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EFFECT OF SHEAR CONNECTORS ON BEHAVIOR OF 3D SANDWICH COMPOSITE PANELS SUBJECTED TO FLEXURE

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Abstract
Analysis of 3D Sandwich Composite Panels (3DSCP) subjected to flexure is carried out using Finite Element Analysis (FEA). Analysis Model parts, input data and results are described. For the sake of model's validation, Results of the analysis model is compared to published experimental test results. Effect of shear connectors on behavior of 3DSCP is studied. The analysis program consists of two series of 3DSCP samples with different aspect ratios. Volumetric ratio of shear connectors is increased by two ways. In first series, shear connectors' diameter is increased. While in second series, spacing among shear connectors is reduced. Improvement in 3DSCP composite action is analyzed via strain distribution across the panel's thickness. Other enhancements in 3DSCP behavior like load bearing capacity and deflection are investigated. Recommendation regarding effective method to increase 3DSCP load capacity is proposed.

Keywords: Sandwich Panels; Shear Connectors; Precast Concrete; Slab; Finite Element

1. Introduction

3D Sandwich Composite Panel (3DSCP) consists of two concrete layers reinforced with steel wire meshes. Concrete layers are separated by Expanded Polystyrene Foam layer. Diagonal steel shear connectors are used to achieve a composite action between both concrete layers. Typical 3DSCP cross section is shown in Figure (1).

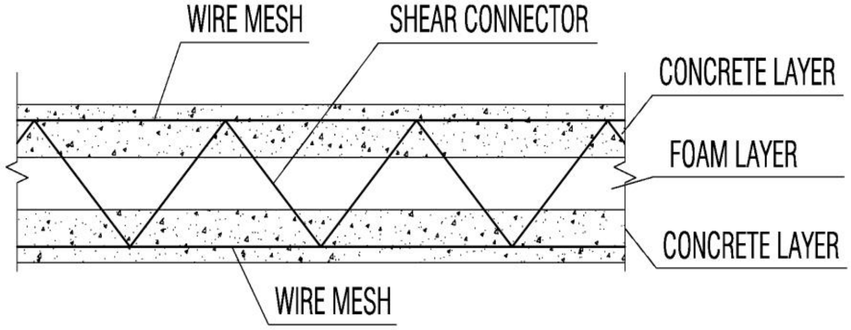

Fig. (1): 3D Sandwich Composite Panel-Cross Section

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3DSCP can be used as wall panel and floor. Prefabricated walls and floors 3DSCP are arranged and assembled using reinforced connections. Shotcrete is then used to form both concrete layers and to fill solid connections [1].

Research related to the 3D panels construction mostly is experimental for panels used as walls or floor slabs [2-6]. Numerical simulation of the specimens is carried on by modeling them using a commercial analysis software and results are compared to experimental work results [2, 5].

Awy from tests on standard traditional 3D panels, Others tested samples with different details like a study used orthogonal shear connectors [4], and a study used corrugated foam to increase interaction with concrete layers [5]. Behavior of samples reinforced with additional rebar in tension concrete layer was investigated in [6].

Samples with double reversed shear connectors to increase panel's capacity were presented in [7]. Theoretical analysis of 3DSCP is presented in limited studies like [8]. Tests on samples using Fiber Reinforced Polymers (FRP) instead of steel are performed in [9,10].

3DSCP exhibits semi composite behavior that lies between two extremes, one extreme is the fully composite behavior like that of a solid concrete panel and the other extreme represents the zero-composite behavior in which each concrete layer behaves independently. Shear connectors determine the degree of the composite action achieved in the panel.

3DSCP is usually produced in standard forms regarding thicknesses and wire mesh reinforcement. Researchers mainly choose tests samples like those commercially produced panels to reflect reality. In this study, Methods to increase load capacity of 3DSCP is investigated. Among several ways to increase load capacity, Effect of shear connectors volumetric ratio is studied. Two options are considered to increase shear connectors volumetric ratio. First option is to increase shear connectors diameter. Second option is to reduce shear connectors spacing. Two series of 3DSCP samples represents each option. Analysis of 3DSCP samples is carried out by Finite Element Analysis (FEA) based software ABAQUS.

2. Analysis Model Validation

2.1 Experimental Work

Benayoune, A.A. Abdul Samad, D.N. Trikha, A.A. Abang Ali, S.H.M. Ellinna [2] studied behavior of 3DSCP slabs under flexure loading experimentally. Testing program included six slabs specimens with different aspect ratios. Samples were tested using four-point test. Results of specimen P12 in this experimental program [2] is used for verification of the analysis model results. Tables (1, 2) describe details of the test specimen and the used materials.

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Table (1): Experiment Specimens Dimensions

ID	t_{c1}	t_f	t_{c2}	t	h	b	h/t	h/b	RFT	Cover	Connector
P12	40	40	40	120	2000	750	16.67	2.67	φ6@100	15	φ6@250

Table (2): Experiment Specimens Materials Details

Concrete Strength (MPa)	Concrete Young Modulus (kN/mm^2)	Splitting Strength (MPa)	Steel Yield Strength (MPa)	Steel Young Modulus (kN/mm^2)
24.15	22.45	2.51	250	215

2.2 Analysis Model

The specimen used in the experimental work [2] were investigated by Finite Element Analysis using ABAQUS. Experimental specimen details were precisely used as the input data of the analysis model.

2.2.1 Model Parts

Concrete layers were modeled using Eight-node brick hexahedral elements C3D8R. The 8-node linear brick elements have reduced integration and hourglass control. Shear locking occurs in first order fully integrated elements (C3D8) that are subjected to bending. Reduced integration element is used to avoid shear locking and to limit processing time. This element has hourglass control, and for further control they are used with reasonably fine meshes.

Truss elements T3D2 are used to model reinforcing steel both mesh and diagonal shear connectors.

2.2.2 Concrete Properties

The concrete damage plasticity model (CDP) is utilized to model concrete. The CDP model uses the yield function of Lubliner et. al. [11] that was based on Drucker-Prager strength hypothesis. These functions were later modified by Lee and Fenves [12] to cover several evolutions of strength against compression and tension loadings.

The CDP model assumes that the material fails due two mechanisms which are cracking in tension and crushing in compression.

2.2.3 Steel Properties

Steel bars used as reinforcement for concrete layers and used as shear connectors are assigned a bilinear stress strain curve characterized by two points representing yielding and failure points. The stress strain relationship describes the elastic range and beyond yielding behavior of steel.

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2.3 Analysis Model Validation

Results of the numerical model were compared to experimental results regarding the load-displacement relationship [13]. Figure (2) shows a plot of the load-displacement curve for the analysis model versus that of experimental specimen P12. The graph shows reasonable agreement as failure load in the numerical model is 26.2 kN while that of the test is 24.5 kN. The difference in the ultimate load is 6.9%.

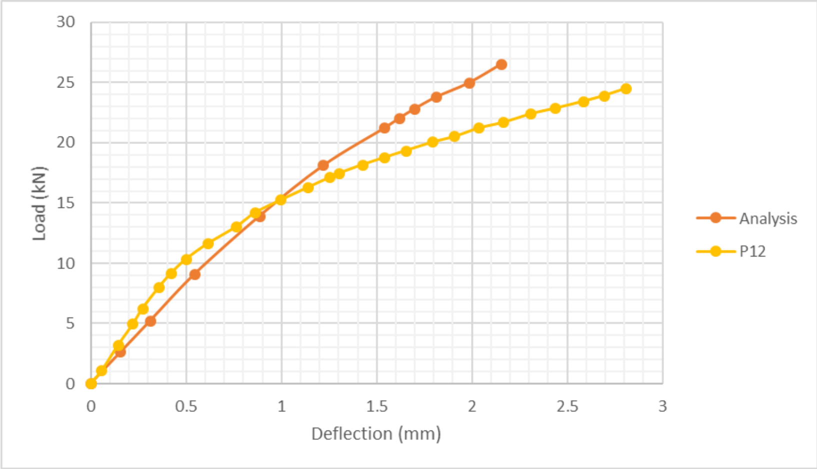

Fig. (2): Analysis Model Vs Experiment Load-Deflection Curve

3. Parametric Study

3.1 Analysis Program

In order to study the effect of shear connectors on behavior of 3DSCP slabs subjected to flexure, two series of samples are proposed. Each series contains samples with different aspect ratios h/t ranging from 14 to 27.33 as shown in Tables (3, 4). The proposed two series have similar details except for shear connectors. In series (I), shear connectors are spaced with double the space of the tension reinforcement mesh. While shear connectors in Series (II) are spaced as half as spaced in series (I).

Series (I) reflects practicality as it represents the common practice case in manufacturers products catalogues and data sheets. The common case is to use shear connectors with same diameter but with double spacing of tensile steel like in the Samples PS-A1 to A5. Analysis of samples PS-B, C and D responds to the query of to what extent the behavior of 3DSCP will improve if shear connectors diameter only is increased.

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Table (3): Samples Series (I) Details

ID	t_{c1}	t_f	t_{c2}	t	h	b	h/t	h/b	RFT	Cover	Connector
PS-A1	50	50	50	150	2100	1000	14.00	2.1	φ6@200	15	φ6@400
PS-A2	50	50	50	150	2500	1000	16.67	2.5	φ6@200	15	φ6@400
PS-A3	50	50	50	150	3100	1000	20.67	3.1	φ6@200	15	φ6@400
PS-A4	50	50	50	150	3500	1000	23.33	3.5	φ6@200	15	φ6@400
ID	t_{c1}	t_f	t_{c2}	t	h	b	h/t	h/b	RFT	Cover	Connector
PS-B1	50	50	50	150	2100	1000	14.00	2.1	φ6@200	15	φ7@400
PS-B2	50	50	50	150	2500	1000	16.67	2.5	φ6@200	15	φ7@400
PS-B3	50	50	50	150	3100	1000	20.67	3.1	φ6@200	15	φ7@400
PS-B4	50	50	50	150	3500	1000	23.33	3.5	φ6@200	15	φ7@400
ID	t_{c1}	t_f	t_{c2}	t	h	b	h/t	h/b	RFT	Cover	Connector
PS-C1	50	50	50	150	2100	1000	14.00	2.1	φ6@200	15	φ8@400
PS-C2	50	50	50	150	2500	1000	16.67	2.5	φ6@200	15	φ8@400
PS-C3	50	50	50	150	3100	1000	20.67	3.1	φ6@200	15	φ8@400
PS-C4	50	50	50	150	3500	1000	23.33	3.5	φ6@200	15	φ8@400
ID	t_{c1}	t_f	t_{c2}	t	h	b	h/t	h/b	RFT	Cover	Connector
PS-D1	50	50	50	150	2100	1000	14.00	2.1	φ6@200	15	φ10@400
PS-D2	50	50	50	150	2500	1000	16.67	2.5	φ6@200	15	φ10@400
PS-D3	50	50	50	150	3100	1000	20.67	3.1	φ6@200	15	φ10@400
PS-D4	50	50	50	150	3500	1000	23.33	3.5	φ6@200	15	φ10@400

Series (II) aims to investigate improvement in 3DSCP behavior with regards to composite action and accordingly amelioration in load capacity and limitation of deflection if spacing among shear connectors drops to half its value in series (I).

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Table (4): Samples Series (II) Details

ID	t_{c1}	t_f	t_{c2}	t	h	b	h/t	h/b	RFT	Cover	Connector
PS-E1	50	50	50	150	2100	1000	14.00	2.1	φ6@200	15	φ6@200
PS-E2	50	50	50	150	2500	1000	16.67	2.5	φ6@200	15	φ6@200
PS-E3	50	50	50	150	3100	1000	20.67	3.1	φ6@200	15	φ6@200
PS-E4	50	50	50	150	3500	1000	23.33	3.5	φ6@200	15	φ6@200

ID	t_{c1}	t_f	t_{c2}	t	h	b	h/t	h/b	RFT	Cover	Connector
PS-F1	50	50	50	150	2100	1000	14.00	2.1	φ6@200	15	φ7@200
PS-F2	50	50	50	150	2500	1000	16.67	2.5	φ6@200	15	φ7@200
PS-F3	50	50	50	150	3100	1000	20.67	3.1	φ6@200	15	φ7@200
PS-F4	50	50	50	150	3500	1000	23.33	3.5	φ6@200	15	φ7@200

ID	t_{c1}	t_f	t_{c2}	t	h	b	h/t	h/b	RFT	Cover	Connector
PS-G1	50	50	50	150	2100	1000	14.00	2.1	φ6@200	15	φ8@200
PS-G2	50	50	50	150	2500	1000	16.67	2.5	φ6@200	15	φ8@200
PS-G3	50	50	50	150	3100	1000	20.67	3.1	φ6@200	15	φ8@200
PS-G4	50	50	50	150	3500	1000	23.33	3.5	φ6@200	15	φ8@200

ID	t_{c1}	t_f	t_{c2}	t	h	b	h/t	h/b	RFT	Cover	Connector
PS-H1	50	50	50	150	2100	1000	14.00	2.1	φ6@200	15	φ10@200
PS-H2	50	50	50	150	2500	1000	16.67	2.5	φ6@200	15	φ10@200
PS-H3	50	50	50	150	3100	1000	20.67	3.1	φ6@200	15	φ10@200
PS-H4	50	50	50	150	3500	1000	23.33	3.5	φ6@200	15	φ10@200

Samples of both series (I, II) were modeled on ABAQUS utilizing the same procedure of the described verified analysis model.

3.2 Analysis Models Description

While elements types are like that used in verification model, materials input data in analysis model are listed in Tables (5, 6).

Table (5): Concrete Material Details

Concrete Strength (MPa)	Concrete Young Modulus (MPa)	Concrete Mass Density (N/mm^3)	Poisson's Ratio
30	26,600	2.5E-09	0.2

Table (6): Steel Material Details

Steel Yield Strength (MPa)	Steel Young Modulus (MPa)	Steel Density (N/mm^3)
420	210,000	7.8E-09

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Meshing of concrete and steel elements is made using partition cells as indicated in Figures (3-a, b).

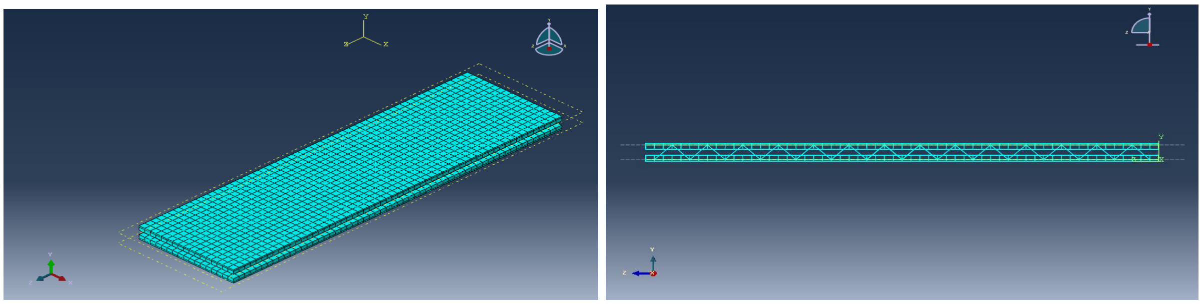


Fig. (3-a, b): Analysis Model-Elements meshing

The slabs were loaded by surface pressure. The bond between steel meshes and the surrounding concrete is guaranteed by embedment constraint feature. Boundaries conditions are hinged at both ends as appear in Figures (4-a, b).

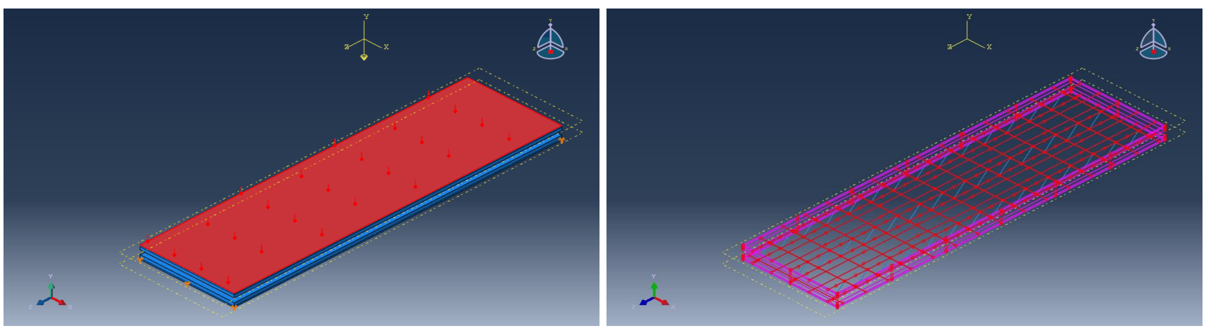
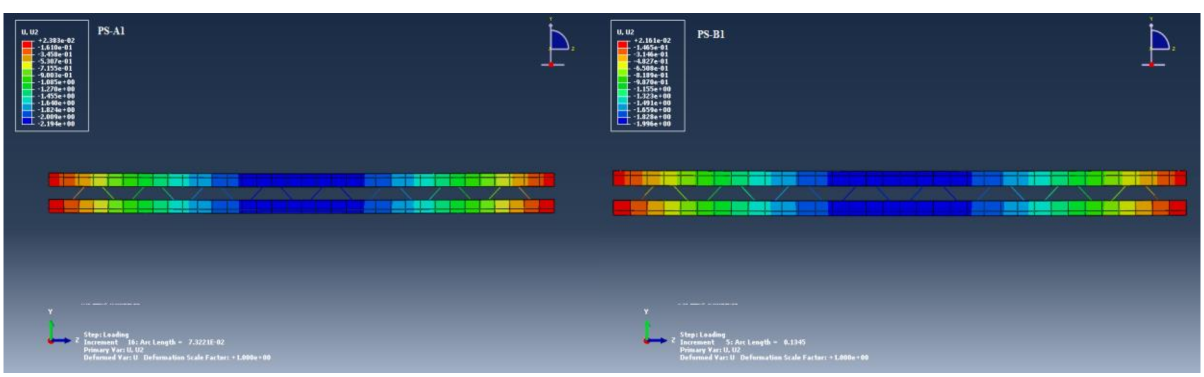


Fig. (4-a, b): Analysis Model-Loading and Steel Embeddment Constraint

3.3 Analysis Models Results

All samples for both samples' series exhibit normal deformed shape for simply supported panels. Figures (5-a, b, c, d) show deflection for sample PS-A1, PB-B1, PB-C1 and PS-D1 for instance.



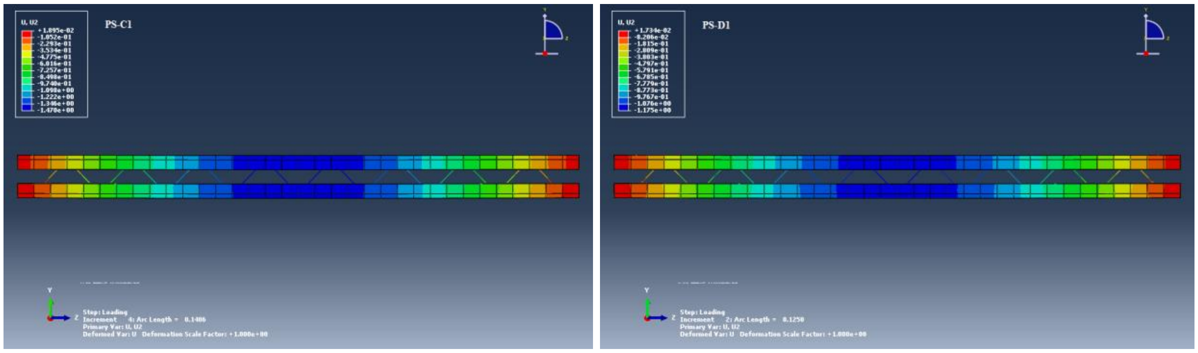


Fig. (5-a, b, c, d): Deflection for Samples PS-A1, PS-B1, PS-C1 and PS-D1

Complete values of ultimate load and the corresponding deflection of series (I) samples are listed in Table (7). Percentage of increase in load capacity and decrease in deflection for each panel were calculated with reference to the datum sample which is the first sample in each aspect ratio h/t.

Table (7): Samples Series (I) Results

Sample ID	Load (kN/m^2)	% Load increase From PS-A1	Deflection (mm)	% Deflection decrease From PS-A1
PS-A1	18.19	Reference	2.20	Reference
PS-B1	18.38	1.04	2.00	9.09
PS-C1	18.92	4.01	1.47	33.18
PS-D1	19.21	5.61	1.18	46.36
Sample ID	Load (kN/m^2)	% Load increase From PS-A2	Deflection (mm)	% Deflection decrease From PS-A2
PS-A2	14.29	Reference	2.88	Reference
PS-B2	14.31	0.08	2.34	18.75
PS-C2	15.22	6.51	2.18	24.31
PS-D2	16.39	14.69	1.85	35.76
Sample ID	Load (kN/m^2)	% Load increase From PS-A3	Deflection (mm)	% Deflection decrease From PS-A3
PS-A3	14.02	Reference	4.76	Reference
PS-B3	15.51	10.63	4.36	8.40
PS-C3	16.46	17.45	4.02	15.55
PS-D3	17.38	23.98	3.55	25.42
Sample ID	Load (kN/m^2)	% Load increase From PS-A4	Deflection (mm)	% Deflection decrease From PS-A4
PS-A4	10.19	Reference	5.53	Reference
PS-B4	11.41	11.92	5.18	6.33
PS-C4	12.06	18.28	4.75	14.10
PS-D4	12.26	20.26	4.10	25.86

Results of series (II) samples are listed against the corresponding samples of series (I) in Table (8). Percentage of amelioration in load capacity and reduction in deflection were calculated.

Table (8): Samples Series (I) Vs Series (II) Results

Sample ID	Load (kN/m^2)	Deflection (mm)	Sample ID	Load (kN/m^2)	Deflection (mm)	% Load increase	% Deflection decrease
PS-E1	22.09	1.80	PS-A1	18.19	2.20	21.44	18.18
PS-F1	22.88	1.49	PS-B1	18.38	2.00	24.49	25.50
PS-G1	24.40	1.36	PS-C1	18.92	1.47	28.96	7.48
PS-H1	24.91	1.11	PS-D1	19.21	1.18	29.65	5.93
Sample ID	Load (kN/m^2)	Deflection (mm)	Sample ID	Load (kN/m^2)	Deflection (mm)	% Load increase	% Deflection decrease
PS-E2	17.09	2.47	PS-A2	14.29	2.88	19.56	14.24
PS-F2	18.01	2.18	PS-B2	14.31	2.34	25.90	6.84
PS-G2	19.13	2.02	PS-C2	15.22	2.18	25.67	7.34
PS-H2	20.33	1.77	PS-D2	16.39	1.85	24.01	4.32
Sample ID	Load (kN/m^2)	Deflection (mm)	Sample ID	Load (kN/m^2)	Deflection (mm)	% Load increase	% Deflection decrease
PS-E3	17.00	4.18	PS-A3	14.02	4.76	21.27	12.18
PS-F3	18.55	3.89	PS-B3	15.51	4.36	19.61	10.78
PS-G3	19.56	3.64	PS-C3	16.46	4.02	18.81	9.45
PS-H3	20.13	3.18	PS-D3	17.38	3.55	15.82	10.42
Sample ID	Load (kN/m^2)	Deflection (mm)	Sample ID	Load (kN/m^2)	Deflection (mm)	% Load increase	% Deflection decrease
PS-E4	12.21	4.87	PS-A4	10.19	5.53	19.81	11.93
PS-F4	14.11	4.85	PS-B4	11.41	5.18	23.71	6.37
PS-G4	14.72	4.56	PS-C4	12.06	4.75	22.12	4.00
PS-H4	15.25	4.03	PS-D4	12.26	4.10	24.42	1.71

3.4 Results Discussion

3.4.1 Analysis of Series (I) Results: Increase of Shear Connectors Diameter

3DSCP exhibits partial composite action between both concrete layers. Degree of composite action achieved in 3DSCP depends mainly on shear transfer capacity of the shear connectors. Increasing shear connector's diameter only while maintaining the same main reinforcement meshes enhances the shear transfer capacity of the 3DSCP and accordingly increases load capacity.

Strain distribution across the panel's thickness reflects the achieved composite action between concrete layers in 3DSCP. Figures (6-a, b, c, d) show strain distribution across panel's thickness for samples PS-A1 to PS-D1. Increasing shear connector diameter from 6mm to 10mm causes more interaction between concrete layers leading to nearly full composite action.

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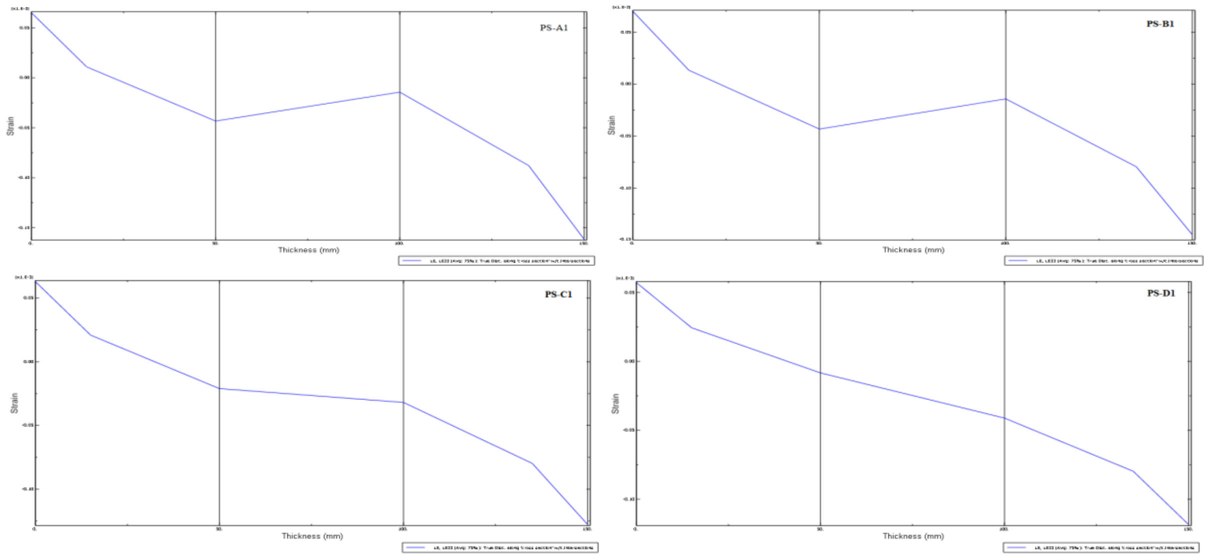

Fig. (6-a, b, c, d): Strain Distribution for Samples PS-A1, PS-B1, PS-C1 and PS-D1

Increasing diameter of the shear connectors from 6mm to 10mm for samples PS-A1 to PS-D1, limited increase in panel's load capacity by 5.61% and significant decrease in deflection by 45% were found. Having small aspect ratio h/t=14, the sample PS-A1 already has high load capacity and small deflection. Any increase in load capacity in sample PS-D1 will be divided by the high load capacity of sample PS-A1 will lead to limited percentage of load increase. Similar interpretation for deflection, any decrease in deflection in sample PS-D1 will be divided by the small deflection value of PS-A1 leading to high decrease percentage.

For the remaining samples with higher aspect ratio h/t, increases in load capacity with maximum 23.98% were calculated and decrease in deflection with maximum 35.76%.

Increasing diameter from 6mm to 10mm will increase volume of shear connectors by 177.78% and will increase total steel volume per square meter of slab by 25.8%. This tremendous increase of shear connectors steel volume will enhance load capacity by only 23.98%.

3.4.2 Analysis of Series (II) Results: Decrease of Shear Connectors spacing

In series (II) shear connectors are arranged at spacing equal to that of tension reinforcement mesh instead of doubling the spacing like in series (I). Figures (7-a, b) show the amelioration took place to strain distribution across 3DSCP's thickness in panel PS-F1.

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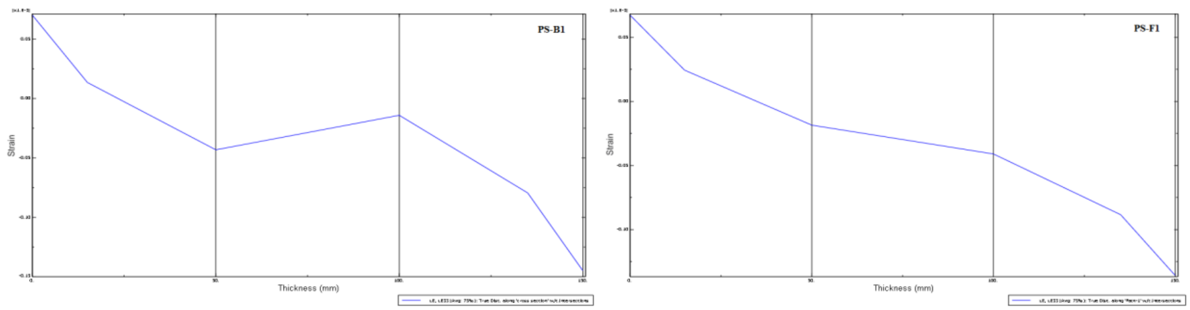

Fig. (7-a, b): Strain Distribution for Samples PS-B1 and PS-F1

Comparing results of series (II) to those of series (I) reveals that maximum percentage of increase in load capacity is 29.65%. This increase could be attributed to that when tension steel and shear connectors have the same spacing, a vertical truss is formed every spacing. This truss consists of top chord of compression bar, Bottom chord of tension bar and diagonals of shear connectors. Forming those trusses every spacing significantly improves the composite action of the 3DSCP.

Reducing the shear connectors spacing to half values of that in series (I) will increase shear connectors volume by 100% and will increase total steel volume per square meter of slab by 14.5%.

There are several ways to increase load capacity for any 3DSCP. Two ways of them are discussed, to increase diameter of shear connectors only or to reduce shear connectors spacing to be like spacing of tension steel. Results discussion illustrates that reducing shear connector spacing is more effective. This option is more effective as it enhances the load capacity with less shear connector's steel volume.

4. Conclusion

Analysis of 3DSCP is performed using Finite Element. Improvement to the behavior of 3DSCP due to increase of shear connectors volumetric ratio is investigated. Reference volumetric ratio is considered in 3DSCP where shear connector's diameter is 6mm and spaced 400mm. Volumetric ratio of shear connectors is increased by two ways. First, shear connectors diameter is increased. Second, spacing among shear connectors is reduced. Based on results of analysis models of several 3DSCP samples, the following conclusions can be drawn:

1. Increasing volumetric ratio of shear connectors only while maintaining the same tensile reinforcement mesh increases load bearing capacity and limits deflection.
2. Increasing volumetric ratio of shear connectors by increasing shear connectors diameter from 6mm to 10mm raises load bearing capacity by maximum value 23.98%. This option increases total steel volume per square meter of slab by 25.8%.
3. Increasing volumetric ratio of shear connectors by reducing spacing among shear connectors to half its value from 400mm to 200mm raises load bearing capacity by

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maximum value 29.65%. This option increases total steel volume per square meter of slab by 14.5%.

4. Reducing spacing among shear connectors is more effective than increasing shear connectors diameter in enhancing load bearing capacity as both alternatives achieve almost similar enhancement ratio, however reducing spacing involves less steel volume.

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