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Studies On Economical Configuration Of RCC And Prestressed Shell Roofs By Using ANSYS

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ABSTRACT

This paper presents the nonlinear Finite Element Analysis (FEA) that has been carried out to simulate the behaviour of failure modes of Reinforced Concrete (RC) beams strengthened in flexure and shear by Fibre Reinforced Polymer (FRP) laminates. Four beams were modeled in FEM software using ANSYS. In those four beams, two beams were control beams without FRP and other two beams were Carbon Fibre Reinforced Polymer (CFRP) strengthened beams. A quarter of the full beam was used for modeling by taking advantage of the symmetry of the beam, loading and boundary conditions. From the analysis the load deflection relationships until failure, and crack patterns were obtained and compared with the experimental results available in the Literatures.The load deflection plots obtained from numerical studies show good agreement with the experimental plots. There was a difference in behaviour between the RC beams strengthened with and without CFRP layers. The crack patterns obtained in FEA in the beams were also presented. The use of computer software to model these elements is much faster, and extremely cost-effective. Therefore, modeling of experimental beams can be adoptable in ANSYS. Validation of experimental results can also be done using ANSYS.

Keywords:Studies On Economical Configuration Of RCC And Prestressed Shell Roofs By Using ANSYS

1. INTRODUCTION

Natural structures offer unique examples of high structuralstiffness and performance while preserving the aestheticbeauty of their shapes. There exist interesting examples of manmade structural forms which have been developed bymimicking natural structures. The result is a 3-D structurewhich, by itself, offers great stiffness while diminishing itsweight. One particularly attractive approach to the above is theuse of a "minimal" (term valid both from the mathematics asfrom the architectural standpoints) structure under the"tensegrity" concept, which describes a structure stabilized bymeans of tensors, which allows to associate the functionalrelationship of the structure as a closed circuit of structuralelements. These structures possess a minimum weight with thecapability of sustaining important mechanical loads under arational scheme and efficiency.The present study deals with a spatial structure constituted bya special arrangement of elements that provide substantialstiffness to the structure. The original structure was proposed.The outstanding features of this buildingcorresponds to a original structure that combines a catenoidmembrane with a tensegrity-based structure, being the windeffect of particular interest due to its membrane nature.

2. MATERIALS AND METHODS

Fourrelevant elements of the structure can be observed: pinnedtubular sections; rings with flexure stiffness; supporting cablesof the membrane and cables applied inside the hexagons. Thefinal project of the structure includes five concrete columnsand a bearing wall.

2.1 Prototype Was Built In Alfalfares Park In Queretaro City

Correspond to link 3-D elements with tensile or compressionbearing capacity. The cables are considered as linked elementstoo, with the capability of accepting initial tensile strains. Themembrane is modeled using shell elements of 8 nodes with aminimum stiffness normal to its plane in order to stabilize itsbehavior under loading. Due to its thickness and geometry, themembrane tends to get unstable even with small loads. This leads to a non linear analysis of the membrane.Therefore, the membrane is only able to sustain tensilestresses, collapsing under compression. Similarly, the cablesonly sustain tensile stresses. (Figure.1)

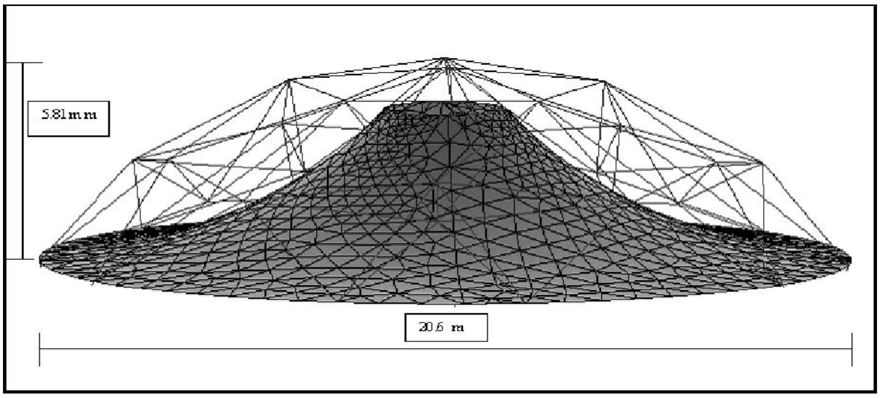


Figure1. Discrete Model Of The Roof Cover

2.2Optimization Model

An optimization model is required to establish an optimumequilibrium state in the system, due to the initial prestresstension at the cables that develop stresses on the entire system.These change when the loads are applied, thus generating newpre-stress tensile stresses and a new equilibrium state. If prestresslosses are involved in the membrane and the cables, thefinal evaluation of such structures becomes complex.(Figure.2)

Solutionproposed herein is to evaluate the initial and final pre-stresstensile stresses in the cables before and after the stressdistribution of the entire system, so these values comply withthe range of pre-established values. Also, the maximum stressof the membrane has to lie within the pre-established range.All this needs to be achieved, under the restriction ofminimum weight of the structure.Representing the structure weight by the objective function Z, and the design variables beforementioned DEFC and DEFM, corresponding to the initialstrains of group of cables 1 and 2 respectively, optimizationproblem can be enunciated.

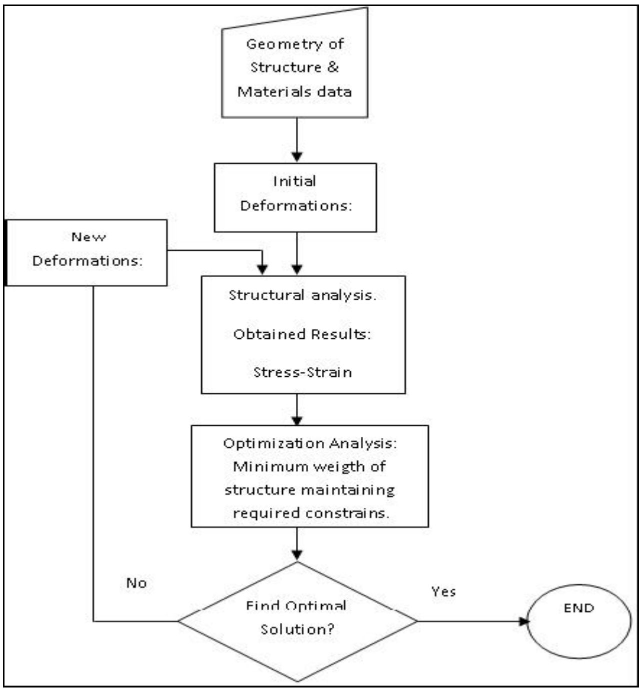


Figure. 2. Block-Scheme Of Coupled Structural-Optimization Analysis

State equations refer to the code constraints on stresses anddisplacements of the structure. The final tensile constrains inthe cables are due to the design conditions (LRFD Normsabove). Also, the membrane stress ranges have beenconstrained in such a way that generates minimum stress andcreates a stable geometrical shape, without weak zones wherethe material could wrinkle. On the other hand, a maximumstress that avoids material rupture is needed. According to thescheme shown in Figure 4, the analysis-design process iscoupled with the non-linear optimization subroutine.

Inaddition to the considered aspects aforementioned, the studycouples a design phase based on limit states. Every cycle ofoptimum analysis-design gives a group of design solutionswhich could belong to the space of feasible solutions.

Theprocess repeats with new variable design values given byoptimization procedure or even by the user, until a localminimum of the objective function is reached and the userconsiders it to be an appropriate solution. The analysis wascarry out by taking the next load combinations:

i). (Dead load + Augmented presstress)(1.0)
ii). (Dead load + Live load + Presstress load (relaxed))(1.4)
iii). (Dead load + Prestress in cables (relaxed) + Temperature)(1.0)
iv). (Dead load + Wind + Presstress (service))(1.1)
v). (Dead load + Presstress in cables (relaxed) + rain)(1.1)

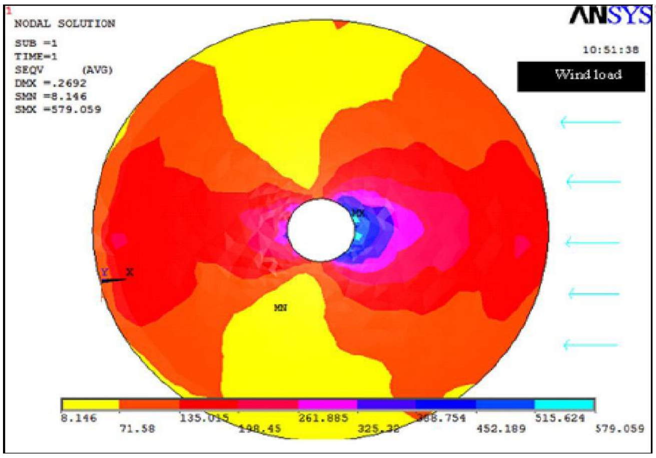
Prestress losses in the steel cables range from 1 to 2 % after1000 hours, according to the type of cable, also it isrecommended membrane cables be subjected to a tensile prestressof 4 to 10% of the cable rupture stress, if it is to beoverstressed. On the other hand, themembrane operating tensions range from 1 to 4 kN/m forpolyester membranes with vinyl coatings. This implies that,for a thickness of 0.85 mm a maximum operating stress of 50kg/cm2 (5.0 MPa) is applicable. Also, madein situ experimental studies of a membrane, by measuringtensile relaxation stresses of about 30% of that which wasinitially applied. In addition to the stress relaxation on thefabric shell, its degradation due to ultraviolet rays alsobecomes a very important factor. Due to all these factors, thepresent study has taken into account the following pre-stressvalues:

- Membrane maximum operating stress (overstressed) <2.5 MPa + 30%
- Membrane maximum operating stress (Relaxed) < 2.5 MPa
- Operating tension stress of a cable of ½ inch < 2.4 Ton

The coupled procedure takes into account the initial pre-stressstrains in two groups of cables: those supporting themembrane (DEFM), and those that carry tensile stress to theinner hexagons (DEFC). Then, it is possible to perform theanalysis of the initial strains, including the pre-stress valuesmentioned earlier. These values change in every cycle untilequilibrium is reached, in which the stresses in the differentstructural members, as well as the tensile stresses in cables andmembrane are at the desired level.

2.3.Results

The proposed procedure of analysis-design of this singularroof cover shows that the structure has a natural stiffness andstability, being the displacements under loading very smallwith respect to Mexican Regulations. Also, the stresses in allthe structural elements have ample security factors that discardthe possibility of its failure under loads. The grid structurewithout tensors is, by itself, an unstable structure, and due tothe presence of cable tensors, becomes stable. Coupling of thefabric shell and grid steel bars makes the entire structurestiffer, creating a pre-stressed closed circuit. The membranemaximum stresses occur in the region close to the top ring.(Figure.3)



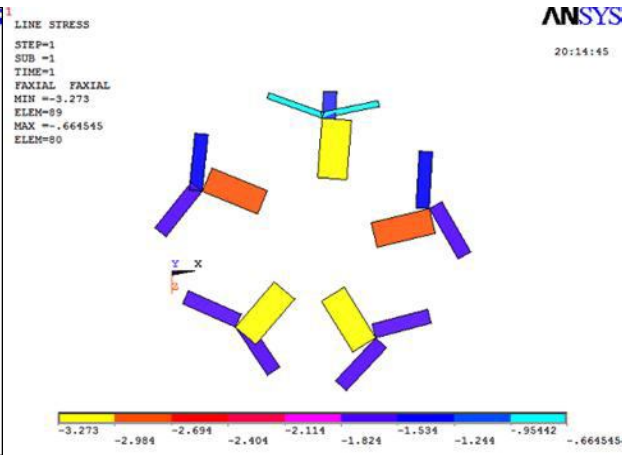
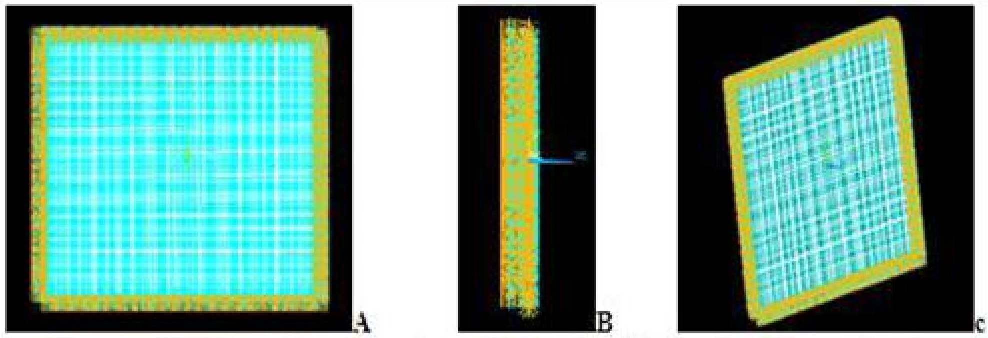


Figure. 3. Load Combination 4. Membrane Von-Misses Stresses DueTo Wind Load (T/M2) (X10-2 Mpa)

Figure. 4. Load Combination - Tensile Load In Steel Structural Tubes (Ton)

Therefore, this zone has to be properly reinforced.Wind loading generates suction effects on windward side tending to uplift the membrane in lower zones. Also it has been observed that maximum stresses are concentrated on the top. These zones must be reinforced to overcome the acting stresses. Some representative results are shown as follows. Figure 5 shows the wind effects on the shell. Also, Figure. 6 shows tensile loads in the structural tubes pinned

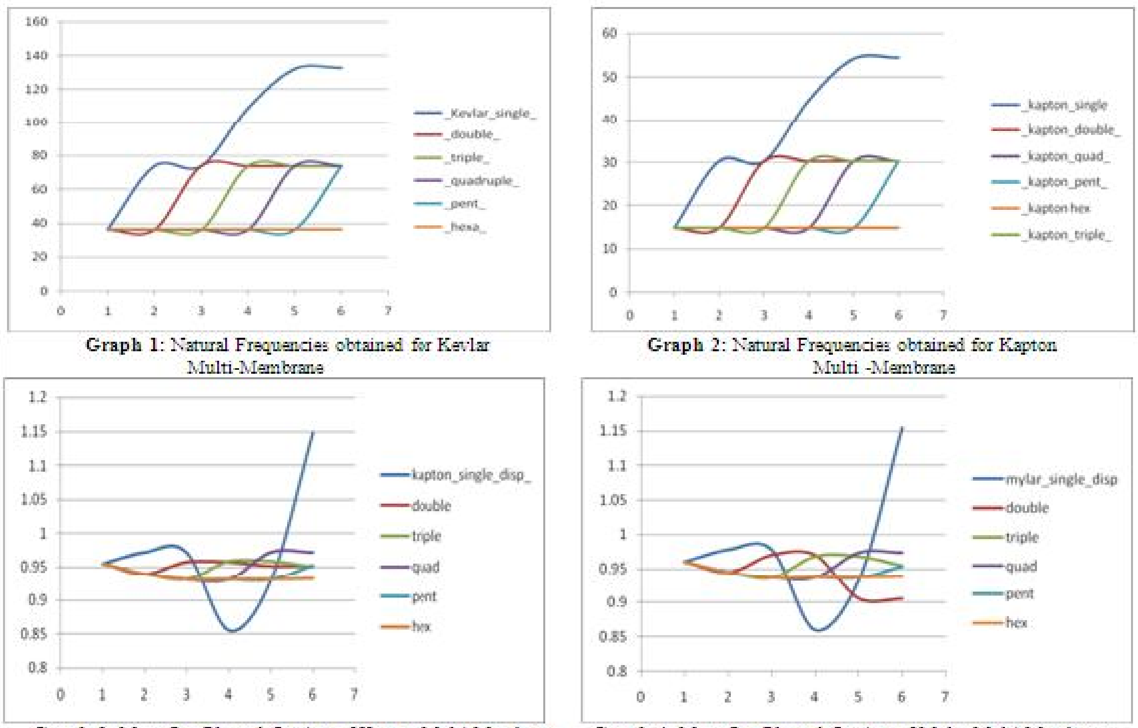
elements.Final sections of the structure, as well as the initial prestress strains applied to the cables are studied. The cables considered here are of the Extra High Strength type (776.9 Mpa)(Figure.4).
Modal Analysis was carried out in ANSYS on multilayer Kapton, Kevlar and Mylar sheets joined by epoxy. The properties of epoxy are A) Density 58Kg/m3 B) Poisson Ration 0.34 C) Stiffness = 1.5Gpa In the boundary conditions, element type Shell-91 is used and all edges are fixed. Each layer of membrane is discretized in total 225 elements. Hexahedral elements with a size ratio of 3 are used. Mode shapes up to 6 are extracted using Block Lanczos mode extraction method with lumped mass approximation.
(Figure.5).



Multilayer Kapton membrane a) Front view b) side view c) isometric view

Figure.5 Depicts The Geometry, Meshing And The Boundary Conditions Of The Multi-Layered Membrane Used For Analysis.

Natural Frequencies and out plane displacements of single layered smart material are obtained in the first 6 mode shapes, similarly in all the cases frequencies are determined from single layer to six layers joined by epoxy. The natural frequencies are plotted from single layer to six layers for kapton, Kevlar and Mylar material and a general pattern of variations has been noted. Similarly Graphs of natural frequencies and out plane deformations are obtained for Kapton, Kevlar and Mylar and the patterns are noted (Figure.6).



Graph 1: Natural Frequencies obtained for Kevlar Multi-Membrane
Graph 2: Natural Frequencies obtained for Kapton Multi -Membrane
Graph 3: Max. Out Plane deflection of Kapton Multi-Membrane
Graph 4: Max. Out Plane deflection of Mylar Multi-Membrane

Figure.6 Mode Shape Vs. Natural Frequency And Mode Shape Vs. Out Plane Deflection

The Graph 1-4 on mode shape vs. natural frequency and mode shape vs. out plane deflection is plotted for single to hexa layers and it is noted that natural frequency decreases with increasing layers also the natural frequency increases as we increase the mode number. Graph 1 and Graph 2 depict the saturation frequency obtained at 6 layered packing for both Kapton and Kevlar membranes. The advantage gained for Hex layer in terms of Minimum Aggregate Maximum out Plane deflection is depicted clearly in Graphs 3-4 for Kapton and Mylar Membranes. It is also seen that Kapton has the minimum aggregate out plane deflection as compared to Mylar and Kevlar (Figure.7 and Figure.8)

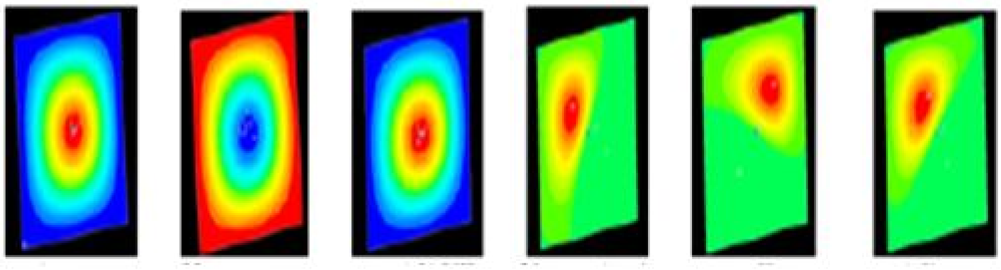


Figure.7 Mode Shapes ,Triple Layered Kapton

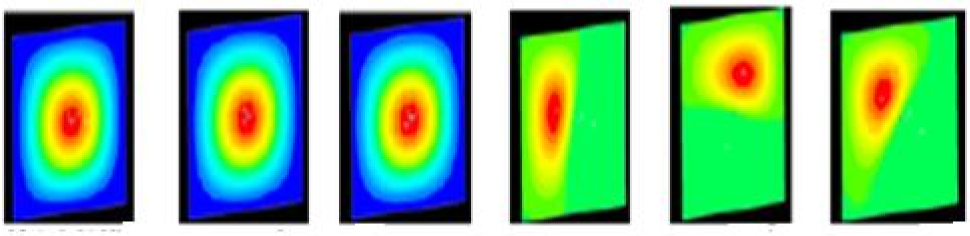


Figure.8 Mode Shapes ,Triple Layered Kevlar

2.5.Theories Used For Analysis

Membrane theory has been used for analysis of reinforced concrete inverted umbrella shell roof. The panel shear for a uniformly loaded hyperbolic paraboloid can be found using the formula (w *a*b/2*h), where a and b are horizontal projection of the shell and h is the central dip and is equal to a/3. RankineGrishoff theory has been used for the analysis of grid roof system which is based on IS 456:2000. The grid roof is analyzed as a solid slab for computation of moments and forces. The ribs are designed as flanged sections to resist moment and shear. In RankineGrishoff theory, twisting moment due to beams is neglected.

2.5.1Quantity Used

Tables 1, 2 and 3 show the volume of steel and concrete used for Inverted umbrella shell roof, grid roof, prestressed hyperbolic shell roof, respectively.

Table 1 Volume of Steel and Concrete for Inverted Umbrella Shell Roof.

Element	Dimension (m)	Volume of concrete (m^3)	Volume of steel (m^3)
Edge member	0.2 * 0.3	4.8	0.076
Compression member	0.2 * 0.65	6.25	0.076
Bays	20 * 20	28	0.134

Table 2. Volume Of Steel And Concrete Required For Prestressed Hyperbolic Shell Roofs

Element	Volume of concrete (m^3)	Volume of steel (m^3)
Parabolic	4.40	0.39
Circular Arc	4.44	0.31
Rectangular Hyperboloid	4.37	0.29

Table 3 Volume Of Steel And Concrete For Grid Roof.

Element	Volume of Concrete (m^3)	Volume of steel (m^3)
Grid roof	135	0.46

5.2 DAMPING TREND ANALYSIS

For the purposes of this trend analysis, the results from the PC spherical shell were added to a databaseprepared in an earlier study, along with results from two papers presented by other researchers to make a total population of 51 shell structures.

6. COST COMPARISON

Table 4 shows the cost comparison of Reinforced concrete Inverted umbrella, Prestressed hyperboloid ofdifferent geometry such as circular arc, parabolic and rectangular hyperboloid and grid roof system.

Table 4 Cost Comparison Of All The Roofs Considered.

Element	Cost of concrete (Rs)	Cost of steel (Rs)	Total cost (Rs)
Prestressed parabola	419510	214060	633570
Prestressed Circular Arc	423716	219365	643081
Prestressed R.H	416365	206136	622501
Inverted umbrella	390500	134878	525378
Grid floor	1350080	217992	1568072

Table.4. shows the total cost comparison of different geometry such as circular arc, parabolic and Rectangular hyperbola of prestressed hyperbolic paraboloid.From the studies, it is clear that circular geometry of prestressed rectangular hyperboloid is costlier than the other two geometry.

6.1 EXTERNAL PRESSURE FAILURE

The mechanism of external pressure failure is different from internal pressure failure. Internal pressure failure can be understood as a vessel failing after stresses in part or a large portion exceeds the materials strength. In contrast, during external pressure failure the vessel can no longer support its shape and suddenly, irreversibly takes on a new lower volume shape. It loses its stability. The first picture shows an internal pressure failure. The second picture shows vessels of reduced volume after external pressure failure. Stability: A stable system is one that is stronger than required. When the vessel is pushed on, it pushes back and returns to its original shape. As external pressure is added to the system, the vessel has less reserve strength left to push back.

The factors which determine the ability of a pressure vessel to withstand external pressure are 1) Thickness (t) 2) Unsupported length of vessel (L) 3) Larger dimension of rectangular cross section of vessel shell (D) 4) Material of construction Maximum allowable pressure for vessel can be increased by

1. Increasing thickness (t)
2. Decreasing unsupported length of vessel (L)
3. Decreasing larger dimension (D)

But D and L cannot be changed because vessel size is designed for required volume for reactions taking place inside. Hence only thickness can be changed.

6.2 SHELL PLATE DESIGN

Design procedure for calculating thickness of shell plate suitable to withstand external pressure is given in UG-28 of ASME Sec viii Div 1 code. First assume initial thickness and calculate maximum allowable external pressure (Pa). This calculated allowable pressure must be greater than external design pressure (P). If it is lower than external design pressure then thickness of vessel should be increased. Given input is D = 1734 mm, L = 3960 mm, t = 5mm, P = 1 Bar = 0.01033 Kg/mm2. Rectangular shell with above dimensions is shown in figure 3.

6.3 DESIGN OF STIFFENER

Stiffeners have to satisfy certain requirements of moment of moment of inertia. Condition is that available moment Of inertia with neutral axis parallel to axis of shell should be greater than required moment of inertia. If above condition is not satisfied then stiffener with new cross section is taken. Required moment of inertia is calculated using procedure given in UG-29 of ASME Sec viii Div 1.

$$B = {}^{3}/_{4}\left(\frac{P * D}{t + {}^{A_s}/_{L}}\right)$$

From the formulaValue of factor B is 18.34 From value of B and from CS-2 chart value of factor A = 0.00017 Required moment of inertia is given by

$$I_s = \frac{\left[D^2 * L * \left(t + {A_s}/{L}\right) * A\right]}{14}$$

After substituting values we get Is = 199479.4 mm4. Cross section of stiffener is „C‟ shape. Available moment of inertia for „C‟ shape cross section is 441203.5 mm4. Therefore, Available moment of inertia of stiffening ring is greater than required moment of inertia.

Stiffeners are welded to the shell with leg size of 6 mm. Procedure for checking of strength of attachment weld is given in UG-30 of ASME Sec viii Div 1. Criterion for checking weld strength is that actual load acting on the weld must be less than allowable weld for the load. The actual load on the weld is a combination of the radial pressure load between the stiffeners, the weld shear flow due to the radial load through the stiffener, and the external design load carried by the stiffener. After calculations of strength of attachment weld, it is found that actual load acting on weld is 108.04 N/mm. And allowable load for weld is 455.4 N/mm. Thus, allowable load for weld is greater than actual load on the weld. Therefore, fillet weld leg size of 6 mm is strong enough to withstand external pressure load.

7.EXPERIMENTAL METHOD

Vibration measurements were obtained using the impact from dropped sandbags. The purpose ofimpact vibration tests with sandbags is to ascertain damping characteristics at vibration amplitudeslarger than normal micro tremors.

7.1 MEASUREMENT SYSTEM

In this study, servo velocity detectors (Figure.9) were utilized for their high accuracy and stability, andthe data fed to a computer via portable vibration meters, where it was converted into velocity graphs.Velocity detector sensitivity specifications taken from the website of Tokyo Sokushin Co. Ltd. Aregiven in Table 5(Tokyo Sokusin URL)(Table.5).

Figure.9 ServoVelocity Detectors

Table .5 Velocity Detector Sensitivity

Component		1 component (horiz./vert. switchable)
Measuring frequency		0.2～100Hz
Range	Velocity	±0.1m/s (±10cm/s)
	Acceleration	$\pm 20 m/s^2$
Sensitivity	Velocity(H)	0.1V/m/s
	Velocity(L)	100V/m/s
	Acceleration	$500 mV/m/s^2$

8 FINITE ELEMENT MODEL

A “SOLID45” finite element model was constructed to model the spherical shell roof of the FunabashiGeneral Education Centre’s planetarium. The beams that for the joint area of the shell are modeled using the “BEAM4” element type, and analysis carried out with pin support boundary conditions at theedge of the model. The equivalent young modulus value of 3.3N/m2 used in this analysis was derivedas the average of 2.9N/m2 (reinforced concrete) and 3.7N/m2 (prestressed concrete), values used in previous studies. The model elements and parameters used are shownbelow.

8.1 Shell

Element type: SOLID45
Material properties
Equivalent Young’s modulus: E = (2.9, 3.3, 3.7) × 1010N/m2
Mass density: ρ= 3.8 × 103kg/m3, Poisson’s ratio: ν= 0.2

8.2 Beams

Element type: BEAM4
Material properties
Equivalent Young’s modulus: E = (2.9, 3.3, 3.7) × 1010N/m2

Mass density: ρ= 3.8 × 103kg/m3, Poisson's ratio: ν= 0.2
Geometrical moment of inertia: I = 5.94 × 10-3m4

8.3 Analytical Results

The results of natural vibration analysis of the above model are shown below. Modal analysis wascarried out using the block Lanczos method. An overview of natural frequencies is given in Table.6 ,and examples (Figure.10,11,12 & 13) of natural vibration modes.

Table.6 Natural Vibration Modes

Order	Equivalent Young's modulus $\times 10^{10}$(N/m^2)		
	2.9	3.3	3.7
1	21.94	23.40	24.78
2	27.00	28.80	30.49
3	27.01	28.82	30.52
4	36.50	38.94	41.23
5	36.55	38.99	41.28
6	42.45	45.28	47.95
7	42.73	45.58	48.26
8	48.93	52.19	55.27
9	52.98	56.52	59.84
10	54.05	57.65	61.05
11	56.98	60.78	64.36
12	57.42	61.25	64.85
13	58.37	62.27	65.93

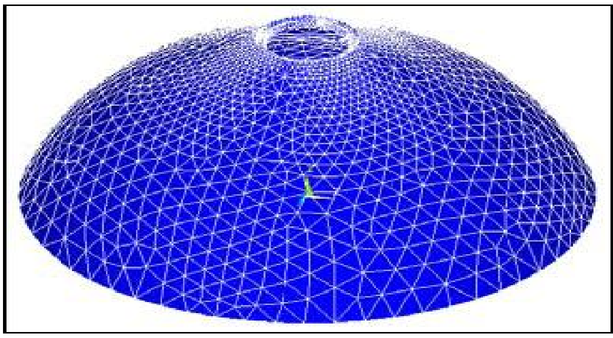
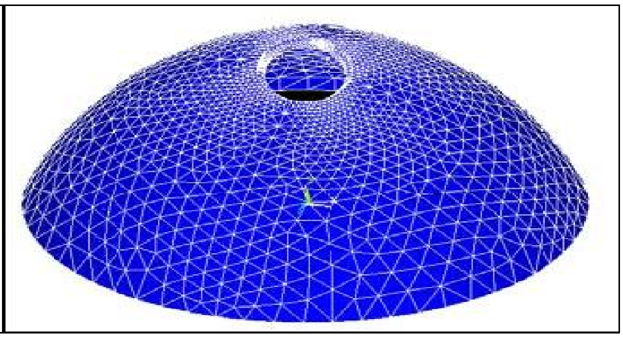



Figure 10 1st Natural Vibration ModeFigure 11 2nd Natural Vibration Mode

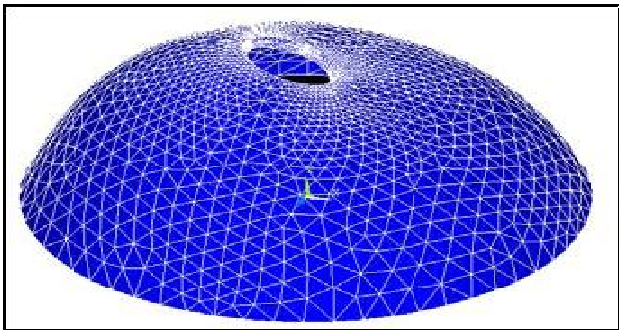
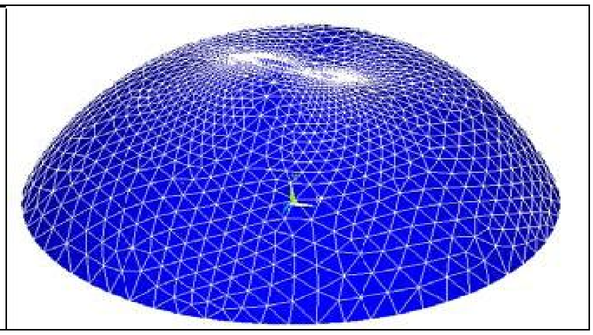
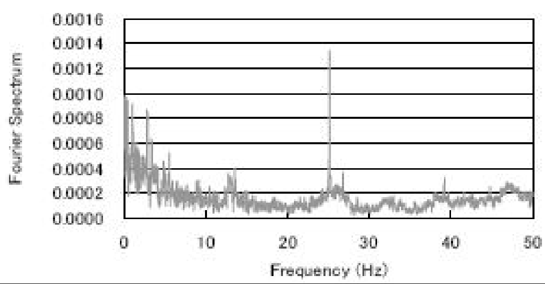



Figure 12 3rd Natural Vibration ModeFigure 13 4th Natural Vibration Mode

4 Comparison Of Experimental Data And Analytical Results

The following is a comparison of data obtained experimentally with the results of the natural vibrationanalysis.Waveform processing of the damping free vibration data from the sandbag drop test has not beencarried out. However, for reference, an example Fourier spectrum from the edge beam part and mainbody of a PC spherical shell.



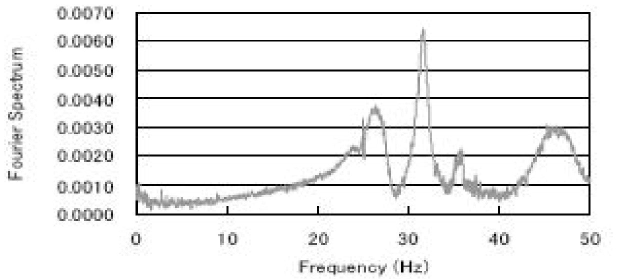


Figure 14 Fourier Spectrum At Edge Beam

Figure 15 Fourier Spectrum Near Top Of Shell

A peak is visible at around 25Hz. This is a much larger response than in micro tremor observation, sothis is thought to indicate the effect of the shell itself at the edge beam.In the Fourier spectrum from near the top of the dome, we can see that, as with micro tremorobservations, there are peaks between 25Hz and 40Hz. Therefore in future research it is necessary tocarry out waveform processing for predominant frequencies over 25kHz(Figure.14 & Figure.15).

8. CONCLUSION

Generally speaking, the analysis of prestressed structures withcomplex geometries, such as the roofing presented here involvesnon-linear models, able to adjust the complicated interactionsstress-strain taking place in the structure. The approach wehave explained above allows to simultaneously determiningstresses and strains in the prestressed condition over the closedstructural system, while ensuring a minimum weight structure.This is achieved by coupling the mechanical-structural modelwith a subroutine for optimization. Nevertheless, the globalanalysis of these roofing structures must take into accountother parameters, such as the wind fluctuations, which causeaero dynamical instabilities on the membrane. These can betaken into account by feed backing into our model,experimental data from wind tunnels. The coupled modelpresented herein allows including many other design/performance parameters.Shell thickness of 6 mm with four stiffeners is safe for external design pressure of 1 Bar. Tube sheet thickness according to ASME code is 19 mm. And according to TEMA it is 22 mm. For external loading of 0.1 Mpa, ‘C’ shape cross section of stiffener with given dimension is suitable. Stiffener is welded to shell with one side continuous weld and other side intermittent weld. Weld leg size of 6 mm is enough to carry radial pressure load between stiffeners and weld shear flow doe to radial load between stiffeners. Forty two bolts with size of M 12 are sufficient to hold water box and absorber side tube sheet assembly for design pressure of 1 Bar. Thirty six bolts with size of M 12 are sufficient to hold water box and evaporator side tube sheet assembly for design pressure of 1 Bar.

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