yielding of their material.
- 4- The necking area at the ends of each connected members near joints was the weakest point in the system since most of models have started to yield at this point, and what observed with composite specimens. where a sudden collapse occurred in the top mid chord member.

- 5- Increasing model angle (height) resulted in different methods of load transfer. models' members with angle 30 and 45 have noticed that both main and diagonal type shared the load. In the contract to models with angle 60, where large percentage of the load transmitted to the diagonal parts directly.

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