Collapse of reticulated domes, a case study of Talakan oil tank
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Abstract
In this paper, instability of single layer reticulated domes is discussed. This purpose is elaborated by a case study on Talakan oil tank dome which is analyzed in this work with research package. This paper provides technical information related to the design, fabrication and collapse of Talakan dome. The secondary paths, especially in unstable buckling, can play an important role in the loss of stability and led to failure of the structure. The authors show that the stiffness of the dome is not adequate to prevent buckling under the prescribed snow loads. It is also shown that the capacity of the dome to resist eccentric snow load is about half of its capacity to resist symmetric snow loads. Although six combinations of load and support fixity are included in design assumptions, considerable attention has been focused on the bifurcation behavior in Talakan dome. The stiffness of the aluminum sheets of the roof cover have not been taken into account in the stability analysis.

Keywords: bifurcation point, secondary path, reticulated dome, unstable buckling, eccentric load,

1. Introduction
Domes have many advantages, from both a structural and an architectural point of view [1-3] and these advantages have been comprehensively analyzed by Makowski [4]. Furthermore research on these structures has been a popular topic in recent years[5]. Although linear static analysis shows its ability in classical buildings, it is important that the nonlinearity which contains both material and geometry[6] be considered in a full analysis [7, 8]. The behavior of single layer structures is highly nonlinear and is affected by diverse factors [1], such as mesh density, joint rigidity [9], geometrical shape of domes, non-uniform loads, support conditions, rigidities of the connection between members [10], members slenderness ratio, half-subtended angle for members, initial imperfection, etc. [11]. Furthermore, it is required that instability check be examined through the nonlinear analyses [12-14]. Instability is one reason of collapse in single layer lattice domes, a comprehensive review of which has been carried out by Dulácska and Kollár [15].

As mentioned, non-uniform load which Kato and coworkers [11] are pioneers in researching about is one of the collapse reasons. Additionally, collapse due to non-uniform loads is affected by buckling, whether locally in elements or globally in whole of the structures. Accordingly, buckling is one of the most important parameters in the structural design of single layer reticulated domes[2, 16] because these domes are very sensitive to buckling and start to lose their load bearing capacities after buckling [11].

Knowledge about the stability behavior is one the requirements for investigation of nonlinear response of structures. In such an analysis, limit points and bifurcation points are very important and must be found by a standard procedure. For a bifurcation point, this means, that all the branches emerging from the bifurcation point
are of interest in the analysis [17]. The main difficulty in analyzing the stability behavior of a structure follows from the fact that the examined system of equations is singular at a limit point or a bifurcation point [18-20]. In a critical point the structure may jump to an inverted equilibrium shape and load capacity reduces notably [21, 22]. Buckling behavior can be interpreted as an instability which is induced by a singular tangent-stiffness matrix [23, 24]. This matrix is not constant throughout the deformation history of the structure. This is due to the effect of the material and geometric nonlinearities [14]. With these in mind about tangent-stiffness matrix and bifurcation path, the nonlinear analysis of the present structure, Talakan oil tank dome, is done by the technique which is provided by the authors in their previous work [14]. In this technique a method for reliable detection and accurate computation of singular points on the load path is presented and applied to a space structure subjected to symmetric and asymmetric snow loads. In recent years, finite element method has been very popular [3, 25, 26]. In the present work the method is implemented in a combined materially and geometrically nonlinear finite element analysis computer program based on an incremental/iterative Newton–Raphson solution procedure. It is necessary to mention that the effect of velocity [27] in loading is not assessed in this study. After making some explanations about bifurcation path a brief discussion will be made on construction of Talakan oil tank dome in Russia. There are two reservoirs which have 50000 m$^3$ capacities individually and were used to cover a circular area with aluminum sheets for oil tanks in Russia. The aluminum geodesic dome is a self-supporting cover used usually in refineries and tank farms, where the storage product and the structural tank components must be protected from atmospheric and environmental influences, while at the same time, minimize hazardous vapor emissions and preventing water from entering the storage tanks. Each dome in Talakan is located on 78 supports. Nonetheless, they were damaged due to some design problems. The purpose of this article is to find the cause of destruction in Talakan oil domes. Figure 1 and Figure 2 show a preview of Talakan oil domes. It is notable to mention that the present study is focused on a circular dome but can be extended on other shapes such as cylindrical [28]. The most important issues in these structures are “geometry”, “loading” and “connection and support”. They are usually symmetric in geometry and structures with symmetry have played an important role not only in decades, but also in centuries [23]. Another important factor in geometry of domes is the ratio of span to height [29]. Three factors are envisaged to parameters can participate in the collapse of Talakan oil tank domes. These are: geometric imperfections, secondary path in nonlinear unstable buckling and connections.

Figure 1: Installation of Talakan oil tank dome

Figure 2: Collapsed dome a) real photo b) simulated structure

2. Construction

The aluminum dome is a fully triangulated structure designed as a self-supporting dome roof where by only its outer edge is connected to the tank shell around the outer rim. The domes are fabricated from high strength aluminum alloys that resist corrosion as well as chemical, ultraviolet and ozone degradation. Aluminum domes are generally constructed according to API 650 appendix G [30], the standard setting details for aluminum domes on storage tanks.

Reticulated domes like Talakan, made of aluminum alloy, are often used for covering the oil tanks. Typical domes in Talakan have the spherical surface with a diameter on the support from 30 to 60 meters and the crest rising from 6 to 11 meters.

The structure for covering the oil tank, which is discussed in this paper, is a single layer reticulated dome, made of straight I-section elements with the material of aluminum alloy. The roof of the domes is from aluminum sheeting fixed to the I-section element and the supports of the domes are at the nodes of lowest ring.

The stability of the dome being designed for dead, snow and wind loads is of paramount importance for the ultimate load bearing capacity of the structure. It is also necessary to calculate the effects of temperature changes and relative temperature in which the dome is built. The stability of the dome can be affected by following design parameters:

- The radius of the spherical surface.
- The form of the dome meshes cells.
- The shape and area of the members cross section.
- Design and construction of the connections at the nodes of the dome.
- Construction and condition of the supports.

3. Geometry
The middle surface of the dome is a cap forming part of a sphere. The primary dimensions of the cap are as follows:
Radius of the sphere 48.600 m
Radius at the supports 30.000 m
Height of the cap 10.364 m
The cap is subdivided into 14 rings that subtend equal angles at the midpoint of the sphere. The area of the dome is subdivided as shown in Figure 3. The plan consists of 6 congruent sectors. Figure 4 shows the elevation in the direction of axis y.

All members are made of aluminum with modulus of elasticity $7 \times 10^7 \text{kN/m}^2$. The members on the outer perimeter are square boxes $200 \times 200 \text{mm}$ with wall thickness 12 mm. The area of their cross-section is $9024 \text{mm}^2$. All other members in the dome are I-sections with depth 200 mm, flange width 100 mm, web thickness 4.5 mm and flange thickness 6.0 mm. The area of their cross-section is $22046 \text{mm}^2$. The dome is covered with 1.2 mm thick aluminum sheets. Figure 5 shows the cross section of members.

Figure 3: Plan of the dome from in-house Space-Frame software (all dimension in m)

Figure 4: Elevation of the dome from in-house Space-Frame software (all dimension in m)

Figure 5: a) ring members connected to vertical supports b) typical dome members (all dimension in mm)

4. Loads
The self-weight of the members is concentrated at the corresponding nodes. Their unit weight is taken as $27.1 \text{kN/m}^2$. The aluminum sheets covering the dome weigh $0.0342 \text{kN/m}^2$. The design value of the snow load is determined by the following formula [31]:

$$S = S_g C_e C_t \mu$$

- $S$ design snow load ($\text{kN/m}^2$ of horizontal area)
- $S_g$ design weight of snow ($2.4 \text{kN/m}^2$ of horizontal area for Talakan)
- $C_e$ pressure coefficient (1.0 for full enclosure)
- $C_t$ temperature coefficient (1.0 for unheated roof)
- $\mu$ pressure distribution factor

Let the radius at which the tangent to the cross-section of the dome makes an angle of 30 degrees with the horizontal be denoted by $r_{30}$. Three cases are considered for the pressure distribution factor:

**Case 1**: symmetric snow load

$$r \leq r_{30} : \mu = 1.0$$

$$r > r_{30} : \text{linear variation from } \mu = 1.0 \text{ at } r = r_{30} \text{ to } \mu = 0.74 \text{ at } r = r_{\text{sup port}}$$

**Case 2**: eccentric snow load

The snow load is assumed to be eccentric in the direction of the positive x axis. The second and third quadrants of the plan of the dome are unloaded. The pressure distribution factor in the first and fourth quadrants depends on the angle $\beta$ and the distance $z$ from the axis of the dome as defined in Figure 6:

Figure 6: Variable definition for eccentric load
$r \leq r_{30} : \quad \mu = 2.36\left(\frac{z}{r_{30}}\right)^2 \sin \beta$

$r > r_{30} : \quad \text{linear variation from } \mu = 2.36 \text{ at } r = r_{30} \text{ to } \mu = 1.87 \text{ at } r = r_{support}$

**Case 3**: eccentric snow load

The snow load is assumed to be eccentric in the direction of the positive x axis. The second and third quadrants of the plan of the dome are unloaded. The pressure distribution factor in the first and fourth quadrants depends on the angle $\beta$ and the distance $z$ from the axis of the dome as defined in Figure 6.

$$\mu = 3\sin \beta \sqrt{\frac{2f}{d}} \sin 3\alpha \quad \text{where } \alpha = \arcsin \frac{z}{R}$$

$f$  height of the dome  
$d$  diameter of the dome at the support  
$R$  radius of the sphere  

These three cases are shown schematically in Figure 7.

Figure 7: Schematic figures of snow load cases a) load case1 b) load case2 c) load case3

**5. Supports**

The dome is supported at every node on its outer perimeter. Two support cases are considered:

*Fixed*: Each node on the perimeter is fixed against translation in the direction of each of the three coordinate axes.

*Free*: Each node is fixed against translation in the vertical direction. The perimeter node on the positive x axis is fixed against translation in both horizontal directions. The perimeter node on the negative y axis is fixed against translation in the direction of axis x.

The fixed supports model a situation where the ring members, the wall of the tank, and the studs are so rigid that they prevent horizontal motion at the perimeter of the cap.

The free supports prevent translation of the dome as a rigid body in the horizontal plane, but allow the outer perimeter to change its shape (extension of the members in the outer ring).

The real support of the dome in the structure is intermediate between the two cases that are analyzed here.

Well-designed joints are essential for satisfactory performance of a structure. In aluminum frameworks with riveted or bolted gusset plates it has been estimated that the weight of the joints is about 20-30 % of the weight of the structure; in cost terms the ratio is much larger[32]. In Talakan dome connections are made by bolting. Figure 8 and Figure 9 shows the connections which are used in Talakan.

Figure 8: Bolted connections of aluminum sheets to members (all dimension in mm)

Figure 9: Member connections in Talakan domes

**6. Generator**

The nodes, members, supports, loads and load combinations of the dome are generated for the analysis by the software. The topology of the nodes and members is described with the ring and line numbers defined in Figure 10. The rings are numbered consecutively from 0 at the apex to 14 at the perimeter.

Figure 10: Ring and line indices for the plan of the dome

The number of nodes for rings 1 to 13 is 6 times the ring index. Ring 0 has 1 node (the apex), ring 14 has the same number of nodes as ring 13. The node on the x-axis of a specific ring has line number 0. Only in ring 14 the nodes are shifted by half an interval in the anti-clockwise direction. The line numbers of the other nodes on the same ring are consecutive in the counter-clockwise direction.
The location of a node is denoted by the ordered pair (ring, line). The initial numbering of the nodes starts with 0 at the apex, followed by the nodes on ring 1 in the order of their line numbers. The numbering continues with rings 2 to 14. The node with number k is denoted by “nk”, so that node 47 has the name “n47”. The ordering of the nodes in the system equations differs from the numbering that is introduced here. It will be discussed later in this paper.

A member is identified by the numbers of its start and end nodes. If a node number has less than 3 digits, it is expanded with leading digits 0. The member from node k to node m is denoted by “bkm”, so that the member from node 24 to node 7 is denoted by “b024007”.

The supports at a node whose number has the digits xyz are named “sxyzX1”, “sxyzX2” and “sxyzX3”. The vertical load at node xyz is named “Lxyz”.

The nodes are generated in the sequence of their numbers. The members and the tri-angular facets of the roof cover are generated per sector in a ring. The sectors are numbered consecutively from 0 to 5 in the counterclockwise direction, starting with 0 at axis x. For rings 1 to 13, the members and facets in sector s are generated according to the rules in Figure 11:

Figure 11: Members and facets in a sector of a ring in range $1 \leq \text{ring} \leq 13$

The members and facets in the outer ring 14 are generated independently from the segments. The indices for a typical interval are shown in Figure 12.

The self-weight of the members is computed in a cycle over all members and added per node. The weight of the roof cover is computed in a cycle over all triangular facets. One third of the weight of a facet is attributed to each of its nodes. The snow load is also computed in a cycle over the facets. The snow load intensity is computed at the midpoint of the facet. One third of the load on the facet is attributed to each node.

Figure 12: Members and facets in ring 14

7. Population of the System Equations

The numbering of the nodes in the generator leads to a system stiffness matrix with a nearly triangular band structure. The band is narrow at the apex displacements and wide at the support displacements. The number of multiplications required to decompose the matrix is given approximately by the following formula:

$$N \approx \frac{1}{2} \sum_{i=1}^{n_R} (k_i + 6)^3$$

(7)

$N$ number of multiplications for decomposition of the stiffness matrix

$k_i$ number of degrees of freedom in ring i

$n_R$ number of rings in the dome

The number of degrees of freedom in ring i is a function of the ring index:

$$k_i = i n_S n_F$$

(8)

$i$ ring index

$n_S$ number of sectors per ring (6)

$n_F$ number of degrees of freedom per node (3)

Expression (8) is substituted into (7):

$$N \approx \frac{1}{2} 6^3 \sum_{i=1}^{15} (3i + 1)^3 = 45.7 \times 10^6 \text{ operations}$$

(9)

Since the system matrix is decomposed at least twice per load step, and on the order of 80 load steps are required for one analysis, the nodes are renumbered to reduce the solution effort. A wave front algorithm is used for this purpose. The number of operations required for decomposition is given approximately by:
\[ N_w \approx \frac{1}{2} \sum_{w=1}^{n_w} (k_w + 6)^3 \] (10)

\( N_w \) \text{ number of multiplications for decomposition with wave front numbering}
\( k_w \) \text{ number of degrees of freedom in wave front } w
\( n_w \) \text{ number of wave fronts in figure 7}

The number of degrees of freedom in the wave fronts is:
\[ k_w = 3n_{nw} + 6 \] (11)

\( n_{nw} \) \text{ number of nodes in wave front } w

Expression (11) is substituted into (10):
\[ N_w \approx \frac{1}{2} 3^3 \sum_{i=1}^{15} (n_{nw} + 4)^3 = 7.17 \times 10^6 \text{ operations} \] (12)

The number of multiplications for the decomposition of the stiffness matrix is reduced by a factor 6.4 through the use of the wave front renumbering algorithm. The time required for the determination of a complete load path up to the unstable state is approximately 8 seconds on a conventional laptop computer.

8. Load Combinations

The self-weight of the dome including its sheeting enters all load combinations with a factor 1.10. Each of the three snow load cases described in section 2 is combined with each of the 2 support cases described in section 3. Two models for space truss and space frame analysis of the dome were established. This leads to 12 load and model combinations shown in Table 1. It is necessary to mention that temperature loads are not considered in the analysis.

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<th>Load and model combinations details</th>
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To analyze these combinations, in-house or research packages which have been developed by the authors were used. In these packages the first bifurcation point can be found at unstable buckling mode with use of frame elements with six degrees of freedoms.

The physically nonlinear part is based on a three-dimensional von Mises based plasticity with return mapping to the yield surface. The geometrically nonlinear part is based on an updated Lagrangian large deflection small strain formulation \[33\]. In the package Space-Truss bifurcation point can be found at stable buckling mode with using of truss elements by three degrees of freedoms. The final package which was also used is commercial software, ANSYS (registered by Mahabghods Company). Four analyses were made with ANSYS. These analyses are, 1) stable buckling mode with using of truss elements, 2) stable buckling mode with using of frame elements, 3) limit point mode with using of truss elements, 4) limit point mode with using of frame elements. In the present work the stable buckling mode with frame elements in ANSYS will be shown to compare how these commercial packages cannot detect the singular points on the load path of the dome.

9. Summary of the Results of the analysis for truss models

The load that is generated for a load combination is regarded as a load pattern. The load pattern is multiplied by a load factor to yield the load that is acting on the dome. For each of the load combinations, the load factor that leads to instability has been determined. A maximum load factor of 1.0 implies that the dome buckles at the load of the load pattern.

The initial load factor increment per load step is 0.002. The number of steps required to reach instability is determined by the solution algorithm. The maximum load factors \( LF_{\text{max}} \) for the load combinations are shown in the second column of Table 2. The maximum snow load intensity \( q_{\text{max}} \) corresponding to the maximum load factor is shown in column 3. It is computed as the product of the design weight of snow \( S_e \) with the maximum value of the pressure distribution factor \( \mu_{\text{max}} \) and the maximum load factor \( LF_{\text{max}} \) for the load combination:
\[ q_{\text{max}} = S_e \mu_{\text{max}} LF_{\text{max}} \]
Also shown are the displacement coordinates $u_1, u_2, u_3$ of node “n469” (which lies on axis x in ring 13) for the singular configuration of the dome. The buckling of the shell is initiated at this node in all 6 load combinations.

The very low load factors obtained demonstrated that the assumption of space truss behavior is not really valid for this structure.

### 10. Summary of the Results of the analysis for frame models

The maximum load factors $LF_{\text{max}}$ for the next six combinations (C7 to C12) are shown in the second column of Table 3. The following table shows the load factor which the dome reaches a singular configuration.

Furthermore, to clarify how the commercial programs work in such structures one model was established by ANSYS and the load factor was compared. The load factor which is found by ANSYS is $LF_{\text{max}} = 2.500$ in combination C7.

### 11. Conclusions

The analysis clearly shows that the stiffness of the dome is not adequate to prevent buckling under the prescribed snow loads. The computed load factors shown in Table 2 vary from 0.109 for combination C5 to 0.285 for combination C1. The required value of the maximum load factor in each load combination shall be at least 1.0. This table also shows that the capacity of the dome to resist eccentric snow load is about half of its capacity to resist symmetric snow loads. The load carrying capacity with fixed supports exceeds the capacity with free joints by 30% for symmetric snow load and by 15 to 20% for eccentric snow load. But as mentioned before, the very low load factors obtained demonstrated that the assumption of space truss behavior is not really valid for this structure.

The case C11 of Table 3, with $LF_{\text{max}} = 0.852$, constitute the reason for collapse of the Talakan dome. This lead combination corresponds to the free system of support nodes aggravated by the heavily unsymmetrical snow loading pattern depicted by load case 2 in Figure 7. Careful consideration of Table 3 shows that all the factors of safety are greater than 1 for load combination C7, C8, and C9 where the support nodes were assumed to be fixed regarding all translational degrees of freedom. In load cases C10 to C12 only vertical restraint is assumed in the support nodes together with some extra global restraints to remove the rigid body motions. A substantial drop is observed in the factor of safety as a result, even for load case C10 where symmetrical snow loading is applied to the structure, coming down from 2.182 to 1.492. For proper and expected behavior of any shallow reticulated dome, existence of adequate horizontal restraint at the supports is a must. It seems that the axial stiffness of ring members depicted in Figure 5-a could not perform this duty adequately for the huge Talakan dome. This deficiency together with the unsymmetrical snow load case of Figure 7-b knocked down the safety factor to 0.852 of case C11 in Table 3. Considering the self-weight multiplies of 1.1 of Table 1 this undeniably low factor of safety could be reasonably increased to $0.852 \times 1.1 = 0.937$, a value not in itself large enough to rescue the magnificent Talakan oil tank of Figure 2 from inevitable collapse in the rather snowy weather of Siberia.

The true limit load of an as-built dome, owing to the existence of initial imperfections in its construction, is always below the limit load of its primary equilibrium path. This knock-down effect may be quite substantial if there is a bifurcation point in the “perfect analysis” of the dome with as-designed geometry and perfect connections. Analysis shows that however connections were assumed perfect but the stability of dome was not
sufficient under prescribed loads. So it was concluded that the main reason for collapse of Talakan oil dome is the secondary path and low buckling load resistant in elements. In Figure 15 load-displacement curve of center node for Talakan dome is shown. In this figure perfect and secondary path are plotted.

Figure 15: Load-displacement of dome for center node

The primary reason for the low buckling load is the little difference in the direction of the members that meet at a node such as “n469”. The consequences of this geometric property are illustrated in Figure 16. For R=48.6 m and $\Delta \theta = 2.72$ degrees, the elevation of node “n469” above the secant between nodes “n397” and “n547” is:

$h = R(1.0 - \cos \Delta \theta) = 48.6 \times (1.0 - 0.998873) = 0.0547$ m

The stability theory of a regular 2-member truss under vertical load shows that a vertical displacement of 42.3% of the height of the truss leads to snap-through. A displacement of the order of magnitude $0.423 \times 0.0547 = 0.0286$ m of node “n469” relative to nodes “n397” and “n547” in the direction to the midpoint of the sphere is thus sufficient to cause instability. The small angle change $\Delta \theta$ is a consequence of the large radius of the sphere, given a member length of approximately 2.3 m. The question arises whether a dome with this large radius should be conceived as a single layer space truss.

The results of the analyses show that the displacements prior to buckling are small. They cannot serve as indicators of imminent instability. The displacements of node “n469” tend to be significantly larger than the displacements of the neighboring nodes not only at instability, but also at loads which are smaller than the buckling load. This may be attributed to the geometric pattern of the members in the vicinity of node “n469”.

Figure 16: Angle change between adjacent members at node n469

The change in direction of the diagonal members in the x axis leads to a discontinuity in the stiffness of the shell along axis x. It is well known from shell theory that discontinuities in stiffness lead to unfavorable bending moments. These moments cause transverse displacements, which increase the tendency of the shell to buckle locally.

As seen previously in this paper, a same truss is analyzed with a commercial software package. The nonlinear structural analysis algorithm of this package does not detect the singular points on the load path of the dome. It could be a big challenge in designing such structures with a high ratio of snow load compared to their dead loads. General-purpose-nonlinear-computer programs that do not contain an option to obtain the lowest bifurcation path should be used with great caution by designers who are dealing with the analysis of shallow reticulated domes or other similar structures.

The snow load intensity under which the dome buckles in the field can exceed the computed snow loads due to secondary load carrying mechanisms that are not included in the space truss analysis that is presented in this report:

(1) The joints of the structure may be able to transmit bending moments. This will increase the buckling load. The degree of improvement depends on the detailing of the joints.

(2) The stiffness of the aluminum sheets of the roof cover has not been taken into account in the stability analysis. While the thin sheets cannot resist significant compressive stresses, they are capable of resisting tension fields. This can lead to an increase in the buckling load. Additional insight into the stability behavior of the dome can be gained by means of the interactive graphical user interface of the software, which permits the presentation of the behavior of all components of the dome at all points of the load path.

References


**Figures caption**

Figure 1: Installation of Talakan oil tank dome
Figure 2: Collapsed dome a) real photo b) simulated structure
Figure 3: Plan of the dome from in-house Space-Frame software (all dimension in m)
Figure 4: Elevation of the dome from in-house Space-Frame software (all dimension in m)
Figure 5: a) ring members connected to vertical supports b) typical dome members (all dimension in mm)
Figure 6: Variable definition for eccentric load
Figure 7: Schematic figures of snow load cases a) load case1 b) load case2 c) load case3
Figure 8: Bolted connections of aluminum sheets to members (all dimension in mm)
Figure 9: Member connections in Talakan domes
Figure 10: Ring and line indices for the plan of the dome
Figure 11: Members and facets in a sector of a ring in range $1 \leq \text{ring} \leq 13$
Figure 12: Members and facets in ring 14
Figure 13: C4, singular configuration
Figure 14: The first buckling mode-shape in ANSYS for frame element model, C7
Figure 15: Load-displacement of dome for center node
Figure 16: Angle change between adjacent members at node n469

**Tables caption**

Table 1: Load and model combinations details
Table 2 : Summary of the stability analyses of the dome
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<th>$q_{\text{max}}$ (kN/m²)</th>
<th>$u_1$ (mm)</th>
<th>$u_2$ (mm)</th>
<th>$u_3$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>0.285</td>
<td>0.684</td>
<td>-12.1</td>
<td>0</td>
<td>-16.9</td>
</tr>
<tr>
<td>C2</td>
<td>0.128</td>
<td>0.725</td>
<td>-16.2</td>
<td>0</td>
<td>-22.1</td>
</tr>
<tr>
<td>C3</td>
<td>0.156</td>
<td>0.666</td>
<td>-15.2</td>
<td>0</td>
<td>-20.9</td>
</tr>
<tr>
<td>C4</td>
<td>0.198</td>
<td>0.475</td>
<td>-23.1</td>
<td>0</td>
<td>-31.4</td>
</tr>
<tr>
<td>C5</td>
<td>0.109</td>
<td>0.617</td>
<td>-22.2</td>
<td>0.1</td>
<td>-30.1</td>
</tr>
<tr>
<td>C6</td>
<td>0.125</td>
<td>0.534</td>
<td>-14.1</td>
<td>0.1</td>
<td>-20.2</td>
</tr>
</tbody>
</table>

Table 3: Summary of the stability analyses of the dome

<table>
<thead>
<tr>
<th>Combination</th>
<th>$LF_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C7</td>
<td>2.182</td>
</tr>
<tr>
<td>C8</td>
<td>1.234</td>
</tr>
<tr>
<td>C9</td>
<td>1.402</td>
</tr>
<tr>
<td>C10</td>
<td>1.492</td>
</tr>
<tr>
<td>C11</td>
<td>0.852</td>
</tr>
<tr>
<td>C12</td>
<td>1.000</td>
</tr>
</tbody>
</table>

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