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Parametric Study on Development, Testing and Evaluation of Concrete Funicular Shells

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Abstract*— Reinforced concrete shells and folded plates are the structural systems, widely used to cover the small to large area with more aesthetics at minimum cost. Shell structures are stressed skin structures, has unique character in which the curved shape of the shell carries load rather than material strength. Funicular shells are special type of shells for both flooring and roofing systems. The efficiency of funicular shell is mainly due to its shape which formed from both positive (synclastic) and Negative (anticlastic) curvatures. Funicular shells are mainly subject to compression and concrete is most appropriate material for construction. Concrete has the flexibility to form into any shape and size which permits to explore numerous alternatives to arrive efficient funicular shell geometry. In view of above, Present work aimed to investigates funicular shells of different spans with varying rises and thickness. The grid beams are generally used for supporting funicular shells which increase the cost. The present study also proposed to explore the applicability of funicular shells for one-way slab action to avoid the two-way grid beam. The shell spans between 1 m and 3 m with different span to rise ratio between 5 and 40 are investigated analytically. Both material and geometrical nonlinearities were considered in the finite element analysis of concrete shells and the results are comparable with the experimental results. The results indicate that the span to rise ratio between 5 and 12.5 has better performance. The result also infers that the reinforcements are not required at the shell surface. The reduction in shell thickness is more advantageous in lifting handling and placing of precast shells and present investigation showcased about 30% of weight reduction of shell. Further, the funicular shells performed better in one-way action and the results concludes that the funicular shells are favorable for one-way slab action to avoid two-way grid beams.

***Index Terms*—concrete damaged plasticity, FEM, Funicular shell, nonlinear analysis, reinforced concrete.**

I. INTRODUCTION

Funicular shell is a thin doubly-curved shell of a shape which is purely in compression in most of the parts of shell. Theses shells are used both as floors and roofs. The rise of these shells is prepared to minimum and gaps between the adjacent shells are filled with concrete to get a leveled surface. The fillers don't have any major structural role and hence it can be filled with light materials like renewable, non renewable and non degradable industrial and household wastes without compromising its purpose. This novel feature makes the system become cheaper and sustainable buildings. Because of these reasons it is very vital for study of performance of funicular shell by analytical approach.

The development of computer and the computer software made simpler to predict the behavior of the structures. The selection of a mathematical model for simulation is a very important step in any analysis. The FEM involves dividing a structure into a discrete number of elements from which the approximate numerical solution is obtained. With the ease of programming the FEM on personal computers, this approach provides an accurate solution for many structural analysis problems. The accuracy of the results depends on the selection of the suitable elements with the appropriate material characteristic modeling. The Indian code of practice restricts the span to rise ratio between 10 and 20. It is necessary to investigate the efficiency of shell beyond this range. In view of this, FE analysis is readily available with present software packages with efficient material models.

II. LITERATURE REVIEW

Funicular shells are well known structural systems for both roofing and flooring applications. The funicular shells are placed over the square grid beams to explore its efficiency in two way action. Numerous theoretical and experimental investigations were carried out to understand the behavior of prefabricated shells units. Ramaswamy et al (1961) investigated funicular shells and developed a methodology for finding out the co-ordinates of shell for the implementation. Two methods were used for construction. They stretched the fabric across the mould and pouring concrete over it, allowing the fabric to sag and developed the shell. Zacheria et al [1](1971) described about the salient features of design and construction of funicular shell of size about 5 m * 10 m with bricks which is filled between the curved grid beams. Bhattacharya and Ramaswamy (1978) derived a equation to develop a geometrical form of funicular shell surface which is shown in equation (1).

$$Z = -f\left[1-\frac{4x^2}{a^2}\right]\left[1-\frac{4y^2}{b^2}\right] \qquad (1)$$

Where, x and y are in the horizontal plane through the edges and a and b are the span of funicular shell. Here f is shell rise and Z is vertical coordinate height corresponding to the x, y coordinates. Suresh et al (1985) developed a procedure for the computation of accurate values of rise at close intervals and developed ready-to-use tables using both the finite element method and energy method. Ramaswany (1986) briefly outlined about the innovative applications of funicular shells. Small unreinforced precast units of 25 mm thick with a size of 1 m to 1.25 m are cast by the sagging fabric technique. Three numbers of bars were provided in their edge beams. These shells are placed side by side and

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temporarily propped at junctions of adjacent shells and concrete is poured in the gaps between shells to form grid beams. Elangovan et al (1988) analytically investigated the funicular shells using isoparametric elements with five degrees of freedom at each node and clamped boundaries at the support. Approximate expressions for the calculation of bending moment at the edge of the shell, in-plane force at crown and deflection at crown are arrived. Rajasekaran and Sujatha (1992) discussed about the application of Boundary Integral Element Method (BIEM) for obtaining the geometrical configuration of deep funicular shells. Since the governing equation is nonlinear, it is difficult to get closed form solution to obtain configuration. An incremental iterative technique along with BIEM is applied to solve the non linear differential equation. Vafai et al (1997) conducted numerical studies on funicular shells with square bases supported at four edges. Forty-five models are constructed with various parameters including geometry, form, rise and with or without reinforcements. It was observed that the cracks are initiated at and around the centre nearest to the point of the concentrated load and extended to the corners along the diagonals. Weber et al (1984) reported that the deflections of funicular shells are large in the region of the centrally applied force and comparatively small elsewhere. It is also found that measured deflections are greater than those determined analytically by the finite element method. Chaudhari et al (2012) used the general purpose finite element software for concrete modeling using smeared crack model and concrete damage plasticity approach and found that the results are closely agrees with test results. There is lot of incremental works have been carried out in development of funicular shell. There is minimum information on the ultimate strength of funiculars shells with different rises are available. The void also observed in the earlier study on the recommendations on span to rise ratio. The thickness of shell is maintained to 30 mm to protect the reinforcement for shrinkage and temperature consideration. The shells are mostly investigated with classical equation with linear elasticity. Also the shells are designed for two-way action and these shells are placed over grid beams which increase the cost. In view of above, it is necessary to investigate the funicular shells for the various geometrical parameters with including nonlinear behavior of concrete. To improve the economy of the structure with funicular shells, it is necessary to investigate to avoid the construction of grid beam. The present investigation is aimed to overcome the issues on the contest of efficient configuration and implementation of funicular shell roofs and floors. The funicular shell are also examined experimentally and compared with the analytical results.

III. METHODOLOGY FOR FARM GENERATION

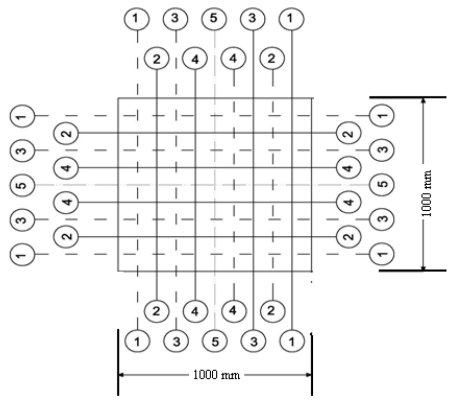
Reinforced concrete shell roofs are widely proposed for covering large clear areas using the minimum or no intermediate supports. The concrete shells are popular in covering the floors of factory buildings, sports area, go downs, power stations, garages, railway platforms and etc. Assortment of shapes employed in modern shell construction, it is difficult to lay down rules. The design and construction of shell roofs is a specialized job needs more knowledge base and skilled peoples. Based on Gauss curvature, the Shell structures are broadly classified in to singly-curved or doubly curved. The singly curved shells are developable in which the gauss curvature is zero. The Doubly curved shells are non-developable and classified in to synclastic (Positive Gauss curvature) or anticlastic (Negative Gauss curvature). The governing equations for the synclastic shells are elliptic and for the anticlastic shells are hyperbolic equations. In addition to the above, there are special shell types in which the positive and negative curvatures are mixed to perform better. Funicular shells are one of the special types of shells widely used for both roofs and floor construction. The geometrical shapes of these shells are not unique nature and the present study adopted the equation from Indian Standard IS-6332 (1984), the code of practice for construction of floors and roofs using precast doubly curved shell units are shown in equation(2).

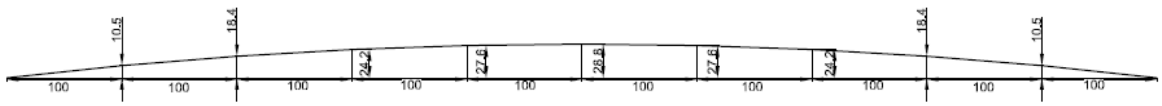
$$Z = \frac{Z_{\max}}{\left(a^2 \times b^2\right)}\left(a^2 - x^2\right)\left(b^2 - y^2\right) \qquad (2)$$

Where, Z is a vertical ordinate at point x, y and $Z_{\max}$ is the maximum central rise between L/10 to L/20 (L= shell size). Here "a" is the half the length of the shell and "b" is half the width of the shell. The surface profile of shell is obtained by substituting the x and y co-ordinates and the resultant rise Z at desired coordinates as shown in figure.1.

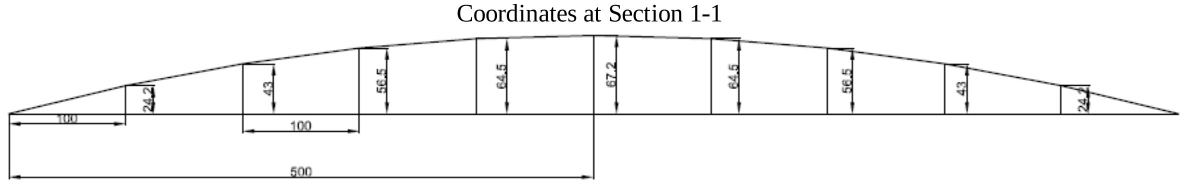
IV. ANALYTICAL SIMULATION

The advent of computers and the computer software simplified the behavioral prediction of any complicated structures. A general purpose finite element software ABAQUS/Standard is used for simulating the behavior of concrete funicular shell. The ABAQUS has its own limitation in accurately modeling the complicated geometry. In view of above, the model was developed in Solid Works and imported to ABAQUS environment. 'Equation driven curve' in solid works is used for creating the surface curve of the shell. The surface profile of shell is obtained by substituting the x and y co-ordinates. For instance, alternatively replacing x and y as zero will get the two perpendicular shell profiles through the apex. These equations are used for the creating of the surface of the shell on the square plan as shown in figure.2a. The ring beam is modeled separately and tied with shell. Tie constraint fuses the two regions together even though the meshes created on the surfaces of the regions may be dissimilar. The surface geometry is imported in to ABAQUS for pre processing. The imported model is meshed with 8 noded linear hexahedron elements as shown in figure.2b.





Coordinates at Section 1-1



Coordinates at Section 3-3

Fig.1 Coordinate development for shell form generation

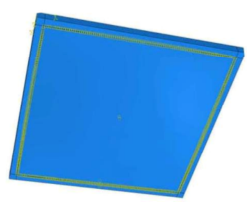


Fig.2a Funicular model in SOLID WORKS

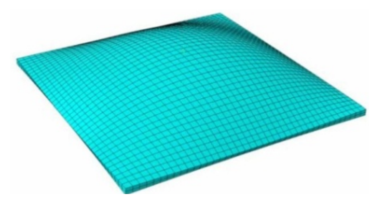


Fig.2b Imported and meshed model in ABAQUS

The shell part of the model is divided into so called brick elements to obtain a proper stress distribution in the 3D analysis. In the present study, C3D8R elements were used to model the concrete shell and the ring beam steel. The present investigation is carried out for twenty eight numbers of funicular shells using static general with nonlinear material and geometrical models. The load is applied over shell and the maximum deflection, maximum principle stress for tension and minimum principle stress for compression are assessed for comparison. The finite element parametric investigation is carried out for span 1000 X 1000mm, 1500 X 1500mm, 2000 X 2000mm and 3000 x 3000mm with various rises as shown in table.1. The study is carried out for both 20 and 30 mm shell thickness with 40mm ring beam depth and width.

Table.1. Parameters for the analytical study

Span (S), mm		Rise (R), mm						
a x b	S/R	40	28.5714	20	12.5	10	8	5
1000 X 1000		25	35	50	80	100	125	200
1500 X 1500		37.5	52.5	75	120	150	187.5	300
2000 X 2000		50	70	100	160	200	250	400
3000 X 3000		75	105	150	240	300	375	600

Concrete is a heterogeneous, non-linear and orthotropic material with relatively high compressive strength and significantly lower tensile strength. Modeling concrete is a difficult task which needs better understanding. Modeling concrete requires the density, elastic property and Poisson's ratio to define its behavior at elastic range. The density of concrete is taken as 24 KN/m³ and the Young's modulus is taken as 25000 N/mm^2 With Poisson ratio of 0.18. In this

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study, Concrete Damaged Plasticity model is used to simulate the concrete material nonlinear behavior. The tensile cracking and a compressive crushing are the main failure mechanisms of the concrete in the concrete damaged plasticity model. The model considers the degradation of the elastic stiffness induced by plastic straining both in tension and compression. The evolution of the yield surface is controlled by two hardening variables linked to failure mechanisms under compression and tension loading. In compression, the stress-strain curve for concrete is linearly elastic up to about 30 percent of the maximum compressive strength. Above this point, the stress increases gradually and reaches the maximum compressive strength σ_{cu}. The curve descends into a softening region, and eventually crushing failure occurs at an ultimate strain ε_{cu}. Several models were available for concrete compression behavior. The numerical expression developed by Hognestad (lakshmikandhan et al (2012)) is used to model. The plasticity of model also considers 34 degree for dilation angle and 1.16 for the ratio between equi-biaxial compressive stresses to uni-axial compressive stress. The ratio of the second stress invariant on the tensile meridian to that on the compressive meridian, K is taken as 0.667. The Viscosity Parameter, μ, used for the visco-plastic regularization of the concrete constitutive equations is taken as 0. The tension stiffening option is used to define the concrete's post failure behavior. The tension behavior of concrete is included as in terms of yield stress and cracking strain as given in table.2.

Table.2. Material data for damaged plasticity model for concrete and plasticity model for steel

Compression Model		Tension behaviour		Concrete Tension Damage		Steel Plasticity	
Stress	Plastic Strain	Yield Stress	Plastic Strain	Damage Parameter	Plastic Strain	Stress	Plastic Strain
7.52	0	2.5	0	0	0	332	0
18	0.0008	0	0.0031	0.9	0.01	352	0.0001
25	0.0015					373	0.0003
27	0.0018					394	0.001
28	0.0023					435	0.002
20	0.003			Note: For damage parameter, 0 represents no damage and 0.9 represents about to fully damaged		435	0.003
						440	0.005
						435	0.01
						400	0.03
						370	0.06

The Static Riks method of structural analysis is carried out to overcome the issues related to instability point which normally may occur in the static general analysis. The history output request is made at the appropriate nodes. The deflection, stress and strains are observed from the analysis. The ultimate load of shell is calculated by multiplying the applied load with the maximum LPF from the history output.

V. EXPERIMENTAL INVESTIGATION

Due to the complicated grid floors construction, the present study is aimed to develop a simple systematic arrangement for the development of floor and/or roof system. The funicular shell is proposed to arrange over the beams with predominant one way application. In view of above, the funicular shells are planned to test for assessing their one way capacity. Two shells were cast with 20 mm and 30mm shell thickness with the following procedures. The central rise (apex) of funicular shell is maintained to 80mm. The mould is proposed and prepared with a wooden flank, nails and cement mortar. The base for the shell is formed with wooden flanks and the nails were fixed to a required height to represent the various coordinates. Mortar of mix proportion 1:4 is prepared to fill the nail work as shown in figure.3b and then the top surface is finished and smoothened with cement paste and wax mixed oil as given in figure 3c and 3d. Several trail mixes are trailed and proposed with concrete mix proportion 1: 1.5: 1.5: 1.5 % by weight (cement: sand: baby gravel aggregate passing through 7 mm: Super plasticizer by cementecious weight) with water cement ratio of 0.5. About 25% of cement is replaced with fly ash to reduce the shell cost as well as the shrinkage crack. The initial analytical studies clearly showed that the tension will not arrive in the shell. In view of above, reinforcement was eliminated from the shell surface. The minimum reinforcement is provided in ring beam to take care of the tension. One number of 6 mm diameter steel bar used in this investigation. The ring beam was filled with concrete and it extended to the crown. The thickness of shell is properly maintained using cover blocks. The sequential steps involved in the casting of shell are shown in figure.4. Two shells with 30 mm and 20 mm thickness were cast. After 1 hour, the shell surface was finished with trowel to avoid any possible shrinkage cracks on the surface. Demoulding was carefully done after 24 hours and placed in to water for curing. Proposed method of shell casting is simple and works well expects the few problems during demoulding. The wooden reapers bulge due to the wet condition and sticks with shell which needs extreme care during demoulding. The shells were weighed on 28th day and found that the 30 mm shell is weighing about 108 kg and the 20 mm thick shell is weighing 79 kg. The percentage of weight reduction is about 30 % between the shells.

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Fig.3 Different stages of funicular shell mould preparation

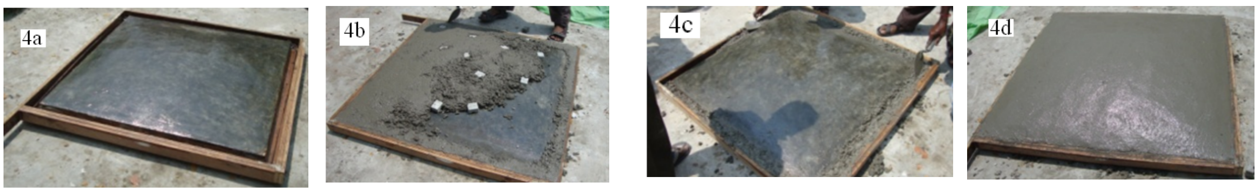

Fig.4 Different stages of casting of funicular shell

The shells are shifted to test floor and kept over the supports. The construction of grid floor increases the cost and the duration. Considering this disadvantage, the present investigation proposed to investigate the strength in one direction so that the shells can be placed over the precast / insitu cast beams. The test is carried out like simple beam testing method. The strain gauges of 60 mm length are fixed over the crown, over the leading diagonals and under the center span of ring beam as shown in figure 5a. The dial gauges were placed under the center span of ring beam and under the crown. The shells were tested by placing the sand bags weighing of 30 kg weight over the top surface. The uniformly distributed condition is nearly maintained by placing the sand bags in a uniform sequential order as shown in figure 5b. The deflection and the strain readings are recorded in the data logger at a regular interval.

Fig. 5a typical view of pasted strain gauges

Fig.5b Typical view shell testing with sand load

VI. RESULTS AND DISCUSSION

Nonlinear finite element analysis is conducted with 30 mm thick shell with square grid patterns. The parametric study was carried out with different spans, different rises and different span to depth ratio of shell. The shell analysis is carried out with both material and geometrical nonlinear. The maximum deflection is observed under the point load as shown in figure.6.

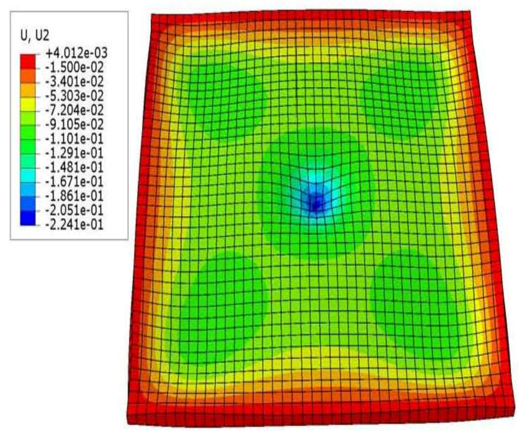

Fig 6: Deflection contour of funicular shell of span 1m with span to rise ratio of 20

The compressive stress distribution is obtained from the minimum principle stress plot as shown in figure.7a and the tensile stress distribution is obtained from the maximum principle stresses plots as shown in 7b. The maximum deflection for different spans are observed from the analysis and the consolidated results for shell with 2.5 kN load is presented in figure 8a and the results for shells with 5 kN load is presented in 8b.

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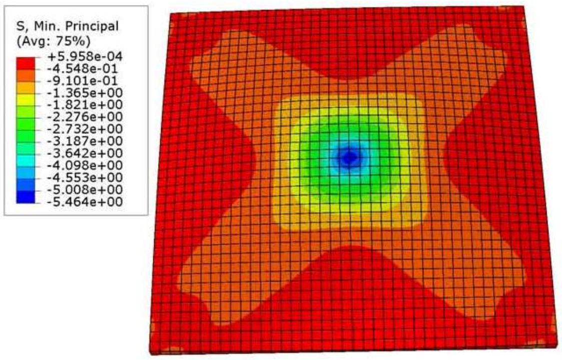
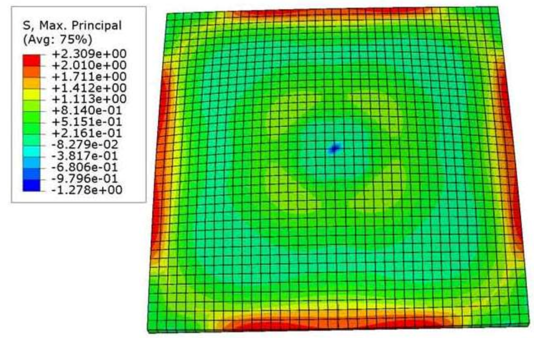


7a. Compressive stress contour



7b. Tensile stress contour

Fig. 7 Typical stress contour for funicular shell of 1m span

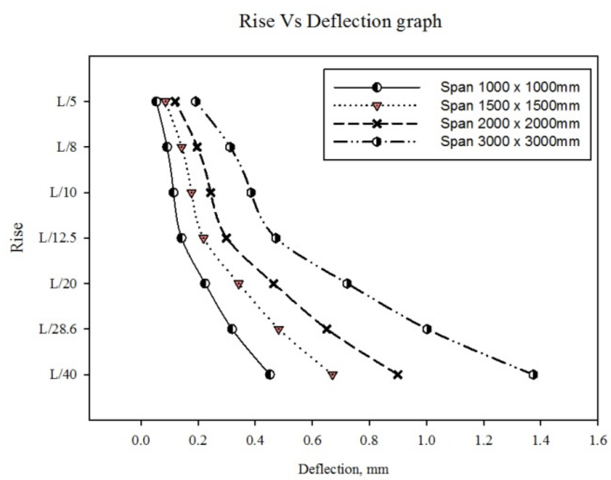


Fig. 8a Comparison of maximum deflection for different spans subject to 2.5KN load

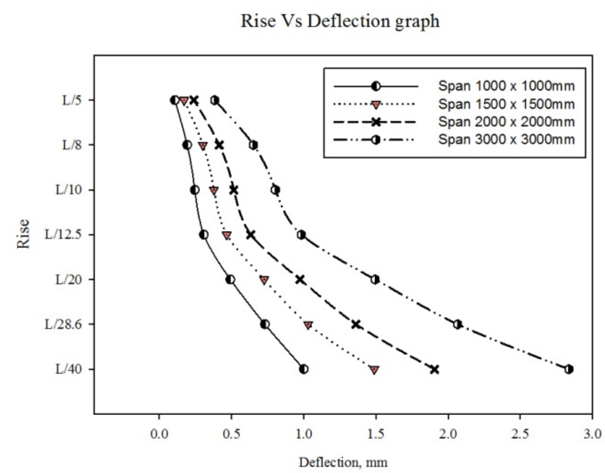


Fig. 8b Comparison of maximum deflection for different spans of shell with 5 kN Load

From the above graphs for different spans, it is observed that the deflection varies with rise in a similar pattern. Also for increase in load, the deflection increases in the same trend of earlier load level. This indicates that the shell is not cracked and all the concrete elements are intact. The span to rise ratio has considerable influences the maximum deflection of shell. The increase in shell rise is result in the reduction of maximum deflection. The influence of shell rise increase on reduction in the deflection is more in higher spans. The reduction in deflection means the increase in the stiffness of funicular shell. Irrespective of shell span, the rise between L/5 and L/12.5 has more influence for deflection reduction. The influence of different parameters on distribution of compressive stress is examined with the contour plots of minimum principle stress. The consolidated results of maximum compressive stress in the funicular shell are presented in figure.9a and figure.9b.

From the above figures, it is observed that the span to rise ratio has considerable influence in the minimum principle stress. Irrespective of shell span, the compressive stress is very low when the rise is L/5. The rise above L/20 is result in decrease of compressive stress. The rise below L/20 has meagre change in the compressive stress. The maximum compressive stress in all the shell cases are below the compressive strength of concrete which infers that the shell can carry higher load. The maximum tensile stress for the different spans are observed from the maximum principle stress at the over stressed points are compared and presented in figure 10a and figure 10b. The maximum tensile stress is observed in the ring beam. The other parts of shell portions are no or negligible tension. It is observed that the span to rise ratio has huge influence in the minimum principle stress. The shell rise has more influence in higher spans then the lower spans. The rise between L/5 and L/12.5 has minimum tensile stress than the other shell rises. For both the load cases, the tension in the shells with span to rise ratio is greater than 28.6 are seems to be critical. The maximum tensile stress observed at the shell ring beam is slightly higher side than the concrete strength in tension which requires minimum reinforcement to take care of the tensile stress.

The analytical study clearly indicates that the span to rise ratio of funicular shell between 5 and 20 has better performance and the above that has inferior performance. The resultant tension and compression on both ring beam and shell are very low compared to material concrete strength. The thickness of the shell can be reduced to optimize the shell weight. Moreover the shell surface does not have any tensile stress which means that the reinforcement can be avoided.

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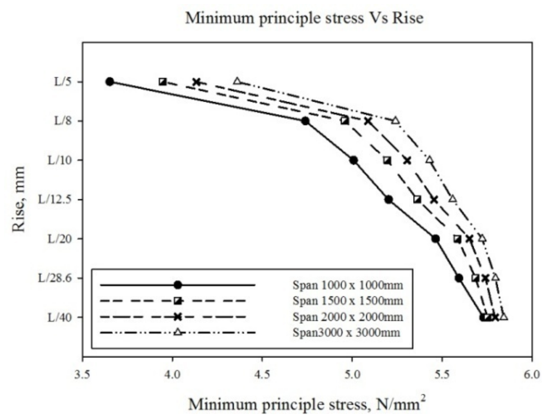

Fig. 9a Comparison of maximum compressive stress for different spans with 2.5 kN load

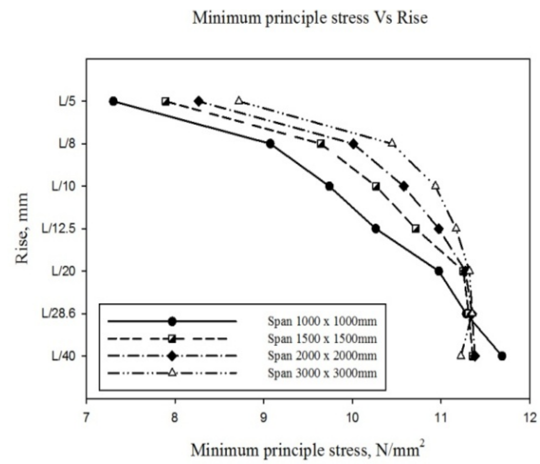

Fig. 9b Comparison of maximum compressive stress for different spans subject to 5 kN load

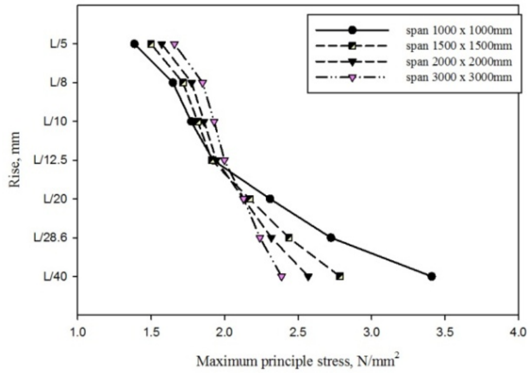

Fig. 10a Comparison of maximum tensile stress for different spans subject to 2.5 kN load

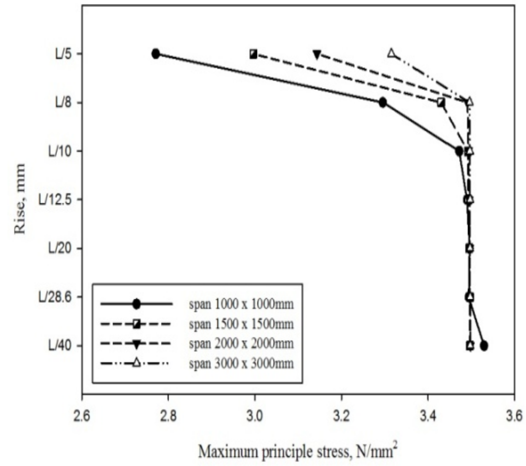

Fig. 10b Comparison of maximum tensile stress for different spans subject to 5 kN load

The minimum thicknesses stated in the references are between 25 to 30 mm which includes the cover for steel. The exclusion of steel from the shell surface allows us to reduce the thickness and dead weight. The chance of straining due to temperature and shrinkage is lesser for the lower thickness compared to mass concrete which also have the positive sense for reducing shell thickness. The shells with 30 mm thickness and shell with reduced 20 mm thickness are tested. The strain observed over the top surface of 20 mm thick shell is shown in figure.11. The maximum compression in the 20 mm thick shell is observed over the apex and the maximum tensile strain is observed at the mid span of ring beam are shown in figure 12. The deflection observed in the 20mm shell is shown in figure.13. The maximum deflection is observed at the centre of span. The shell behaved like the simply supported beam. Along the leading diagonal of 20mm thick shell has the maximum compressive strain about 10 micro strain. The maximum compressive strain about 1500 micro strain is observed over the apex of funicular shell. The test and analytical results clearly indicate and confirms the compressive nature of shell. The shell examined for the one-way span application is not affects the compressive nature of funicular shell. The tensile strains are observed only at the ring beam. The tensile strain recorded at the centre span of funicular shell is presented in figure.14.

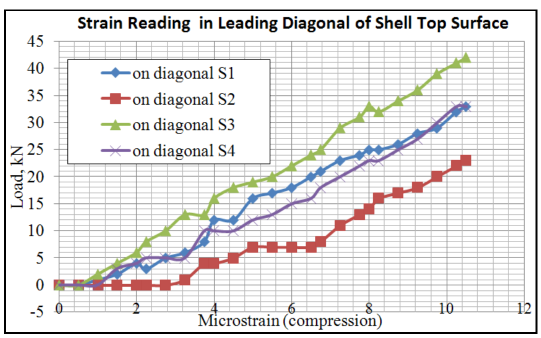

Fig.11 Comparison of load and strain at leading diagonal top surface of 20mm thick shell

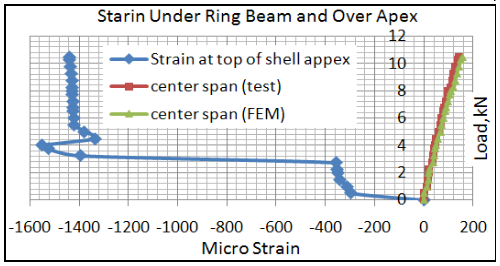

Fig.12 Comparison of maximum compression and maximum tension

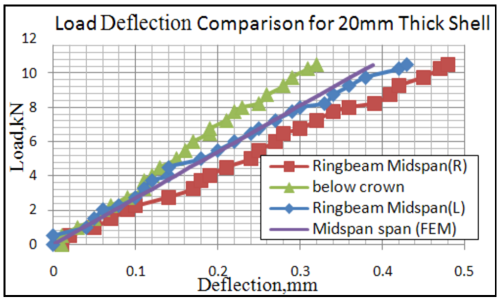

Fig.13 Deflection Comparison of 20mm thick shell

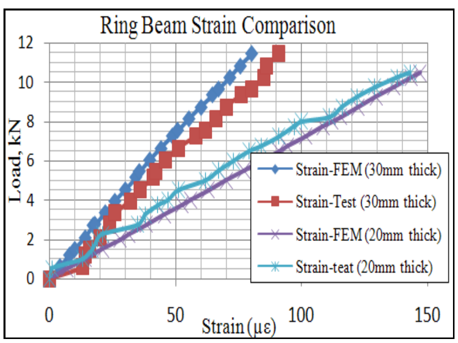

Fig.14 comparison of tensile strain at centre span of ring beam

Both the 30 mm and 20 mm thick funicular shells are tested with uniformly distributed load about 12 kN / m^2. Both the funicular shells, the maximum compressive stress is recorded over the apex. The maximum compressive strain observed is about 1500 micro strain in 20 mm shell. The maximum tensile strain observed in 20mm thick funicular shell is about 140 micro strain and the maximum tensile strain observed in 30mm thick shell is about 80 micro strains. The shells are performed uniformly in a comparable pattern. The nonlinear finite element analysis results are comparable with the test results. Both the shells are not shown any micro or macro cracks on the ring beam or on the shell surface. From the results, it is understood that the 20 mm thick shell it selves is not fully utilized and it infers that the thickness can be reduced further to get the minimum weight. The weight difference between the 30mm and 20mm thick funicular shell is about 30percent and it can be reduced further which eliminates the lifting, placing and other handling problems.

VII. CONCLUSION

The finite element nonlinear parametric analyses of funicular shell with different spans for the varying span to rise ratio are showed noticeable knowledge addition for the selection of span to rise ratio. With the experimental and the analytical investigations, the following conclusions were arrived.

1. The material model used for the nonlinear finite element analysis is performed well and the results are closely comparable with the experimental results.
2. The parametric study carried out on the funicular shells clearly indicates that the span to rise ratio between 5 and 12.5 has better performance. The span to rise above 12.5 performs inferior and the span to depth ratio greater than 20 result in very poor performance. This infers that the upper value of span to rise ratio kept in the codes for design of shell proved its worthiness.
3. The reduction in the span to rise ratio showed reduction in deflection, reduction in maximum compression and the reduction in maximum edge beam tension with improved stiffness.
4. The funicular shells are exhibited good performance in one-way slab action. The testing also proved its worthiness of two-way funicular shells for the one roof or floor constriction. This infers that the funicular shells are well suitable for one-way slab arrangements. This avoids and replaces the construction of cast insitu grid beams with precast/ cast insitu beams.
5. The 20 mm shell performed well and the maximum strains observed from the investigations are comparably lower with material strength. The results from the applied load and strain infer that the shell thickness can be reduced further.
6. Exclusion of the reinforcement avoids the requirement of cover concrete for corrosion and allows constructing shell with minimum shell thickness.
7. The weight difference between the 30mm and 20mm thick funicular shell is about 30percent and it can be reduced further for the thickness reduction which eliminates the lifting, placing and other handling problems.

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