Experimental Investigation on Thin Ferro cement Dome Structures

Wail N Al-Rifaie, Azad Ahmed

Abstract— The paper describes an experimental study for the effect of both skeletal reinforcement and thickness on the strength capacity and behaviour of thin ferrocement dome structures under uniformly distributed load. Four ferrocement domes of 4000 mm covered span were constructed and tested up to ultimate stage. It has been concluded that the construction technique developed in the present investigation reflects the most economic approach, which reduces the nominal cost of such complex structures during construction.

Index Terms— ferrocement, shell, dome, folded plate.

I. INTRODUCTION

The wide spread use of ferrocement in the west began only in the last three decades, gaining publicity with the famous ferrocement domes constructed for the Rome Olympics in 1960.

Das Gupta, N.C., Paramasivan, P., and Lee, S.L. [1], presented in 1980, a study to investigate the feasibility of construction of ferrocement hyperbolic paraboloid shell in the shape of inverted umbrella, having dimensions of 2.44m square in plan and 0.37m vertical rise of the shell center to the edge, and formed by four units of hyperbolic paraboloids which were joined together by horizontal members at the exterior boundary and by inclined members at the interior boundary. The reinforcement consists of two layers having an opening size of 9.5x9.5mm. woven wire mesh of 1.04mm. in diameter spaced by 3mm. mild steel bars of 150mm. centers both ways. The cement, sand and water ratio of 1:1.5:0.5 was used for the mortar. The designed shell thickness was 16mm. The model was tested under uniformly distributed loads up to 4.8 kN/m2 and vertical deflections, strains were measured at 13 positions. An attempt was made to calculate the vertical deflections, strains and membrane forces using the membrane theory. At a uniform transverse loading of 4.8 kN/m2, the maximum deflection of 1.6mm and maximum shear stress of 12.5 kN/m2 were recorded. It was stated that the membrane theory in good agreement with the stresses in the interior face of the shell.

In 1984, Elangouan, S. and Santha Kumar, A.K.[2] presented a study of the behaviour of ferrocement funicular shell roof formed square ground plane of 2.7 x 2.7m with a rise of 0.5m and 30mm. thickness. The shell was supported continuously on edge beams of section 250x250mm. which are supported on eight brick columns of size 250x250mm. at the four corners and at mid-spans of edge beam. The edge beam reinforcement consists of four longitudinal steel bars of 10 mm. diameter and 6mm. diameter mild steel for stirrups centered of 170mm.

The stirrups represent the extension of the skeletal steel reinforcement of the shell which is bent as stirrups to maintain the shape of the shell dimensions. One layer of chicken mesh of 22 gauges was placed on each side of the bars. Cement mortar with 1:3 cement-sand ratios and 0.5 w/c ratios were used during casting. The shell was loaded with sand bags over its entire area to represent a uniformly distributed load. A maximum load of 16 kN was applied.

Kalita, Nambiar, M.K.C., Borthakur, B.C., and Baruah, P. [3], presented in 1986, a work to study the behaviour of various types of ferrocement roofing elements, namely cylindrical shell and segmental shell. The shell element was in the form of a cylindrical shell of 3m width and 3.5m length. The shell element was cast in-situ in angle-iron frame. The skeletal steel was 6mm diameter mild steel placed at spacing of 250mm. in both directions. Two layers of galvanized wire mesh of 0.5mm diameter with hexagonal opening, one on either side of the skeletal reinforcement were used. The shell element was provided with diaphragms at the ends and at the mid-span. It was tested for an effective span of 3.25m and subjected to uniformly distributed load by means of sand bags. The amount of loading to produce maximum allowable deflection was found to be 2.7 kN/m2.

The second type of ferrocement roofing element was “segmental shell”. It was of 0.6m width and 2.5m length and 20mm. thickness. The skeletal reinforcements used were five and ten bars, of 6mm. diameter steel bars in longitudinal and transverse directions respectively. Two layers of galvanized hexagonal chicken wire mesh of 0.5mm diameter covering the skeletal were used. The element was loaded by means of sand bags to simulate the uniformly distributed load. The segmental shell supported an ultimate load of 3.73 kN/m2 when it showed deflection of 1:1987. It was therefore, inferred by the authors that further economy could be achieved by curtailing reinforcement and reducing thickness of the shell element by limiting the deflection to 1:300 for normal loading of 0.7 kN/m2.

In 1989, Singh, P.K.[4], presented an experimental investigation concerning the optimum pre-cast ferrocement roofing shell panel to be used in residential houses and industrial buildings. The shell geometry was selected for “simplicity, cost, structural strength and aesthetic” considerations. The segmental shells were designed for an effective plane area of 3m.x 1m., actual thickness of 25mm. and optimum semi-central angle of 45%. 1:2 cement-sand mortars with 0.45 w/c ratio were selected. The segmental shell was analyzed using stiffness matrix approach by it’s discretization into space grid. This analysis was carried out for the typical four edge support condition of the segmental shell. On the basis of this analysis and the tensile test, the shell skeletal reinforcement was decided to be 9 longitudinal bars of 6mm. diameter at 150mm. on centers and 13 cross bars at 250mm. on centers. Two layers of chicken wire mesh of 0.627mm diameter were selected to cover the skeletal reinforcement. The reinforcement was doubled in the area

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where the tensile stress exceeds the permissible tensile stress value. The test was carried out by placing the model over two parallel girder supports at its ends. Sand bags loading were applied to provide a uniformly distributed load. The failure load was found to be 3.53 kN/m2 at deflection to span ratio of 1:268, limiting it to 1:300. It was stated that high failure load can be realized by supporting the shell panel over edge diaphragms or walls rather than on point supports.

In 1993, Habib, Alamgir, Seng, and Lai Joo [5], made an attempt to develop a simplified method of prefabricated construction for an inverted umbrella type roof for low cost housing, using ferrocement as a construction material. A shell form was generated by site assembly of relatively large number of ferrocement pre-cast triangular flat elements.

The forming of the pre-cast elements was relatively easy as a mortar plastered on wire mesh resting on a flat mould surface, thus, eliminating the necessity for complex formwork and skilled labor. The flat elements were fabricated with 45 degree bents at the edges to allow for easy handling and formation of adequately strong connection at the joints. The assembled shell form, loaded up to both, service and ultimate design load levels, deflected very little and showed no sign of weakness indicating the adequacy of the method of construction employed and the design method used.

In 1994 Dajun Ding[6], presented an application of ferrocement in bridge engineering in China, which has been used for many years in the construction of bridges. Examples of such applications include; two ways curved shallow shells; two way curved shells for formwork, where stones are incorporated in the construction of two-way composite arches; thin-walled slabs for box beams; stiffeners for the bridges in long-span suspension bridges; floating thin-walled cassions for bridge piers; and protective tubes for piles.

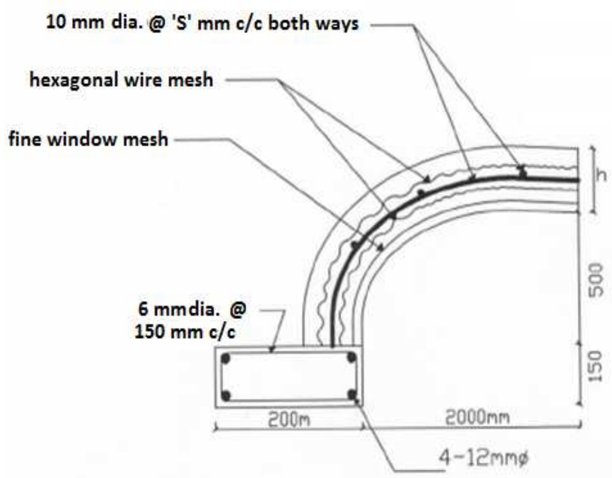
Al-Rifaie, W.N., and Alihmedawi, A.N., in 2000[7], presented an experimental investigation of twelve ferrocement cylindrical shell units. These shell units were cast and tested to failure. The supported span was kept constant while the thickness, mesh layers and the height of the crown were varied. Each shell unit was subjected to a point load applied at the crown. The behaviour has been studied by presenting the load-deflection curves, initial cracking load, ultimate load and the mode of failure and crack pattern at failure.

Al-Rifaie, W.N. and Manasrah, A.A. in 2001[8], presented an experimental investigation of eight segmental ferrocement cylindrical shells of 4000mm. covered span, with 450mm. width and 1000mm. height. The models were reinforced with number of layers of hexagonal chicken wire mesh of 0.7mm diameter. Mild steel bars having 5mm diameter were used as skeletal frame. The mix proportion of cement: sand mortar of 1:2, with w/c ratio of 0.45 was used. Different types of connections for the segmented elements were tested under a knife edge load applied at the crown. It was stated that, ferrocement shell segments were suitable to be used as roofing units.

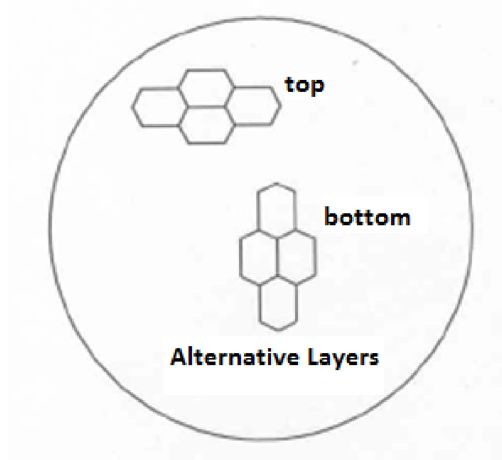
II. EXPERIMENTAL INVESTIGATION

The experimental program consisted of tests on four thin ferrocement domes, which are subjected to uniformly distributed load on their outer-face. All the domes had the same covered span of 4000mm. but differed in skeletal steel reinforcement, and thickness. The domes are designated by a system consisting of three letters refer to the series and the number to the particular dome in the series. The nominal details of the domes are shown in Fig. 1.

The physical properties of the hexagonal mesh, skeletal reinforcement, and the mortar mix are-listed in Table 1. Ordinary galvanized iron hexagonal mesh of 0.7mm. was obtained from the local market. Fine window mesh was used to stand as a formwork. High tensile strength deformed steel bars of 10 mm. diameter was used as skeletal reinforcements in both meridian and circumferential directions. Several strands of wire taken from the hexagonal mesh, straightened and tested under UTM of 100 kN capacity for tension. The stress-strain curve is established and the elastic modulus is determined. With each model fabricated, there were three cubes and three 75x75x300 mm prisms cast and tested to determine the compressive and tensile strengths of the mortar. The average values of the compressive and tensile strengths are given in Table 1. The fineness modulus of sand was 2.343. The sand-cement proportion was 2:1, and the water: cement ratio was 0.45. As the moisture content of the mix varied during construction of the models, the amount of water was adjusted in each mix to obtain a slump of 35 mm.



(a) Geometry and dimensions of the tested domes



(b) Arrangement and orientation of the hexagonal mesh

Fig. 1 Reinforcement details of the tested models

TABLE 1 PROPERTIES OF THE TESTED DOME MODELS

III. FABRICATION OF DOME MODELS

Dome design	Thic. mm	Skeletal steel				Hexagonal wire mesh		
		Dia. mm	Spacing mm	Fy Mpa	Es Mpa	Dia. mm	Fyw Mpa	Ew Mpa
FCD-1	25	10	300	501	189940	0.7	302	67000
FCD-2	25	10	600	501	189940	0.7	302	67000
FCD-3	25	10	1571	501	189940	0.7	302	67000
FCD-4	40	10	300	501	189940	0.7	302	67000
Dome design	Thic. mm	Mortar				Fine window mesh		
		Ec Mpa	$f'c$ Mpa	ft Mpa		Dia. mm	Fyf Mpa	Ef Mpa
FCD-1	25	28970	40.8	4.2		0.08	4.3	788
FCD-2	25	29785	41.2	4.8		0.08	4.3	788
FCD-3	25	27995	40.3	4.1		0.08	4.3	788
FCD-4	40	28884	41.0	4.6		0.08	4.3	788

The models in the present investigation were very thin; therefore extra care was required to control the geometrical dimensions of the models, especially thickness and curvature during construction. The thickness curvature ratio varies from 1/170 to 1/106.25. All models were of 4000mm in the covered span, but differed in thickness and skeletal steel reinforcement spacing which varied from 25 to 40 mm and 300 to 1571.4 mm respectively as shown in Figure 1. It can be noticed that the thickness of the models is too much less compared to their covered spans which forms plane of weakness during lifting such very thin models using an overhead crane to mobile these models on the loading frame without destruction or appearance of early cracks in the dome region prior to testing. Each model was part of a sphere of 4250 mm. radius and 56o 8′24" central angle, the overhead rise of the domes was 500 mm. as shown in Fig. (1). To control all these mentioned institute difficulties during both construction and handling processes, a new technique was used to produce such thin fibrocement domes. In this technique each dome was constructed in three stages.

A. *First Stage: ring beam*

The ring beams were nominally 150mm deep, 200mm wide and 4000mm inner diameter. All the circular ring beams were constructed as separated units. To construct such beams a formwork was prepared which consisted of wooden planks lined with steel plate 1.4 mm thick. To obtain the failure in the dome region, the ring beams were designed to resist extra load (25 kN/m2) than be expected to resist by the thin fibrocement dome regions. The longitudinal hoop steel bars were cut and bent according to the required radius of 2000mm. from a fixed center. The stirrup reinforcement was made with an extra dowel of 400 mm to be extended into the dome region and terminated in a staggered manner to avoid a plane of weakness. Prior to casting, the forms were oiled and fixed. The longitudinal and stirrup reinforcements were tied carefully then lifted and fixed in the formwork. Mortar was mixed well, one circular ring beam with its cubes and prisms were cast per day. A mechanical vibrator was used to density the mortar mix. The sides of the forms were remolded after 24 hours of casting. Subsequently curing of beams, control cubes and prisms were done by wrapping with gunny bags and sprinkling water twice per-day. After seven days the beams were lifted by an electrical overhead crane of 500 ken capacity and fixed on the loading frame.

B. *Second Stage: skeletal steel reinforcement cage*

The skeletal reinforcement was cut and bent-up according to the required curvature in both meridian and circumferential directions using a simple circular tamplet designed to fit the required geometry. The skeletal reinforcements were tied and fixed by binding wires. Then the cage was covered from both, outer and inner faces with two layers of reverse oriented galvanized iron hexagonal wire meshes respectively, and tied well to the skeletal reinforcement using binding wires. An extra layer of fine square galvanized window mesh was used to cover the inner face of the cage and tied well to both the skeletal and hexagonal meshes.

When the domed cage was finished, it was lifted by an overhead crane and well positioned on the ring beam. The skeletal reinforcements were tied to the 400mm stirrup dowels extended from the ring beams to form a monolithic section with the dome region.

IV. SHOTCRETING PROCESS

The domes were fabricated without forms, using a hand manuated shotcreting tool. The inner face of the dome was shotcreted well with liquid mortar mix of 0.60 water/ cement ratio, thus the inner face of the dome was shotcreted uniformly, which formed a thin film to maintain a formwork. Then the shotcreted film was left for 24 hours, and then cured by sprinkling water on the dome inner face. Then the dome part was plastered with a simple Hand-Planer-Tool. The thickness and curvature of the plastered dome had been checked with a simple tamplet designed geometrically to fit the required curvature with nails fixed on the inner side equal to the dome thickness. Thus the models were constructed with uniform thickness and curvature. The domes were then left for 24 hours, and then covered with gunny bags and water was sprayed over twice per-day for 28 days, and then tested.

V. INSTRUMENTATION OF MODELS

The test models were instrumented to obtain the measurement of displacements, angle of twists, vertical deflections and mortar surface strains.

A. *Angle of twist*

Two twist-meters were mounted on the outer face of the dome in both meridional and circumferential directions at node (39) of Figure 3. Another pair of twist-meters was mounted at the juncture of the dome-ring beam in normal and tangential directions, as shown in the figure. Each twist-meter was fabricated by two pieces of plates and on the top plate a level bubble was mounted. The base and top plates were pin-jointed at one end, and at the other end, a micrometer screw was mounted between them, for measurement of their relative movement. The spirit bubbles were initially leveled by the micrometer screws, and as the dome twisted one bubble was more displaced than the other. Micrometer screws were adjusted until they were leveled again. The difference between the initial and current readings of a twist furnished the angle of twist of the section. The difference between the angles of twist of each twist-meter was the total twist over the gauge length.

B. *Vertical deflections*

The vertical deflections for each dome were measured at two locations, at crown and at a point equidistant from crown and the dome edge. At each of the two locations, two small curved nails were glued on the inner face of the domes, from which two ordinary scales were freely suspended through threads fixed to the nails. The readings at each location were taken, using an automatic level after each load increment.

C. Test set-up and procedure

The auxiliary testing frame for the application of transverse uniformly distributed load was designed in the present investigation. The transverse load was applied through concrete prisms of 265N each, as available in the factory. For each stage of loading, the prisms were uniformly distributed over the outer face of the dome and the total load for each stage was calculated. Each dome was tested to failure by applying loads in a series of increments which consisted of increasing the number of concrete prisms to a predetermined level for the transverse load by a definite proportion. It took about 15 minutes, for increasing the loads, after which, they were held constant, while deflections, mortar surface strains, and rotations were measured and cracks marked. The holding period after each increment varied from two to five minutes. Increments of loads were considered as cracking and failure loads were approached. The location of the first cracking and the ultimate failure were noted. Usually 8-15 increments were used prior to failure and the entire test took about 4 hours.

VI. RESULTS

As it was stated earlier that the domes were identically reinforced with two layers of galvanized hexagonal wire meshes. The only difference among the tested models related to skeletal steel reinforcement and thickness. The model FCD-1 was reinforced with 10mm diameter deformed bar at 300mm spacing in both meridional and circumferential directions with uniform thickness of 25mm. While model FCD-2, the spacing of the skeletal reinforcement was increased to 600mm in both directions compared to FCD-1. Consequently the skeletal reinforcement spacing was still increased in FCD-3 in meridional and circumferential directions to 5.28 times the spacing of the skeletal reinforcing in model FCD-4. To investigate the effect of thickness, another model was fabricated designated as FCD-4 with a thickness of 40mm, keeping the reinforcement details same as the model FCD-1. All dome models were tested under uniformly distributed load on their outer face.

The initial cracks of model FCD-1 formed at the juncture between the ring beam and the dome region at a load 60 percent of its ultimate load. The first cracks were evenly distributed along the dome circumferential path. As the load was increased, cracks occurred at the crown of the dome. In the later stages of loading the crown cracks path extended and met the crack arrest pattern propagated in the initial stages at the ring-dome juncture. This dome failed by developing a net of continuous tensile cracks on the interface. Since the cross-section was very thin (i.e. the thickness-curvature ratio << 1), the initial cracks were of significant effects, for their penetration in the thickness through of the dome. Therefore, the pre-cracking stiffness failed down severely and sharply as cracks propagated. The failure was ductile and the maximum limiting deflection was (1/96.59).

The model FCD-2, in which the skeletal steel reinforcing content was reduced by 100 percent, the first cracks formed at a load 73.3 percent of its ultimate strength capacity. The initial cracks were smeared too, but less extensive and fatherly spaced compared to the crack pattern of the tested model FCD-1. Excessive deflection was observed, the limiting deflection was (1/72.65).

As the cracks were far, the first ring-dome juncture cracks linked up to the top crown cracks in later stages of loading. The failure was less ductile and abrupt compared to model FCD-1.

In model FCD-3, where the skeletal was reduced by 405.6 percent over that in model FCD-1, the initial cracks formed at a load 72.72 percent of its ultimate load. The first cracks were smeared and more fatherly spaced compared to model FCD-1. The failure was sudden and less ductile in comparison to model FCD-1. In this model a ring crack path which was continuous at the one-fourth head rise of the dome was formed, through which the dome snapped and failed. The limiting deflection was (1/67). In model FCD-4 thickness was increased 60 percent over that in model FCD-1, while both contained the same amount of skeletal reinforcement. As load was increased initial tensile cracks were formed near the ring-dome juncture which linked at later stages to the crown cracks to form a continuous smeared closed net pattern. More numerous cracks formed which were closely spaced. The failure was ductile and step wise compared to model FCD-1. The initial cracks formed at a load 66.66 percent of its ultimate load. The limiting deflection was (1/137).

The observed nodal displacements at selected nodes through the finite element mesh layout of the tested models were compared with the theoretical displacement fields obtained as given in Tables 2, 3, 4 and 5.

VII. CONCLUSION

A conclusion section is not required. Although a conclusion may review the main points of the paper, do not replicate the abstract as the conclusion. A conclusion might elaborate on the importance of the work or suggest applications and extensions.

TABLE 2: EXPERIMENTAL LOAD-DEFORMATION RELATIONSHIP FOR MODEL FCD-1.

Load	At Crown			At 1/4 Span			Rotation (radian)	
kN/m^2	Δx mm	Δy mm	Δz mm	Δx mm	Δy mm	Δz mm	β_1x 10-2	β_2x 10-2
1	2.08	2.08	2.60	2.03	1.95	2.30	0.139	0.9631
2	3.56	3.56	4.40	2.86	2.37	3.20	0.1401	0.9965
3	4.10	4.26	6.50	2.98	3.22	4.50	0.1436	1.0031
4	4.89	5.01	7.50	3.01	3.86	5.80	0.1520	1.2037
5	6.14	6.00	10.00	3.56	4.39	7.20	0.1598	1.5533
6	7.02	6.98	12.00	4.86	4.98	9.40	0.1602	1.7832
7	7.33	8.62	13.75	5.03	5.32	10.80	0.1667	1.9444
8	8.23	8.77	15.75	5.72	6.03	13.70	0.2011	2.2142
9	9.02	9.98	18.75	6.83	6.88	15.00	0.2267	2.2699
10	11.80	11.43	19.75	7.66	6.98	17.80	0.2286	2.3017
11	13.26	12.94	21.80	8.01	7.62	20.20	0.2333	2.4036
12	14.02	13.54	22.80	8.95	8.25	21.80	0.2901	2.6001
13	14.92	14.46	27.00	9.32	8.94	26.90	0.2978	2.6555
14	15.61	15.02	33.25	9.89	9.05	30.50	0.3201	2.8301
15	16.37	15.96	45.00	10.33	9.65	44.50	0.3780	2.9899

TABLE 3: EXPERIMENTAL LOAD-DEFORMATION RELATIONSHIP FOR MODEL FCD-2.

Load	At Crown			At 1/4 Span			Rotation (radian)	
kN/m^2	Δx mm	Δy mm	Δz mm	Δx mm	Δy mm	Δz mm	β_1x 10-2	β_2x 10-2
1.0	2.67	2.60	2.50	1.964	2.05	3.00	0.1415	0.9384
1.5	3.24	3.43	4.00	2.64	2.98	4.60	0.1426	1.0333
2.0	3.64	3.51	5.50	3.02	3.28	6.20	0.1436	1.1684
2.5	3.82	3.79	7.00	3.69	3.86	8.00	0.1488	1.3014
3.0	3.90	4.01	8.00	4.02	3.96	8.60	0.1527	1.5168

3.5	4.46	4.12	9.50	4.56	4.09	10.00	0.1611	1.6617
4.0	4.90	4.86	11.90	5.23	5.30	12.30	0.1842	1.8013
4.5	5.12	5.03	12.20	5.35	5.66	13.00	0.2067	1.8906
5.0	5.87	5.68	18.00	6.03	5.97	20.00	0.2284	1.9683
5.5	7.65	8.02	26.00	6.83	7.03	22.00	0.2601	2.2637
6.0	9.04	9.68	34.50	8.53	9.00	36.00	0.3000	2.6873
6.5	11.54	12.01	46.50	9.89	9.96	41.50	0.3738	2.9017
7.0	13.33	14.05	51.50	10.33	11.03	52.00	0.4300	3.0210
7.5	18.72	19.05	58.50	11.86	11.90	59.50	0.5617	3.6846

TABLE 4: EXPERIMENTAL LOAD-DEFORMATION RELATIONSHIP FOR MODEL FCD-3.

Load	At Crown			At 1/4 Span			Rotation (radian)	
kN/m 2	Δx mm	Δy mm	Δz mm	Δx mm	Δy mm	Δz mm	β1x 10-2	β2x 10-2
0.5	2.56	2.61	2.60	2.34	2.03	3.0	0.1537	1.0111
1.0	3.04	3.34	4.50	2.86	2.48	4.6	0.1603	1.1690
1.5	3.56	3.62	5.00	3.26	2.98	5.2	0.1723	1.4370
2.0	4.62	4.73	6.00	3.89	3.06	6.4	0.1800	1.6137
2.5	5.09	5.88	7.90	4.23	4.62	7.0	0.1894	1.7634
3.0	5.96	6.04	9.60	5.01	5.86	9.5	0.1907	1.8000
3.5	6.67	7.01	12.50	5.87	7.32	13.0	0.1968	1.8967
4.0	8.87	8.33	13.50	7.06	8.06	13.8	0.2104	2.3200
4.5	9.96	9.27	22.50	8.32	8.99	21.0	0.2617	2.4280
5.0	11.37	10.89	35.00	10.06	11.03	33.0	0.3311	3.1333
5.5	14.62	13.92	50.00	11.86	12.66	48.5	0.4100	3.5067

TABLE 5: EXPERIMENTAL LOAD-DEFORMATION RELATIONSHIP FOR MODEL FCD-4.

Load	At Crown			At 1/4 Span			Rotation (radian)	
kN/m^2	Δx mm	Δy mm	Δz mm	Δx mm	Δy mm	Δz mm	β_1x 10-2	β_2x 10-2
1.5	1.86	1.65	2.40	1.24	1.06	1.80	0.0364	0.2912
3	2.16	1.98	3.40	2.03	1.38	2.90	0.0451	0.3157
4.5	2.34	2.45	4.90	2.41	2.06	3.80	0.0487	0.3414
6	2.61	2.58	5.50	2.56	2.38	4.90	0.0573	0.3998
7.5	2.87	2.69	6.80	2.66	2.50	6.20	0.0807	0.4326
9	2.90	2.76	7.90	2.83	2.68	7.60	0.1067	0.5694
10.5	2.96	2.83	8.80	3.02	2.90	8.50	0.1201	0.6321
12	3.01	2.90	9.40	3.19	3.05	9.20	0.1298	0.6943
13.5	3.56	3.67	12.80	3.32	3.42	12.50	0.1333	0.7064
15	4.87	4.06	14.25	4.06	4.20	13.25	0.1486	1.0031
16.5	6.31	6.24	15.75	5.98	6.01	15.00	0.1968	1.4062
18	7.33	7.62	19.25	6.37	6.87	19.25	0.2436	1.7372
19.5	8.23	8.64	22.00	7.32	8.06	21.25	0.3016	1.9879
21	9.87	9.96	26.00	8.01	8.99	28.00	0.3969	2.0633
22.5	11.83	12.04	31.00	10.98	11.80	32.00	0.4011	2.4369

VIII. CONCLUSIONS

1- This investigation has shown the suitability of the ferrocement material to construct a dome structure.

2- A decrease in skeletal steel reinforcement tends to decrease the initial cracking and ultimate loads.

3- Reduction of skeletal reinforcement is more significant in cracked stage than in the un-cracked stage. As skeletal steel content is reduced, the failure became more severe and sudden.

4- An increase in the dome thickness plays the dominant role in increasing the initial cracking and ultimate loads.

5- The tested models indicated that, whilst cracks appear in the sides and crown of the domes due to an increase of the load beyond the first cracking loads, the remaining parts of the dome were cracked continuously.

6- The construction technique developed in the present investigation reflects the most economic approach, which reduces the nominal cost of such complex structures during construction.

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