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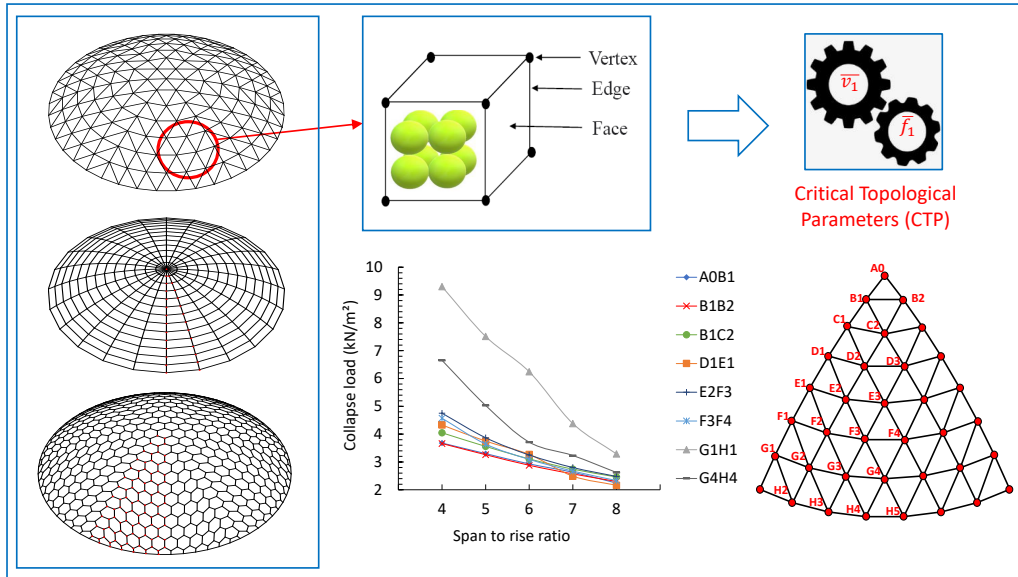
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Graphical Abstract

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Ranjith Kolakkattil, Konstantinos Daniel Tsavdaridis, Arul Jayachandran Sanjeevi



Highlights

Introduction of critical topological parameters on progressive collapse resistance of single-layer lattice domes

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- Crystal lattice formalism is used to identify spatial structural configurations
- Member connectivity in dome configurations is defined based on lattice topological parameters
- Critical Topological Parameters (CTPs) derived to distinguish between configurations
- Higher edge valency of vertices improves the progressive collapse resistance by improving the alternate load paths
- Kiewitt configurations have the highest progressive collapse resistance

Introduction of critical topological parameters on progressive collapse resistance of single-layer lattice domes

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Abstract

Single-layer spatial structures are used to cover large unobstructed areas with efficient material utilisation. As lightweight structures covering large areas with few intermittent columns, they have a higher risk of instability and progressive collapse. The member arrangement in single-layer lattice domes influences stability and robustness, similar to other structural parameters such as the span-to-rise ratio, connection rigidity, and support conditions. The ‘form’ plays a significant role in load resistance, and any change in the form can significantly affect collapse resistance. This research evaluates the progressive collapse resistance of different dome configurations to assess the effect of member arrangement for the first time. The structural configurations of spatial structures are similar to the arrangement of atoms, molecules, or ions in crystal lattices. Therefore, the crystal lattice analogy is introduced in the progressive collapse resistance of lattice domes through this paper. Initially, different topological parameters are introduced for a three-dimensional structure. Then, Critical Topological Parameters (CTPs) are identified to distinguish between different lattice domes based on a crystal lattice analogy. Then, the progressive collapse resistance of lattice domes with varying CTP values is evaluated using the Alternate Path Method (APM). In the final phase, the relationship between configuration types and collapse resistance is investigated based on topological parameters. The results show that a higher edge valency of vertices increases the progressive collapse resistance of single-layer lattice domes. Higher edge valency of vertices and low edge valency of faces result in increasing the alternate load paths in lattice domes, resulting in higher progressive collapse resistance. Therefore, the findings of this research help to identify

the optimum topological parameters to achieve high load resistance in single-layer lattice domes without increasing the overall cost of the structure.

Keywords: Lattice domes, topology, valency, progressive collapse, Alternate Path Method, node buckling, parameterisation

Nomenclature

$\bar{c}_0$	Average vertex valency of cells
$\bar{c}_1$	Average edge valency of cells
$\bar{c}_2$	Average face valency of cells
$\bar{e}_0$	Average vertex valency of edges
$\bar{e}_2$	Average face valency of edges
$\bar{e}_3$	Average cell valency of edges
$\bar{f}_0$	Average vertex valency of faces
$\bar{f}_1$	Average edge valency of faces
$\bar{f}_3$	Average cell valency of faces
$\bar{v}_1$	Average edge valency of vertices
$\bar{v}_2$	Average face valency of vertices
$\bar{v}_3$	Average cell valency of vertices
$B31$	Linear beam element with two nodes
C_0	Vertex valency of cell
C_1	Edge valency of cell
C_2	Face valency of cell

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E_0	Vertex valency of edge
E_2	Face valency of edge
E_3	Cell valency of edge
F_0	Vertex valency of face
F_1	Edge valency of face
F_3	Cell valency of face
N_i	Elements having dimensionality ‘i’
N_0	Number of vertices
N_1	Number of edges
N_2	Number of faces
N_3	Number of cells
N_a^b	Elements having dimensionality ‘a’ and valency for dimensionality ‘b’
V_1	Edge valency of vertex
V_2	Face valency of vertex
V_3	Cell valency of vertex

1. Introduction

Single-layer lattice domes are utilised to cover large spaces with minimum supports. They find widespread applications as roofing for petroleum storage tanks, coal storage yards, and indoor stadiums (Figure 1). The overall geometry of these structures plays a crucial role in efficient load transfer. To ensure structural performance, member cross-sections are optimised to resist external loads effectively. However, because of their lightweight nature and large spans, these structures are susceptible to various forms of instability. The slender members used in such structures are prone to buckling, which can initiate failure of the entire structure. Furthermore, different instabilities such as node instability, line instability, torsional instability, and global instability can initiate the failure of single-layer lattice domes [1]. Snap-through buckling can occur in these structures, which can result in the failure of the



Figure 1: A single-layer lattice dome covering large span. The image reproduced with permission from Geometrica Inc.

entire structure without adequate warning. Therefore, single-layer lattice domes are highly vulnerable to the propagation of different instabilities that can result in the collapse of the entire structure.

Progressive collapse is characterised by the spread of a local failure that leads to a partial or complete structural failure [2]. The collapse of the World Trade Center in 2001 significantly increased research interest in this area [3]. However, most of the research on progressive collapse focused on frame structures, and little attention was paid to spatial structures, especially single-layer spatial structures, although they are more vulnerable to progressive collapse due to their lightness, geometry, and the possibility of different instability propagation [4]. Historical failures, such as the Bucharest dome, due to asymmetric snow loading [5], highlight the importance of investigating progressive collapse in similar structures.

Earlier studies of spatial structures focused on the dynamic effect of instability propagation. The dynamic propagation of instability in a member or node can result in progressive collapse of single-layer spatial structures [6]. Recent studies of progressive collapse of single-layer lattice domes are primarily based on the Alternate Path Method (APM), similar to the numerous investigations on frame structures

[4, 7]. In the APM, the loaded structure is analysed after the removal of a member. If the structure is able to resist the external load even after the failure of a member, the structure is said to have high progressive or disproportionate collapse resistance. However, if the complete structure or part of the structure fails due to the failure of a single structural member, it is said to have low progressive or disproportionate collapse resistance. The APM is widely adopted for different structures to evaluate the progressive collapse potential [8].

There are few experimental studies of scaled-down single-layer lattice domes [9–14]. These experiments contributed to the conduct of thorough numerical analyses considering a range of parameters [15, 16]. Special member-breaking devices were utilised for conducting the APM [9, 10, 14, 17]. The hanging loads at the connections were used to represent the gravity load. The results of the alternate path analysis varied according to the span-to-rise ratio [11], load distribution [18], and the position of member failure [13].

The comparison of dome configurations has revealed differences in collapse resistance. For example, the Kiewitt lamella dome demonstrated a higher Progressive Collapse Resistance (PCR) compared to the geodesic dome [9]. Another experimental study indicated that the load resistance in the Kiewitt dome is achieved through the arch action, while the load resistance of the three-way grid dome is achieved through the beam action [11]. The high intensity of the load distribution can cause progressive collapse at a rapid rate by initiating snap-through buckling [10]. The span-to-rise ratio of the structure can also affect the PCR of lattice domes. Both experimental and numerical investigations revealed that the low span-to-rise ratio of the Kiewitt dome could reduce the chance of progressive collapse, as the arch action was dominant when a low span-to-rise ratio was adopted for the Kiewitt dome [17]. However, a structure with a low span-to-rise ratio results in higher material usage.

Earlier dome collapses indicated the significance of considering the effect of uneven snow-load distribution in reticulated domes [5, 19, 20]. An investigation of the Kiewitt-6 dome indicated that the possibility of disproportionate collapse is higher when non-uniform loading is considered [18]. A partial double-layer structure in the Kiewitt dome can enhance PCR compared to a single-layer structure [12]. Furthermore, connection failure can also initiate disproportionate collapse in spatial structures similar to member failure [14].

Few substructure analyses were performed to identify the progressive collapse mechanism. The member inclination and end restraints can influence the collapse prevention mechanism in spatial structures [21]. Arch action dominated when the inclination of the members in the substructures was high. In another study of substructures, Wei et al. stressed the importance of increasing redundancy in spatial

structures [22]. Tian et al. conducted both experimental and numerical investigations in substructures to identify the effect of reinforcing members in terms of short steel pipes on PCR [23]. The welding of the pipes had a negligible influence on the failure of the substructure due to instability. Therefore, the authors concluded that such reinforcement does not improve the collapse resistance. All substructure investigations were considered fixed support conditions, and they do not replicate the real response of the spatial structures.

Numerical investigations of spatial structures revealed that the sensitivity of the members in single-layer forms was higher than that of multi-layer forms [16]. The major challenge in numerical analysis based on the APM is to initiate local collapse. Zhao et al. initiated the local collapse in FE models by lowering the value of Young's modulus based on a subroutine [10]. It was successful in replicating the results, except for the fracture of the members during the failure. Based on extensive numerical analysis, Yan et al. defined the criticality of members in a spatial structure [24]. Tian et al. found that the optimum span-to-rise ratio was 5 for Kiewitt domes when PCR and steel use were considered [17]. The authors observed that the PCR increases with decrease in the span-to-rise ratio; however, material consumption also increases with the decrease in the span-to-rise ratio of the structure.

A recent study developed a Finite Particle Method (FPM) based framework for analysing the seismic progressive collapse of single-layer lattice domes [25]. This study highlighted the effects of span-to-rise ratios and imperfections on progressive collapse. Another study on suspended domes identified the effect of cable rupture on progressive collapse when cable rupture transforms the structure into a single-layer lattice dome [26]. Compression members with steel bar constraints can improve the progressive collapse resistance of single-layer lattice domes [27]. Another important parameter that affects the progressive collapse resistance of single-layer domes is the joint rigidity of the structure. Domes with triangular grids that have welded joints exhibit a higher progressive collapse resistance than those with quadrilateral grids that have fabricated joints [28]. This study highlights the need to consider joint behaviour in a collapse-resistant design. Recent research on the blast performance of single-layer reticulated domes identified four collapse modes under explosion loads. Research distinguishes between dynamic instability and strength failure mechanisms in similar structures [29]. In another study, a novel node-buckling mechanism was proposed to explain the progressive collapse in suspended domes when subjected to cable rupture. This research emphasised the influence of geometric imperfections and support conditions on stability [30]. Recent studies focus mainly on improving the resistance to progressive collapse in spatial structures. One such study evaluated the progressive collapse resistance of hybrid cable domes and proposed four retrofit

strategies using alternate path analysis [31].

The investigations on the progressive collapse of single-layer lattice domes were primarily based on numerical analysis using the APM after validation with an experimental investigation of scaled-down models. Most of the research was on Kiewitt domes and very few studies considered different configurations, such as geodesic domes and three-way diagonal grids. One study considered different single-layer dome configurations to identify critical members based on the APM [24]. However, the study did not examine the effect of the arrangement of the members on the resistance to collapse or the sensitivity of the members. In fact, no investigations have been conducted so far that have compared the impact of member arrangement on the PCR of single-layer spatial structures. Therefore, the present study considers the effect of member arrangement or the topology of the structure on the PCR of single-layer lattice domes. The parameters of structures can be introduced into the identification of the spatial structure configuration [32]. In this research, the critical topological parameters that define a configuration are identified on the basis of the crystal lattice formalism. These parameters serve as indicators of structural configuration and help to identify alternate load paths during local failure. The objective of this paper is to examine how critical topological parameters affect the PCR of the structures. This study acts as a foundation for understanding structural robustness based on topology. The results of this study help to provide a novel perspective for improving the progressive collapse resistance of single-layer spatial structures based on geometric and topological design.

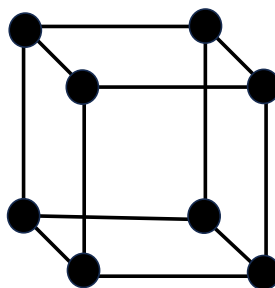
2. Identification of Critical Topological Parameters

The internal member arrangement or internal structure of single-layer lattice dome structures plays a crucial role in their behaviour and PCR. The interconnection and orientation of members are crucial in assessing overall stability and resilience against progressive collapse in such structures. The first step in configuring the spatial structures is to establish the connectivity of the members. Assessing and comparing performance becomes more straightforward once the connectivity of members is defined. The connectivity between members can be defined on the basis of some unique parameters. Therefore, the most important step in defining a configuration is to identify those critical parameters.

The arrangement of members and connections in spatial structures is similar to the atoms or molecular arrangements in the lattices (Figure 2). The lattice denotes a systematic organisation of points within a vector space. In spatial structures, the arrangement of the members and the structural connections repeat some definite pattern within the structure (Figure 3).



(a) A basic crystal unit cell containing eight atoms



(b) Lattice structure illustrating the unit cell

Figure 2: A crystal lattice representation of a molecule with eight identical atoms. Crystal lattices provide a framework for the geometric arrangement of atoms or molecules as spatial points.



Figure 3: Arrangement of structural members and connections following a regular pattern in a spatial structure. Image reproduced with permission from Geometrica Inc.

Similar to crystal structures, the organisation of nodes and members in spatial structures can also be explained as a combination of lattices and basis. Diverse arrangements of members and connections for spatial structures are possible, drawing inspiration from crystal structures. Once a specific pattern is selected, it can be mapped into the ‘structural form’ (a surface where the members are to be positioned) to achieve the desired configurations. Similar to the positioning of molecules or atoms in crystals to minimise the forces acting between them, each member in a reticulated dome configuration is positioned to ensure that member stress remains within acceptable limits while reducing the number of connections between them. The placement of members and connections in a configuration is optimised to ensure the stability of the resulting structure. Therefore, different concepts related to lattices can be brought into the configuration of spatial structures.

The repeating patterns or unit cells present in different spatial structural configurations can be analysed on the basis of mathematical relationships. The interrelation among elements—vertices, edges, faces, and cells—can be defined based on the parameters of a structure. Loeb proposed three key parameters—Dimensionality, Valency, and Extent—to express the internal structure of a configuration [32]. The fundamental goal behind utilising these parameters is to differentiate between diverse configurations through a distinctive nomenclature and evaluate the performance under gravity loading. Dimensionality explains the degree of freedom of an element in a structure. Valency provides the number of elements directly connected to an element with another dimensionality. Extent explains the measurable quantities such as length. Among these parameters, valency provides connectivity in a spatial structure.

Each element can have a valency with any other element, resulting in many valencies within a structure (Figure 4). Here, the ‘vertex valency of a face’ and the ‘face valency of a vertex’ are not the same. Different notations are provided for all valencies in a structure for easy identification (Table 1). Some of the valencies from Table 1 will be critical for dome structure, which can act as basic parameters for configuration identification and structural performance [33, 34].

Table 1: Representation of valencies in a structure

Type of valency	Vertex	Edge	Face	Cell
Vertex Valency	-	2	F_0	C_0
Edge Valency	V_1	-	F_1	C_1
Face Valency	V_2	E_2	-	C_2
Cell Valency	V_3	E_3	2	-

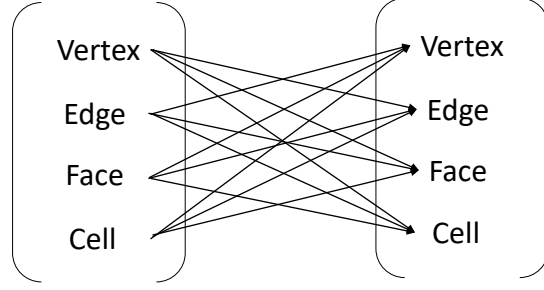


Figure 4: Different combination of elements results in different valencies in a configuration

The number of edges in a structure can be obtained from different valencies related to the edges. The sum of the ‘edge valency of vertex’ is equal to the sum of ‘vertex valency of edge’. This is mathematically represented as:

$$\sum_{r=2}^n rN_0^r = \sum_{s=2}^n sN_1^s \quad (1)$$

As the vertex valency of an edge is always two, the equation can be rewritten as follows:

$$\sum_{r=2}^n rN_0^r - 2N_1 = 0 \quad (2)$$

The valency of a structure varies within the structure (Figure 5). The average valency can be defined to incorporate this change. For example, the average value of ‘edge valency of vertices’ is given below:

$$\bar{v}_1 = \sum_{r=2}^n \frac{rN_0^r}{N_0} \quad (3)$$

The average value of the valency of different elements can be obtained in a similar way. These are represented below:

$$\bar{e}_2 = \sum_{t=2}^n \frac{tN_1^t}{N_1} \quad (4)$$

$$\bar{f}_1 = \sum_{u=3}^n \frac{uN_2^u}{N_2} \quad (5)$$

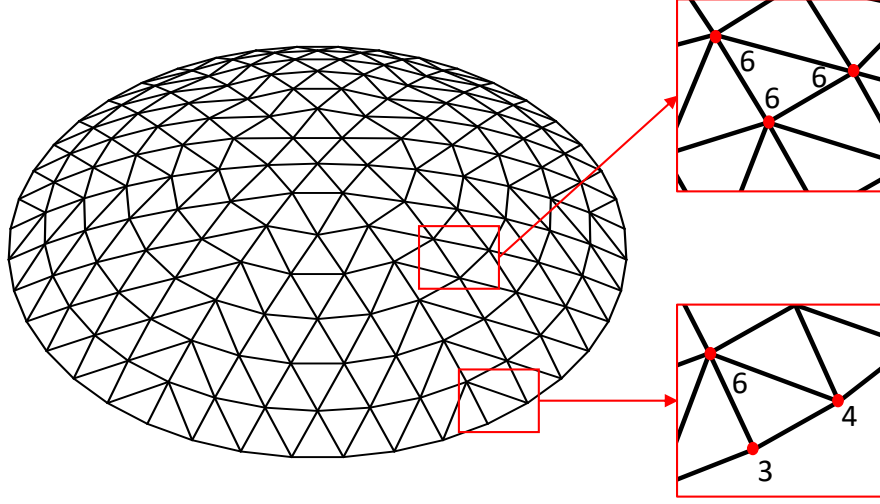


Figure 5: The variation of edge valency of vertices in a single-layer lattice dome

$$\bar{c}_1 = \sum_{x=6}^n \frac{xN_3^x}{N_3} \quad (6)$$

$$\bar{f}_0 = \sum_{u=3}^n \frac{uN_2^u}{N_2} \quad (7)$$

$$\bar{v}_2 = \sum_{y=2}^n \frac{yN_0^y}{N_0} \quad (8)$$

$$\bar{c}_0 = \sum_{b=6}^n \frac{bN_3^b}{N_3} \quad (9)$$

$$\bar{v}_3 = \sum_{a=2}^n \frac{aN_0^a}{N_0} \quad (10)$$

$$\bar{c}_2 = \sum_{g=4}^n \frac{gN_3^g}{N_3} \quad (11)$$

From equations 2 and 3, the following relation can be derived:

$$\bar{v}_1 N_0 - 2N_1 = 0 \quad (12)$$

Equation 12 connects the ‘average edge valency of vertices’ ($\bar{v}_1$), number of vertices (N_0), and number of edges (N_1).

Similarly, ‘face valency of edges’ and ‘edge valency of faces’ are connected by the following equation:

$$\sum_{t=2}^n tN_1^t = \sum_{u=3}^n uN_2^u \quad (13)$$

Using Equations 4 and 5, the above equation can be rewritten as:

$$\bar{e}_2N_1 - \bar{f}_1N_2 = 0 \quad (14)$$

Equation 14 helps to connect the ‘average edge valency of faces’ ($\bar{e}_2$), ‘average face valency of edges’ ($\bar{f}_1$), number of edges (N_1) and number of faces (N_2).

In a similar way, the following relations can be derived on the basis of the different valencies. The detailed derivation is provided by Kolakkattil et al. [34].

$$\bar{e}_2N_1 - \bar{c}_1N_3 = 0 \quad (15)$$

$$\bar{v}_2N_0 - \bar{f}_0N_2 = 0 \quad (16)$$

$$\bar{v}_3N_0 - \bar{c}_0N_3 = 0 \quad (17)$$

$$\bar{c}_2N_3 - 2N_2 = 0 \quad (18)$$

From equations 14, 15, and 16,

$$\bar{e}_2N_1 = \bar{f}_1N_2 = \bar{c}_1N_3 = \bar{v}_2N_0 \quad (19)$$

Expressing N_1 , N_2 , and N_3 in terms of N_0 :

$$N_1 = \frac{\bar{v}_2N_0}{\bar{e}_2} \quad (20)$$

$$N_2 = \frac{\bar{v}_2N_0}{\bar{f}_1} \quad (21)$$

$$N_3 = \frac{\bar{v}_2N_0}{\bar{c}_1} \quad (22)$$

The Euler-Schläfli relation defines the interrelationship among the various elements within a configuration. The general form of the Euler-Schlaefli relation is provided as:

$$\sum_{i=0}^j (-1)^i N_i = 1 + (-1)^j \quad (23)$$

Here, N_i indicates the number of elements having dimensionality i , and j indicates the number of dimensions in a structure. Equation 23 is modified for a three-dimensional structure, such as spatial structure, as:

$$N_0 - N_1 + N_2 - N_3 = 0 \quad (24)$$

Substituting Equations 20, 21, and 22 into Equation 24,

$$\frac{1}{\bar{v}_2} - \frac{1}{\bar{e}_2} + \frac{1}{\bar{f}_1} - \frac{1}{\bar{c}_1} = 0 \quad (25)$$

Furthermore, the following expression can be obtained from Equations 12, 14, and 16:

$$\bar{v}_1 \bar{e}_2 - 2\bar{v}_2 = 0 \quad (26)$$

Equation 26 can be modified when single-layer spatial structures are considered as two-dimensional surfaces, where the value of $\bar{e}_2$ is 2.

$$\bar{v}_1 - \bar{v}_2 = 0 \quad (27)$$

From Equation 15,

$$N_1 - \bar{c}_1 = 0 \quad (28)$$

Therefore, equation 25 is modified based on the above results.

$$\frac{1}{\bar{v}_1} + \frac{1}{\bar{f}_1} - \frac{1}{N_1} - \frac{1}{2} = 0 \quad (29)$$

As N_1 is a constant for a spatial structure, $\bar{v}_1$ and $\bar{f}_1$ are the critical parameters that affect a configuration. They are the fundamental parameters that define a configuration and its structural performance, also known as Critical Topological Parameters (CTPs). The influence of member arrangement, defined on the basis of these parameters, on the PCR of single-layer lattice domes is primarily investigated in this research.

3. Influence of CTPs on PCR

CTPs primarily indicate member connectivity within a configuration, and their value varies with different dome configurations. The member connectivity defined on the basis of CTPs can play a significant role in the PCR of dome configurations. Therefore, different single-layer lattice domes are created, and their collapse resistance is evaluated on the basis of the APM.

In the initial stage, three single-layer lattice domes with different CTPs are selected. Different span-to-rise ratios are also considered while modelling the lattice domes to evaluate the applicability of the results across various span-to-rise ratios of the structure. These dome structures are subjected to alternate path analysis using a general-purpose finite element package in the second stage. The PCR and the influence of CTPs are compared in the final stage.

3.1. Modelling of single-layer lattice domes

Progressive collapse analysis of single-layer lattice domes can be evaluated on the basis of extensive numerical analysis with variations of different parameters. Numerical analysis is the best solution for spatial structures, as full-size experimental investigations are rather impossible for such large-span structures [4]. The present study adopted numerical analyses after validating the experimental investigation of the scaled-down model. General-purpose software Abaqus [35] is used to model different single-layer lattice dome configurations. The members in these structures are modelled as linear elements with rigid connections between the members.

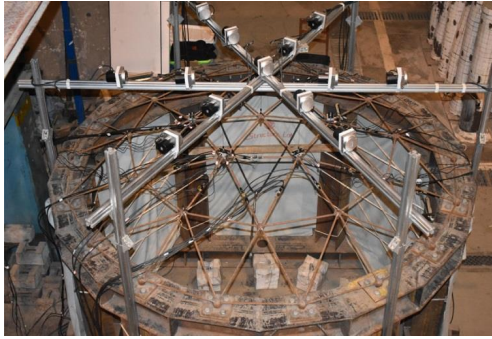
One of the primary challenges of conducting the numerical analysis, in identifying the PCR, is the creation of a member collapse or a local collapse in a structure. The member collapse simulated while doing the numerical analysis should resemble the one that occurs during the real collapse. Some investigations adopted a technique where subroutines, such as VUSDFLD, helped to create local collapse by reducing the modulus of elasticity of the material over time [10, 24]. The implicit time integration solver was adopted for the initial static condition after the load application, and the explicit time integration solver was adopted for the dynamic progressive collapse analysis, where VUSDFLD was used. The model change option in Abaqus can also be used for member deletion or local failure in spatial structures [9]. The element birth-death method was adopted in some progressive collapse investigations of single-layer Kiewitt dome [11, 12, 17].

In this research, local collapse is created by removing the member using the model change option in Abaqus, assuming that the entire member is disconnected from the dome structure. The weight of one member is negligible compared to the total weight of the dome. Therefore, the removal of one member does not create much variation

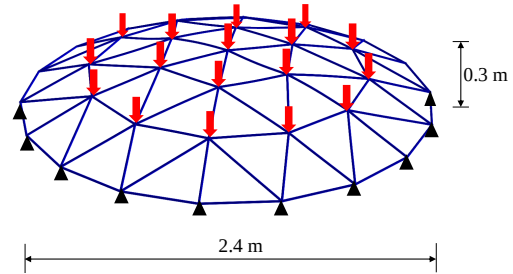
in the total weight of the structure. The member collapse method adopted in this investigation is validated using the experimental results of Tian et al. [11].

3.1.1. Validation of numerical model

The experiment carried out by Tian et al. [11] to compare the progressive collapse of dome structures is used to validate the numerical model. Tian et al. evaluated PCR using a scaled-down model of the Kiewitt-6 dome that has a span of 2.4 m and a span-to-rise ratio of 8 (Figure 6). The construction of the dome structure employed steel members with a tubular cross-section, each having a diameter of 10 mm and a thickness of 1 mm. The members were interconnected with rigid connections, and both rotational and translational degrees of freedom were entirely constrained by the supports. A hanging weight of 40 kg was introduced at each node to represent both live load and dead load. Member-breaking devices were incorporated to induce local failure (i.e., member removal), helping to monitor the potential progressive collapse. In the course of the experiment, the removal of four members was considered consecutively, as shown in Figure 7. The removal of the initial three members did not trigger progressive collapse, but the removal of the fourth member led to the complete collapse of the Kiewitt dome during the experiment.



(a) Experimental set-up of Kiewitt-6 dome [11]



(b) Finite element model in Abaqus CAE

Figure 6: The finite element model is validated with the help of the experimental investigation on Kiewitt-6 dome by Tian et al. [11]. Figure (a) reproduced with permission.

The finite element model for the Kiewitt-6 dome was generated in Abaqus CAE using linear elements 'B31' for the members. The connections between the members were assumed to have complete rigidity, and supports were pin-connected (Figure 6b). The average values of the mechanical properties of the five specimens derived from the coupon test, by Tian et al. [11], were utilised to generate the constitutive

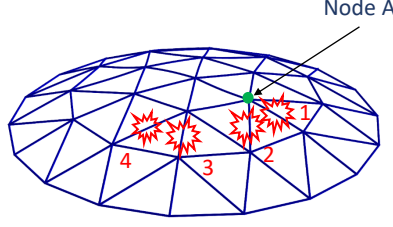


Figure 7: The sequence of member removal used in both the experiment and numerical analysis. The vertical deflection of node A is measured and plotted to validate the finite element model

Table 2: Material properties of the specimens utilised in the progressive collapse experiment conducted by Tian et al. [11]

Specimen Number	E (GPa)	f_y (MPa)	f_u (MPa)	ϵ_u
1	181	330	531	0.16
2	201	358	556	0.20
3	215	369	553	0.19
4	199	333	513	0.20
5	215	360	543	0.17
6	207	365	533	0.17

relation for the material (Table 2). Elasto-plastic behaviour with strain hardening, as shown in Figure 8, is used for the tubular steel sections. The dynamic implicit analysis is employed to assess the progressive collapse potential of the structure.

A damping ratio of 0.03 (Rayleigh damping coefficient) was adopted as per the methodology outlined by Tian et al. [11]. Nodal masses were used to represent the hanging weight provided in the experiment. The authors of the experiment did not specify the measurement of initial geometrical imperfections of the experimental model. Therefore, the numerical model adopted an initial geometric imperfection with a maximum magnitude of span/400 based on the eigenvalue imperfection mode method. The ‘model change’ technique in Abaqus CAE is adopted to create the local collapse in the dome structure. The member removal time was adjusted to match the corresponding value in the experimental setup.

The progressive collapse analysis of the experimental and numerical model was performed using the APM, where the effect of removal of each member was evaluated. The results obtained from the experiment and the numerical analysis indicated that the dome structure did not collapse even after removing the third member. Figure 9 indicates that the disproportionate collapse of the dome occurred after the removal of

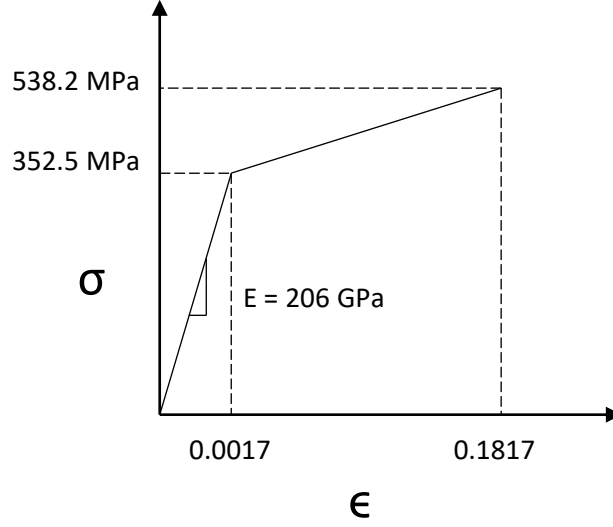


Figure 8: A bilinear elastoplastic constitutive relation is utilised for numerical modelling of the structure.

the fourth member. Therefore, the findings obtained from the numerical investigation align with the outcomes observed in the experimental study.

3.1.2. Configurations with different Critical Topological Parameters

Numerical analysis employing the APM is performed on various dome configurations to investigate the impact of CTPs on the progressive collapse of single-layer spatial structures. Figure 10 illustrates three configurations generated by varying edge valencies of vertices and faces. The nomenclature provided for each lattice dome is based on the parameterisation principles [33]. For example, $V.6^3$ indicates the edge valency of the vertices in the first configuration (Figure 10a).

In the APM, the symmetry of the structure is employed to reduce the number of analyses. Figure 11 shows the number of members identified to perform the alternate path analysis considering the symmetry of the structure. One by sixth for configuration $V.6^3$ and $V.3^6$ and one by twenty-fourth for configuration $V.4^4$ are selected while conducting the alternate path analysis. Five span-to-rise ratios are used for each configuration to identify the effect of the span-to-rise ratio on the PCR of a dome configuration. Therefore, a fixed span of 40 meters and span-to-rise ratios of 4, 5, 6, 7, and 8 are used.

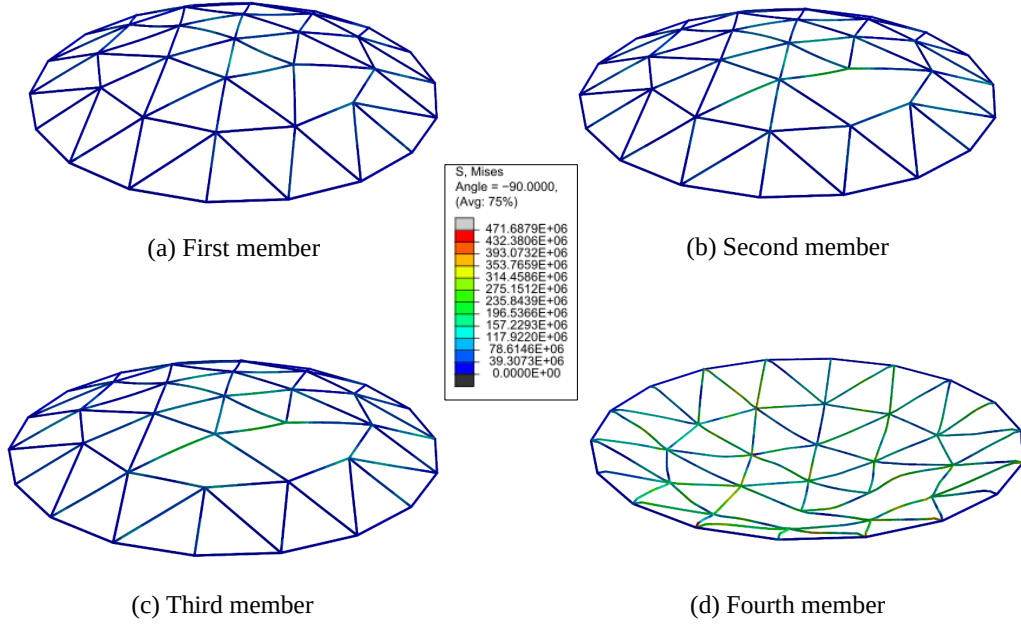
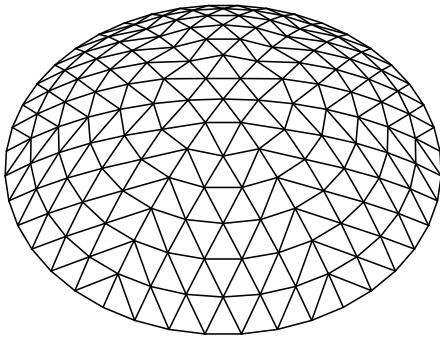


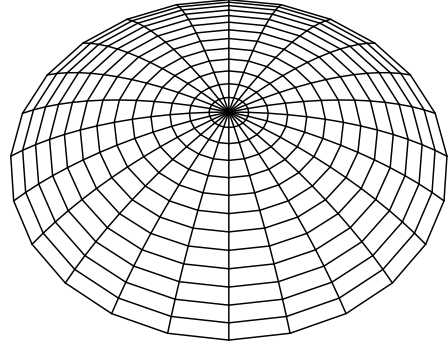
Figure 9: The deflected profile of Kiewitt-6 configuration observed during numerical analysis following the removal of members using the APM.

The members have circular hollow sections with an outer diameter of 89 mm and a thickness of 4 mm (referred to as CHS 80) following the specifications outlined in IS 1161 (1998) [36]. Rigid connections were modelled between members, while the supports were modelled as pinned connections. A bilinear elasto-plastic material relationship with strain hardening was adopted for the material similar to the model used for numerical validation, with Young's modulus of 203 GPa, yield stress of 310 MPa, ultimate stress of 450 MPa, and ultimate strain of 0.18. The material properties for this constitutive relation were obtained from IS 4923 - 1997: Hollow Steel Sections for Structural Use-Specification [37]. All the numerical models adopted initial geometric imperfections with a maximum magnitude of span/400 based on the eigenvalue imperfection mode method.

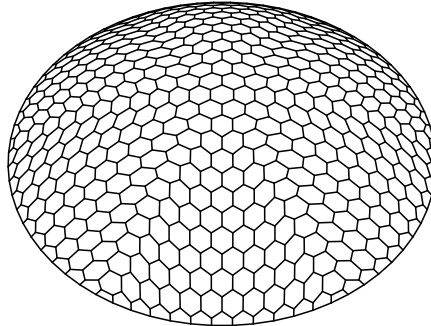
Before employing the APM, eigenvalue buckling analysis and Geometrically and Materially Nonlinear Analysis (GMNA) were performed. The deflected shape and the buckling mode shape were identified for each configuration to investigate potential correlations between highly stressed and deflected members and the sensitive members identified through the APM. The results for the dome structure having the span-to-rise ratio of 5 are shown here, as the results are similar across different span-to-rise ratios. Later, APM is conducted to assess the progressive collapse potential.



(a) $V.6^3$

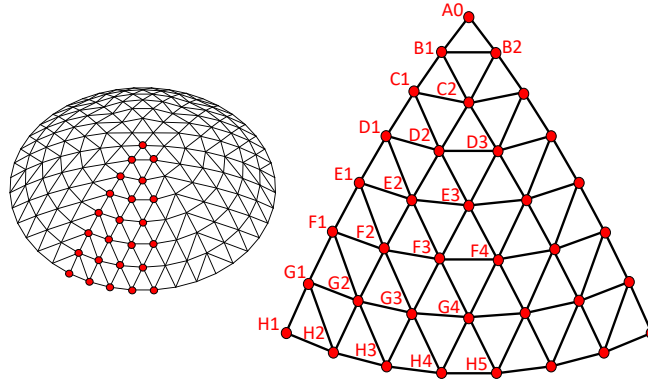


(b) $V.4^4$

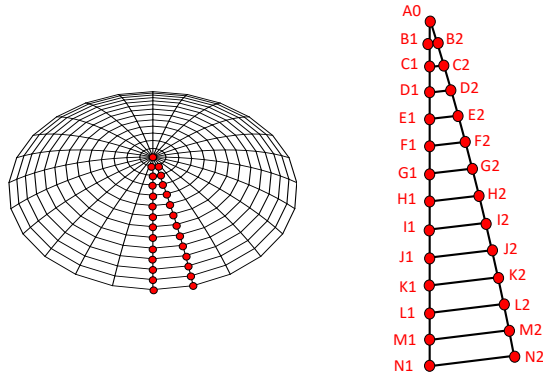


(c) $V.3^6$

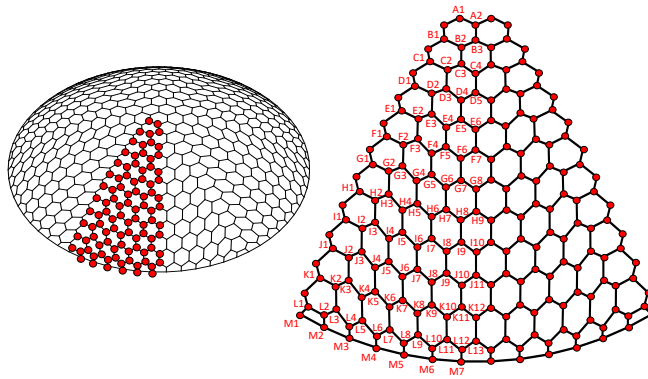
Figure 10: Three lattice dome configurations, each featuring distinct edge valencies of vertices and faces, were chosen to assess the potential for progressive collapse using the APM.



(a) $V.6^3$



(b) $V.4^4$



(c) $V.3^6$

Figure 11: The symmetry of the lattice dome structure is utilised while conducting the alternate path analysis. The nodes are numbered for easy identification of members during the analysis.

The failure of a single member is considered at a time for the three configurations.

3.2. *Progressive collapse analysis based on the APM*

The progressive collapse resistance of the three given configurations was evaluated based on the APM. The APM, suggested by several guidelines, such as GSA - 2013 [7]; ASCE/SEI 7-17 - 2017 [2]; UFC-DoD - 2016 [38], has become the standard approach for assessing the PCR of structures. Each of the three dome configurations, with varying CTPs, is individually analysed to assess its progressive collapse potential using the APM. Initially, static loading was applied to the dome configurations. The uniformly distributed loads are applied as a concentrated force at each node by projecting the loads onto the horizontal plane. The stressed skin effect is not considered in the present analysis, and the distributed loads are converted to point loads acting on the joints [39]. Point masses were applied to each connection to account for inertial effects during dynamic analysis. In the subsequent stage, a member is systematically removed from the structure using the model change method via the implicit dynamic analysis available in Abaqus. The time of local collapse is maintained at less than one-tenth of the fundamental frequency of the dome structure, as recommended by GSA [7].

During the numerical analysis, the load acting on the structure is increased at every step. For every load increase of 0.1 kN, the probability of progressive collapse is evaluated by conducting the alternate path analysis. Following the local collapse, the response of the single-layer reticulated dome structure is monitored, particularly the vertical displacement-time history of the adjacent node, to assess the possibility of progressive collapse. The load per node is increased at a rate of 0.1 kN for each case to calculate the progressive collapse load with great precision. This process is repeated for each member, and the configuration's symmetry is exploited to minimise the number of analyses required for each dome structure. Dome structures with different span-to-rise ratios were also considered for analysis. A flowchart showing the procedure for progressive collapse analysis is provided in Figure 12.

The results derived from the alternate path analysis for each dome configuration are detailed in the subsequent section. Additionally, the results of the eigenvalue buckling analysis and GMNA are included for comparison in each configuration.

3.3. *Results from APM*

The configuration $V.6^3$, also known as the Kiewitt-6 configuration, features triangulation with an 'average edge valency of vertices' of 5.47 and an 'average edge valency of the face' of 3 (Figure 10a). The results of the eigenvalue buckling analysis and the GMNA for the Kiewitt dome with the span-to-rise ratio of 5 are provided in Figure 13.

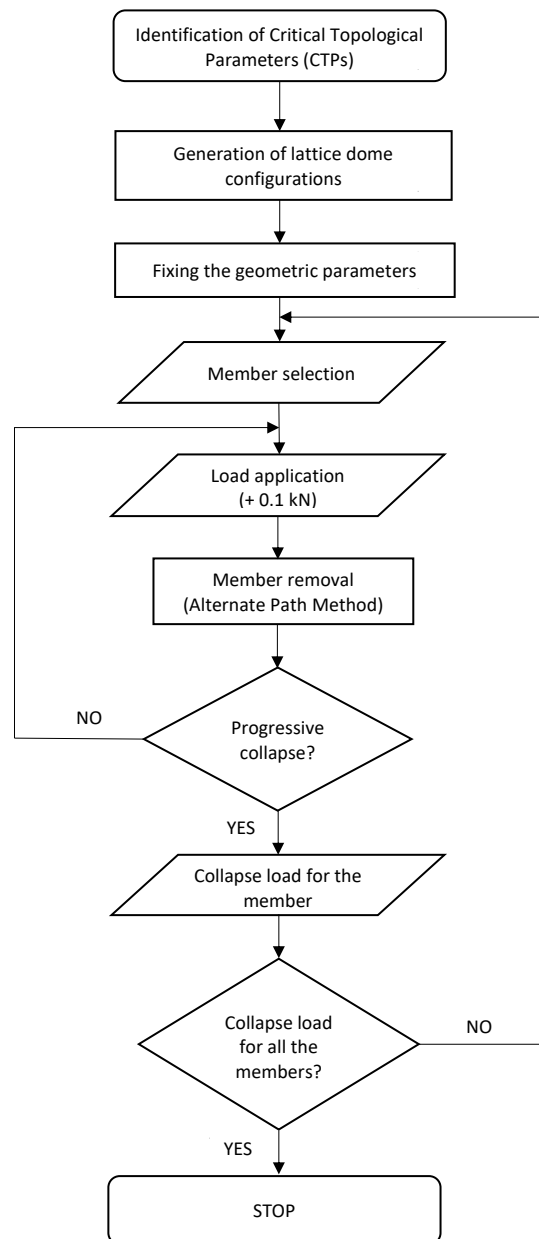
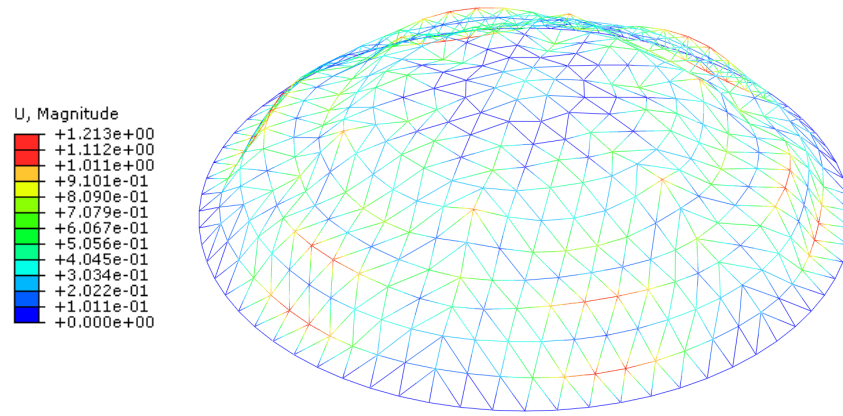
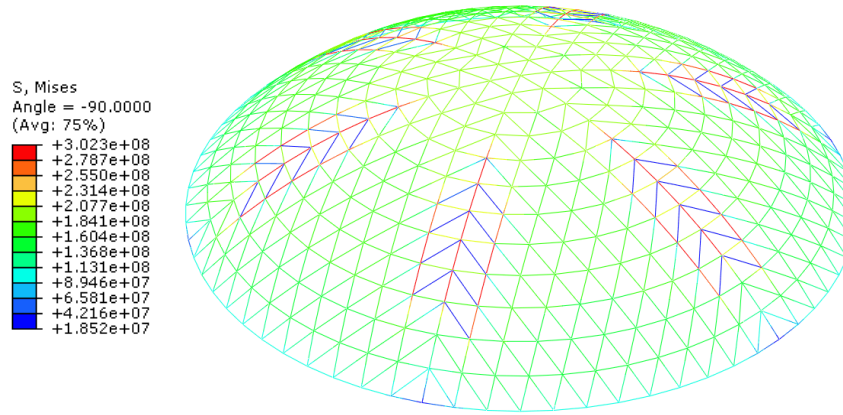


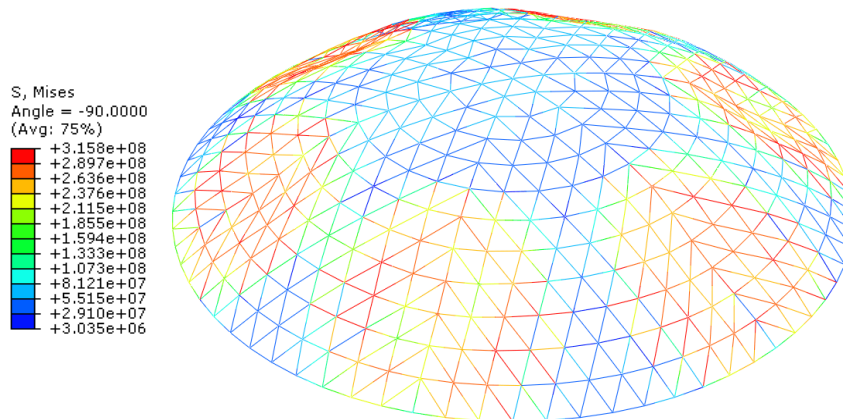
Figure 12: Flowchart showing the procedure for calculating the progressive collapse load for different members in single-layer lattice domes



(a) First buckling mode



(b) Stress distribution while applying 10% of critical load



(c) Stress distribution after reaching the critical load

Figure 13: The results from the eigenvalue buckling analysis and GMNA for configuration V.6³ are presented in the figure. The stress distribution is shown in N/m^2 .

The mode shapes from the eigenvalue buckling analysis indicate that the maximum deflection is observed at the centre of the symmetric portion (Figure 13a). The GMNA results show that the most stressed members are situated along the edge of the symmetry line of the configuration. The maximum deflection for the structure is also obtained for the same region during the numerical analysis. The stress distribution while 10 % of the critical load is provided to examine the load distribution in members during the initial stage. The results obtained for eigenvalue analysis and GMNA are compared with the results obtained from alternate path analysis so that the possibility of identifying the most sensitive elements from the conventional analysis, such as GMNA itself, can be explored. Similar findings reduce the dependency on the APM, which is very time-consuming for a structure such as a lattice dome with many elements.

The results obtained from the APM for the configuration V.6³ with different span-to-rise ratios are presented in Figures 14 to 18. The magnitude of the progressive collapse load decreases as the span-to-rise ratio is increased for V.6³ configuration (Figure 19). When the span-to-rise ratio is decreasing and approaching the value of 5 or 4, the magnitude of progressive collapse load increases when the location of local collapse moves towards the support within the configuration (Figure 20). This trend disappears as the span-to-rise ratio increases to 7 or 8. The variation in progressive collapse load with member removal reduces with a higher span-to-rise ratio of the structure. This observation highlights the importance of adopting a high span-to-rise ratio for the structure to achieve greater resistance to progressive collapse. Comparison of the buckling mode shape and the GMNA results with the sensitivity calculation of members using the APM revealed no similarity in the critical region of the lattice dome. Both the eigenvalue buckling mode and the deflection profile from GMNA failed to indicate the location of the sensitive members.

The results of the eigenvalue buckling analysis and the GMNA for configuration V.4⁴ are provided in Figure 21. The eigenvalue buckling analysis shows that the maximum deflected region is near the apex region. The GMNA shows that the radial members are highly stressed prior to failure and that the highly deflected region varies from location to location. Figures 21a and 21c indicate the significance of radial members in collapse resistance.

For configuration V.4⁴, the variation in the progressive collapse load due to different member removals is minimal (Figures 22 to 26). Similar to the first configuration, the collapse load decreases as the span-to-rise ratio of the structure increases (Figure 27). For span-to-rise ratios of 6 and 8, the collapse load remained consistent for all member removals (i.e., local collapse). The variance for the collapse load for the span-to-rise ratios 8 and 6 are less than 10^{-30} . For span-to-rise ratios 4, 5 and 7,

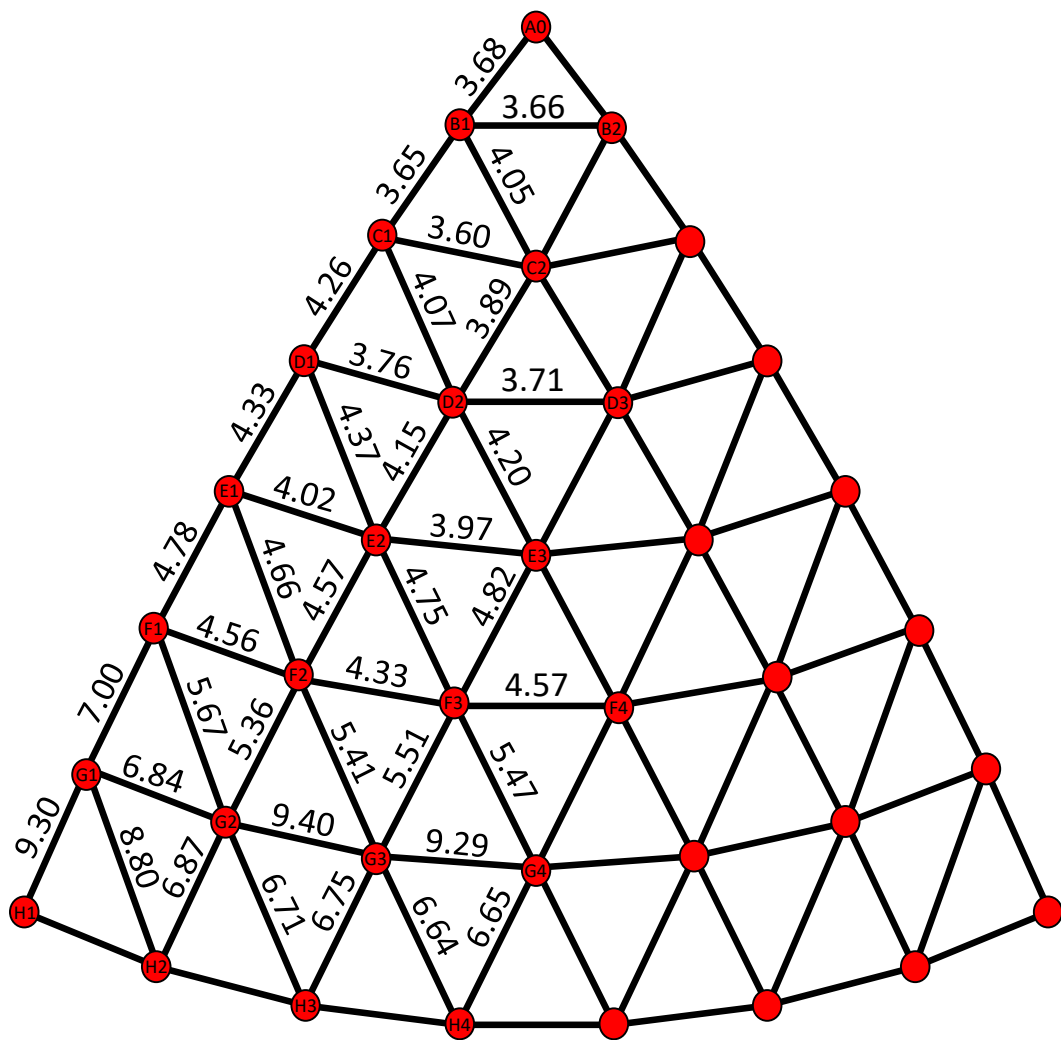


Figure 14: Progressive collapse load for each member based on alternate path analysis for the configuration $V.6^3$ with span-to-rise ratio of 4

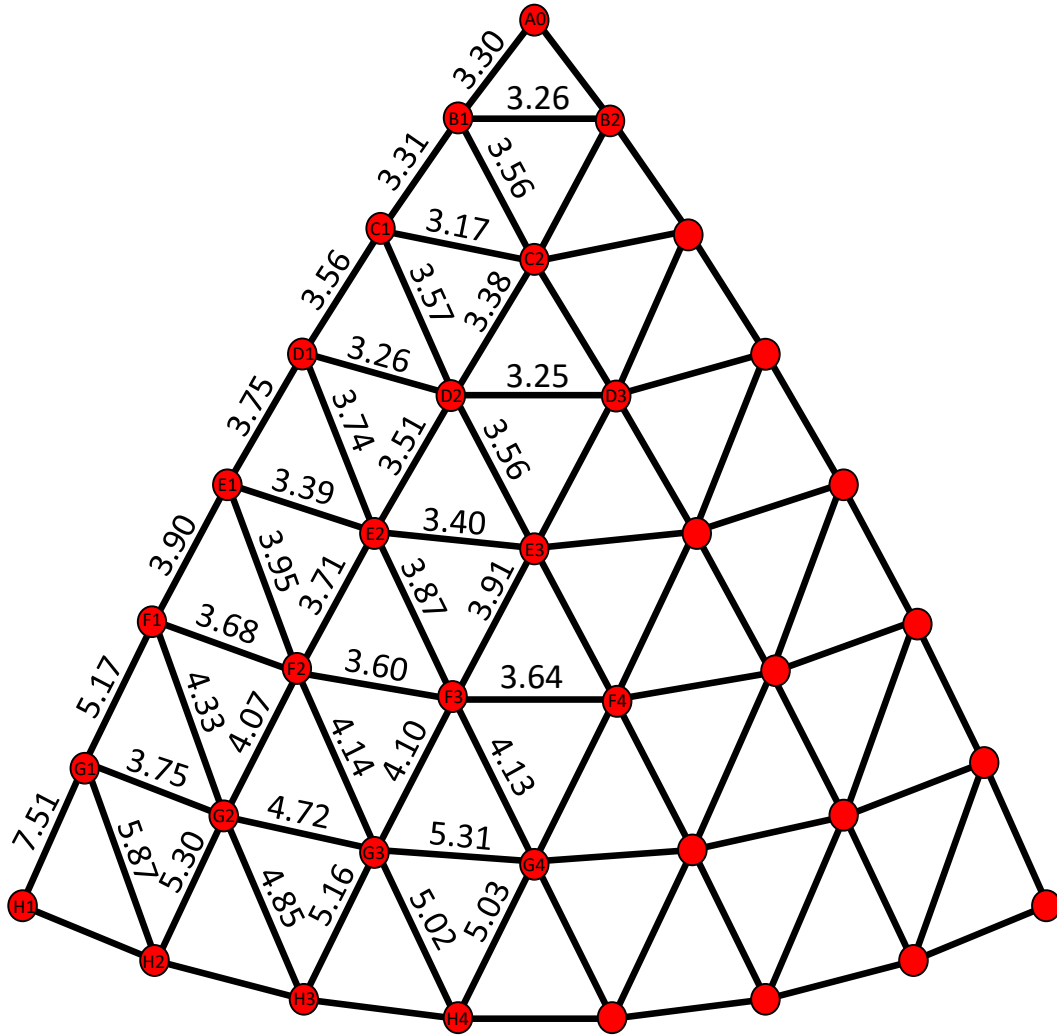


Figure 15: Progressive collapse load for each member based on alternate path analysis for the configuration V.6³ with span-to-rise ratio of 5

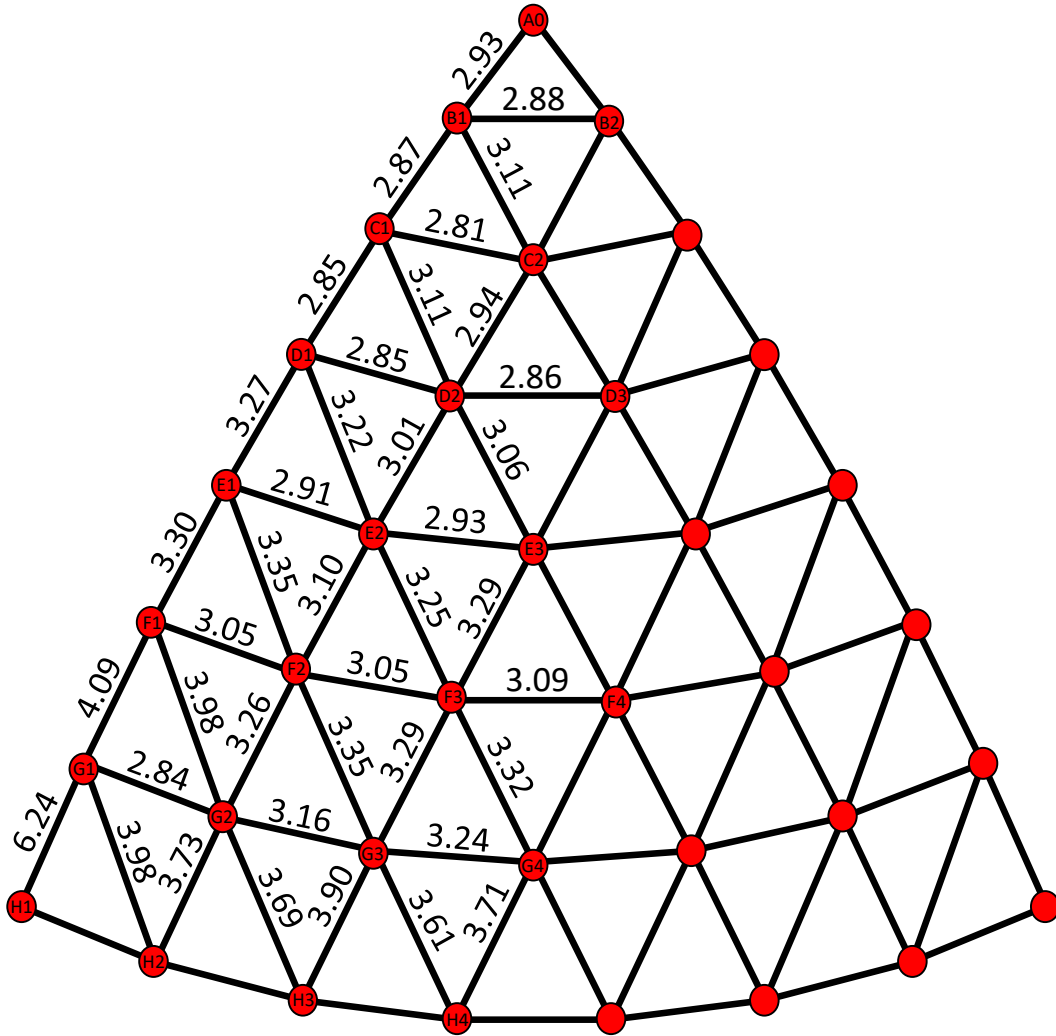
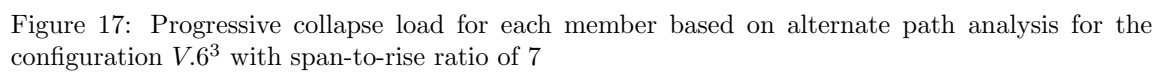
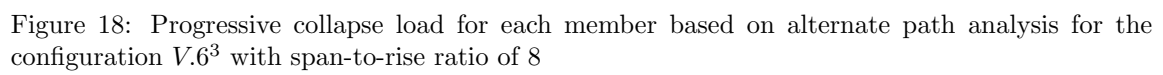


Figure 16: Progressive collapse load for each member based on alternate path analysis for the configuration $V.6^3$ with span-to-rise ratio of 6





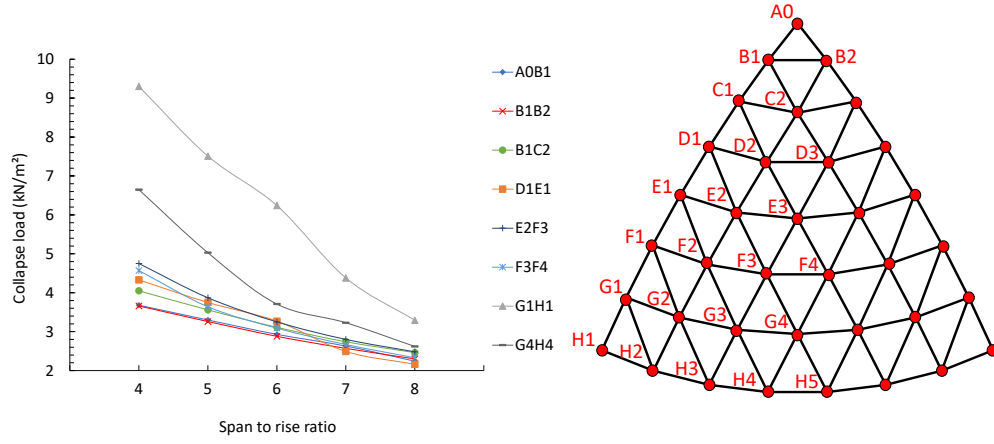


Figure 19: Variation in the collapse load with change in span-to-rise ratio for selected member failure in the configuration $V.6^3$

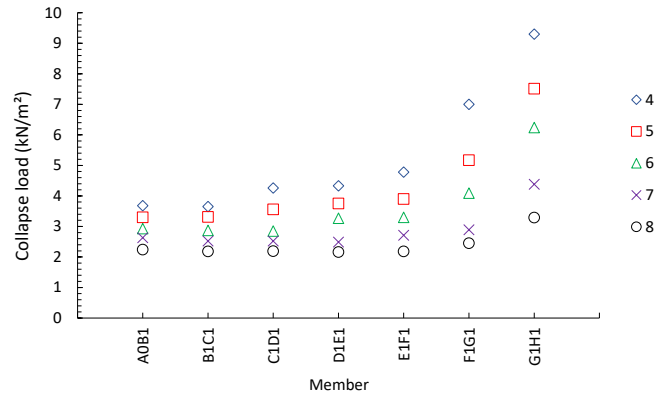
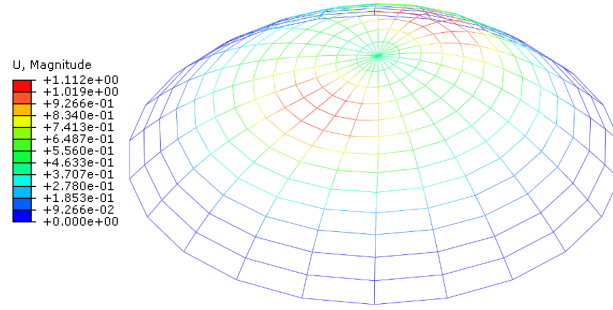
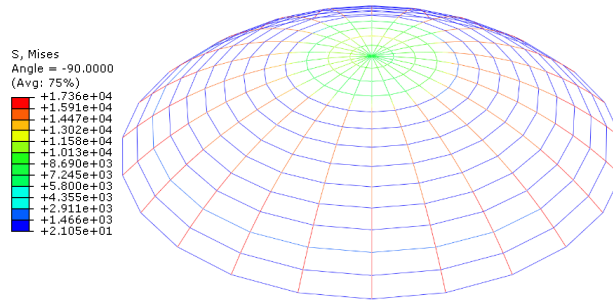


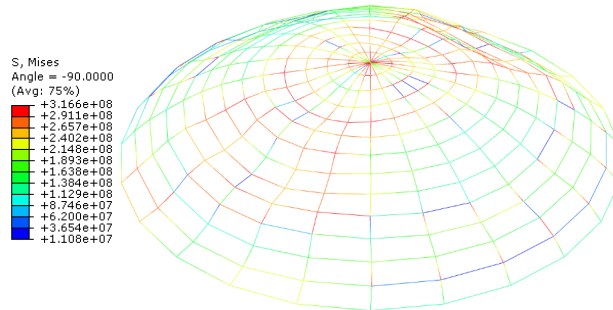
Figure 20: Variation in the collapse load for members from the apex to the support region for the $V.6^3$ configuration. Results are provided for different span-to-rise ratios.



(a) First buckling mode



(b) Stress distribution while applying 10% of critical load



(c) Stress distribution after reaching the critical load

Figure 21: The results from the eigenvalue buckling analysis and GMNA for configuration $V.4^4$ are presented in the figure. The stress distribution is shown in N/m^2 .

the progressive collapse load decreases from the apex of the dome and reduces to a minimum value, then increases towards the support (Figure 28). The magnitude of the variance varies from 0.0002 to 0.001 for all three cases. The collapse load for the radial members is lower compared to the other members when the span-to-rise ratio of the dome is lower. However, this variation diminishes as the dome's span-to-rise ratio increases. This indicates the critical role of radial members in resisting progressive collapse when the dome's span-to-rise ratio is lower. The arch action significantly contributes to collapse prevention when the dome's span-to-rise ratio is lower.

The load at which progressive collapse occurs is lower for this configuration compared to configuration $V.6^3$, according to the APM. The higher edge valency of vertices and the uniform distribution of the members in the configuration $V.6^3$ contribute to its greater resistance to progressive collapse.

The results of the eigenvalue buckling analysis and the GMNA did not identify the most sensitive members, as observed from the APM for the $V.4^4$ configuration. However, in similar configurations, the radial members are more sensitive to triggering progressive collapse compared to other members. This is evident from the stress distribution while applying the gravity load on the lattice dome (Figures 21b and 21c).

The results of the eigenvalue buckling analysis and the GMNA for configuration $V.3^6$ are presented in Figure 29. The unsymmetrical stress distribution during the early stage of load application is due to the initial geometrical imperfections applied in the dome configuration. The first buckling mode shows the highly deflected regions located at three distinct areas from the apex. The GMNA results indicate that significant deflection occurs in the mid-region between the apex and the supports.

For configuration $V.3^6$, the collapse load calculated using the APM decreases with an increase in the span-to-rise ratio of the structure (Figures 30 to 35). For the dome structure across all span-to-rise ratios, the collapse load decreases from the apex of the dome, reaches a minimum value, and then increases as it approaches the support region (Figure 36). These results are similar to the deflection profile of the structure based on GMNA. Therefore, conducting static analysis, such as GMNA, provides some guidelines for locating the sensitive members in the configuration $V.3^6$ where the edge valency of vertices is lower and the edge valency of faces is higher.

The PCR was anticipated to increase with the increase in the edge valency of vertices. The load at which progressive collapse initiates is higher for configuration $V.3^6$ in comparison to configuration $V.4^4$, but lower than for configuration $V.6^3$. Configuration $V.3^6$ has significantly more members and nodes than configuration $V.4^4$, while maintaining a comparable total length of members used in the configurations

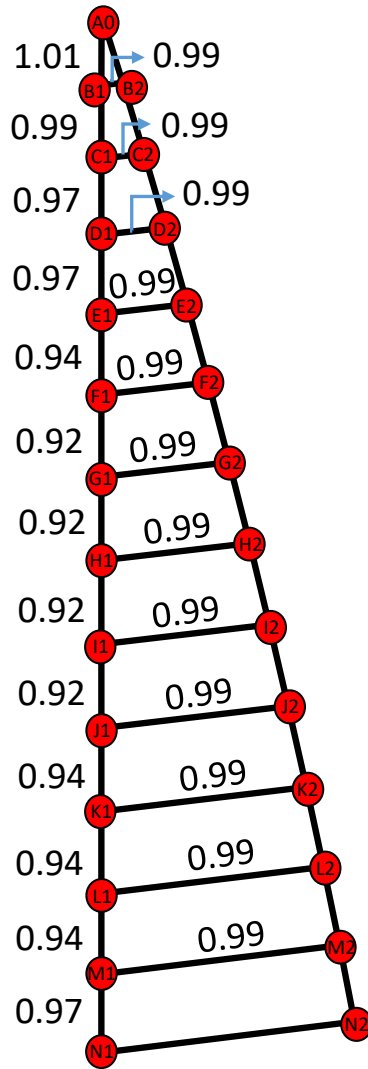


Figure 22: Progressive collapse load for each member based on alternate path analysis for the configuration $V.4^4$ with the span-to-rise ratio of 4

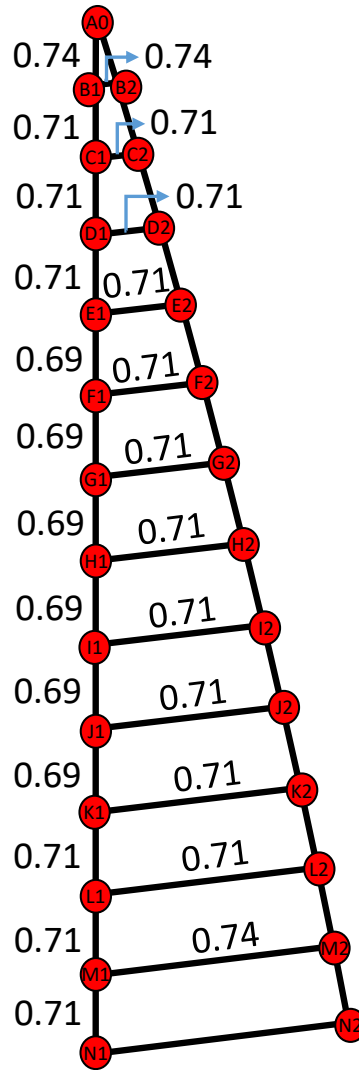


Figure 23: Progressive collapse load for each member based on alternate path analysis for the configuration $V.4^4$ with the span-to-rise ratio of 5

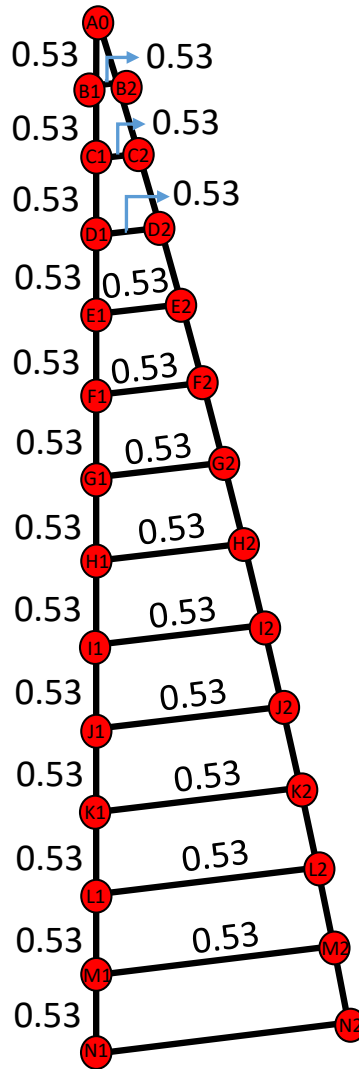


Figure 24: Progressive collapse load for each member based on alternate path analysis for the configuration V.4⁴ with the span-to-rise ratio of 6

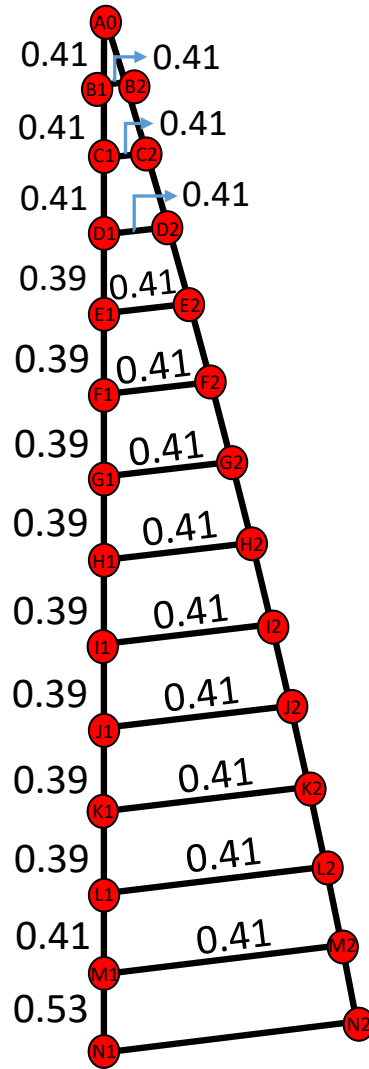


Figure 25: Progressive collapse load for each member based on alternate path analysis for the configuration $V.4^4$ with the span-to-rise ratio of 7

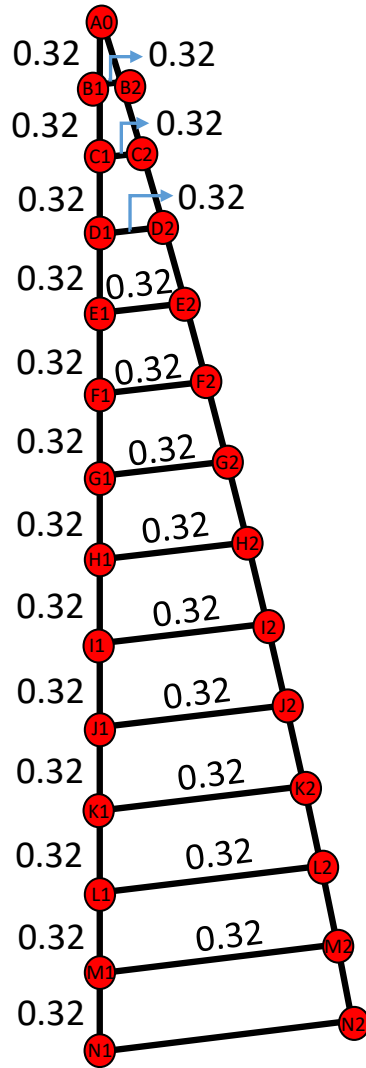


Figure 26: Progressive collapse load for each member based on alternate path analysis for the configuration V.4⁴ with the span-to-rise ratio of 8

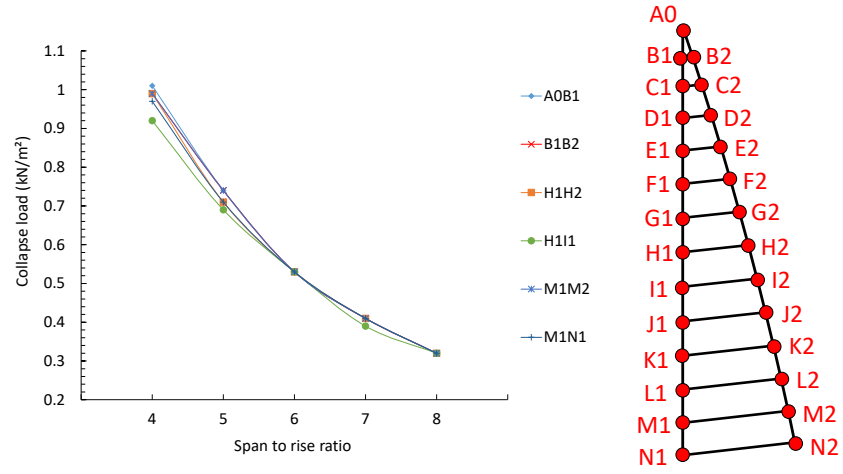


Figure 27: Variation in the collapse load with change in span-to-rise ratio for selected member failure in the configuration V.4⁴

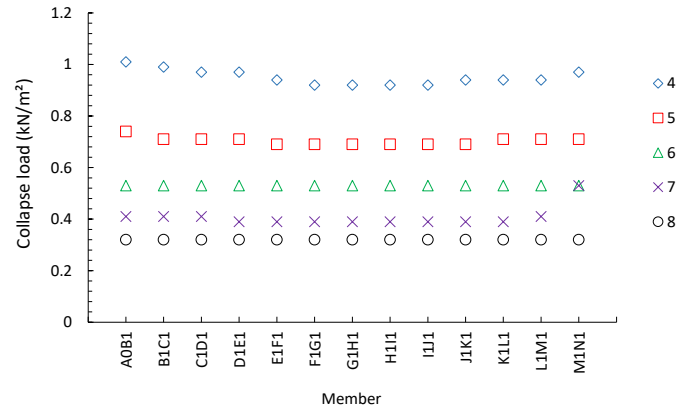
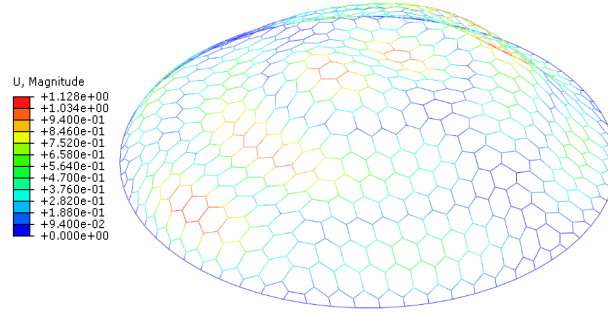
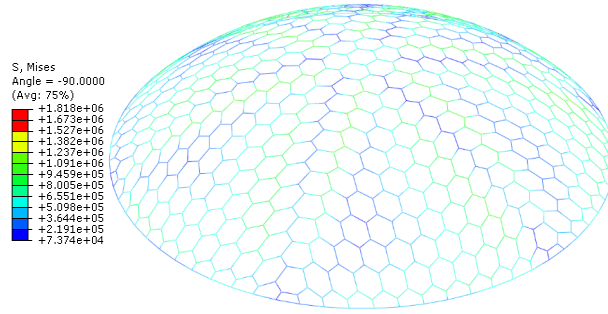


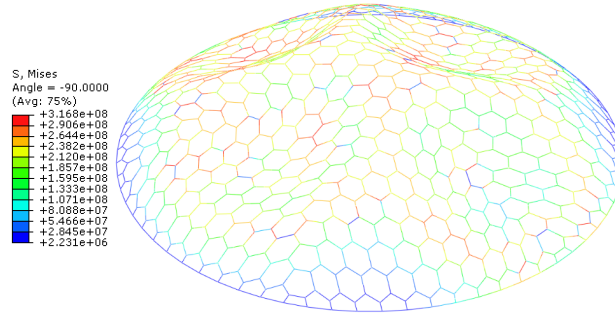
Figure 28: Variation in the collapse load for members from the apex to the supports for the V.4⁴ configuration. Results are provided for different span-to-rise ratios.



(a) First buckling mode



(b) Stress distribution while applying 10% of critical load



(c) Stress distribution after reaching the critical load

Figure 29: The results from the eigenvalue buckling analysis and GMNA for configuration $V.3^6$ are presented in the figure. The stress distribution is shown in N/m^2 .

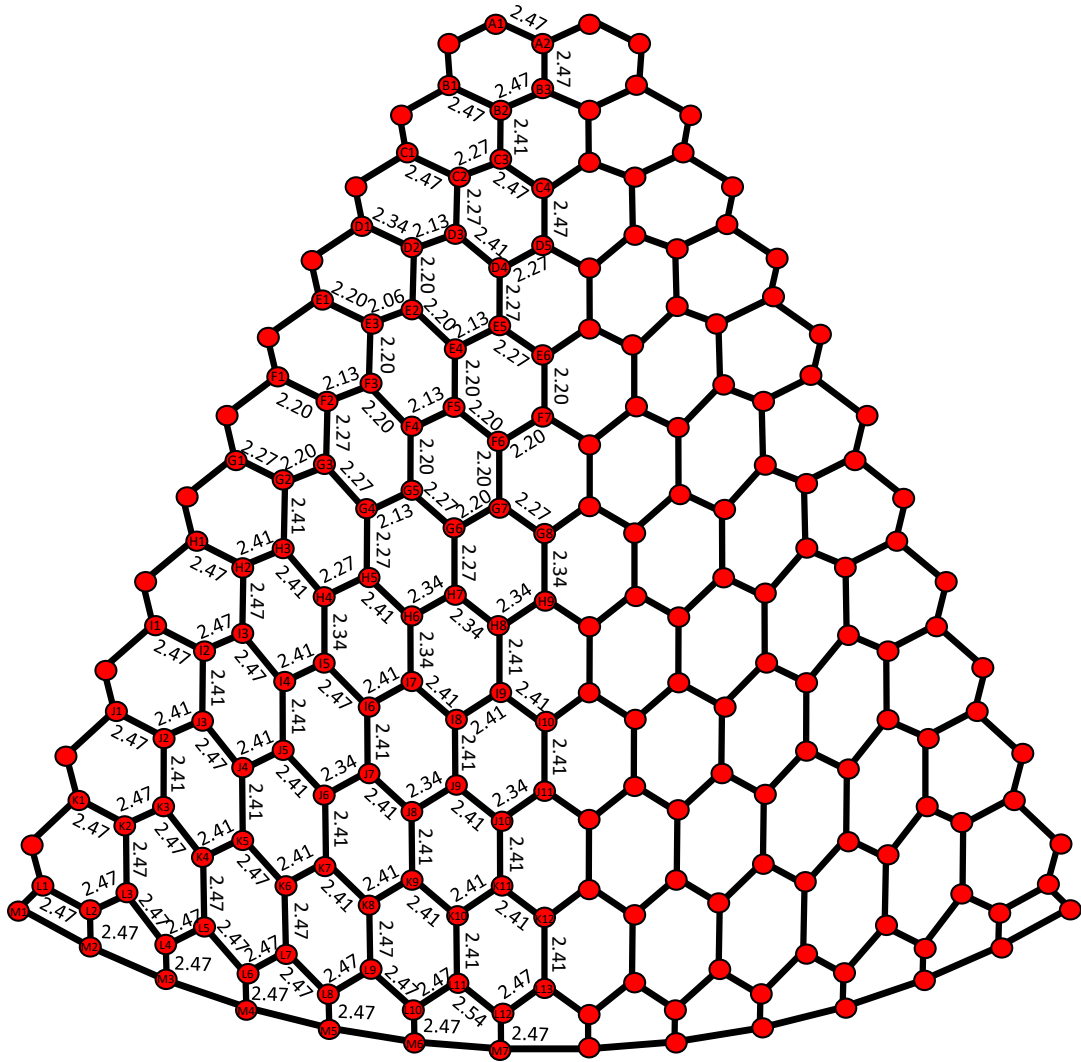


Figure 30: Progressive collapse load for each member based on alternate path analysis for the configuration V.3⁶ with the span-to-rise ratio of 4

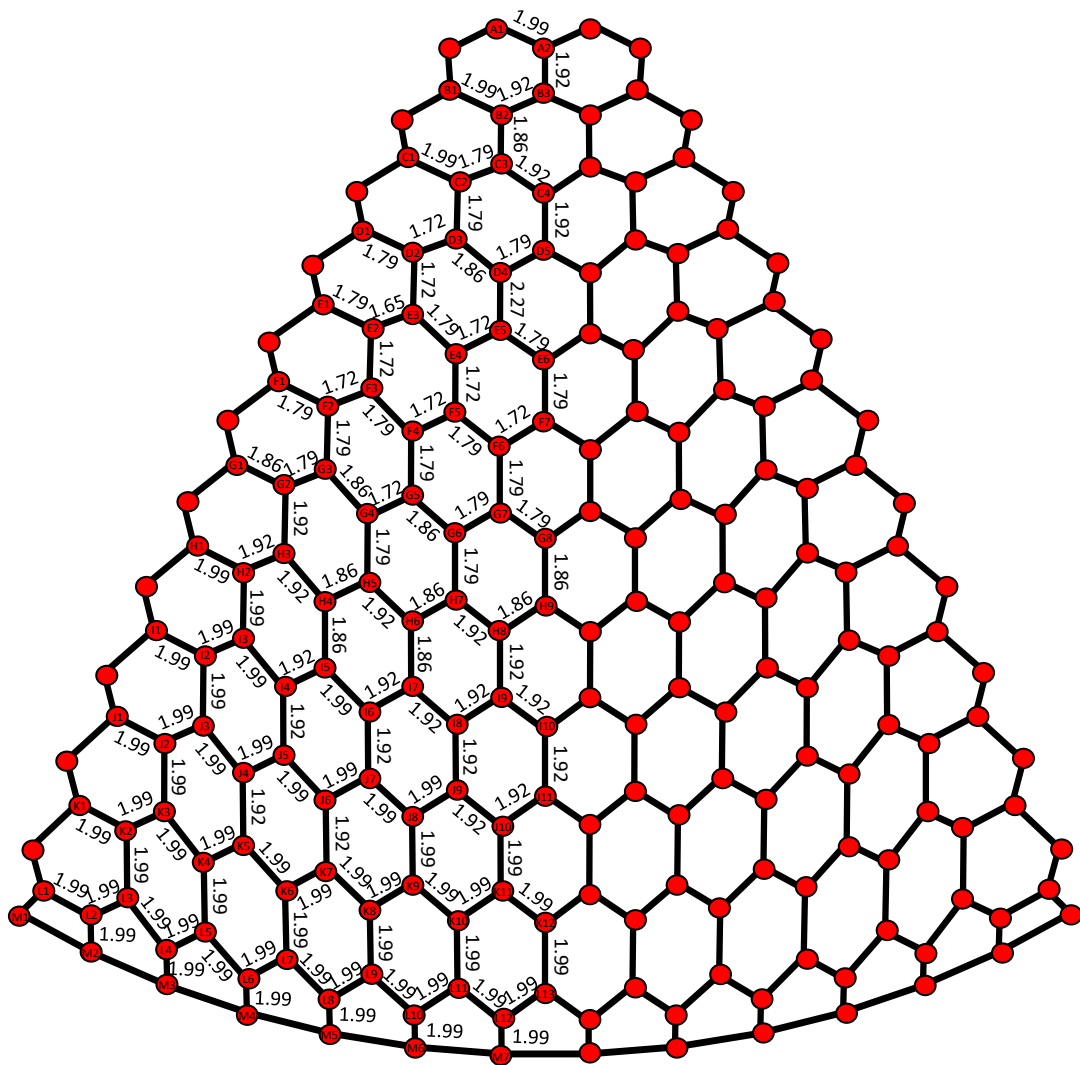


Figure 31: Progressive collapse load for each member based on alternate path analysis for the configuration V.3⁶ with the span-to-rise ratio of 5

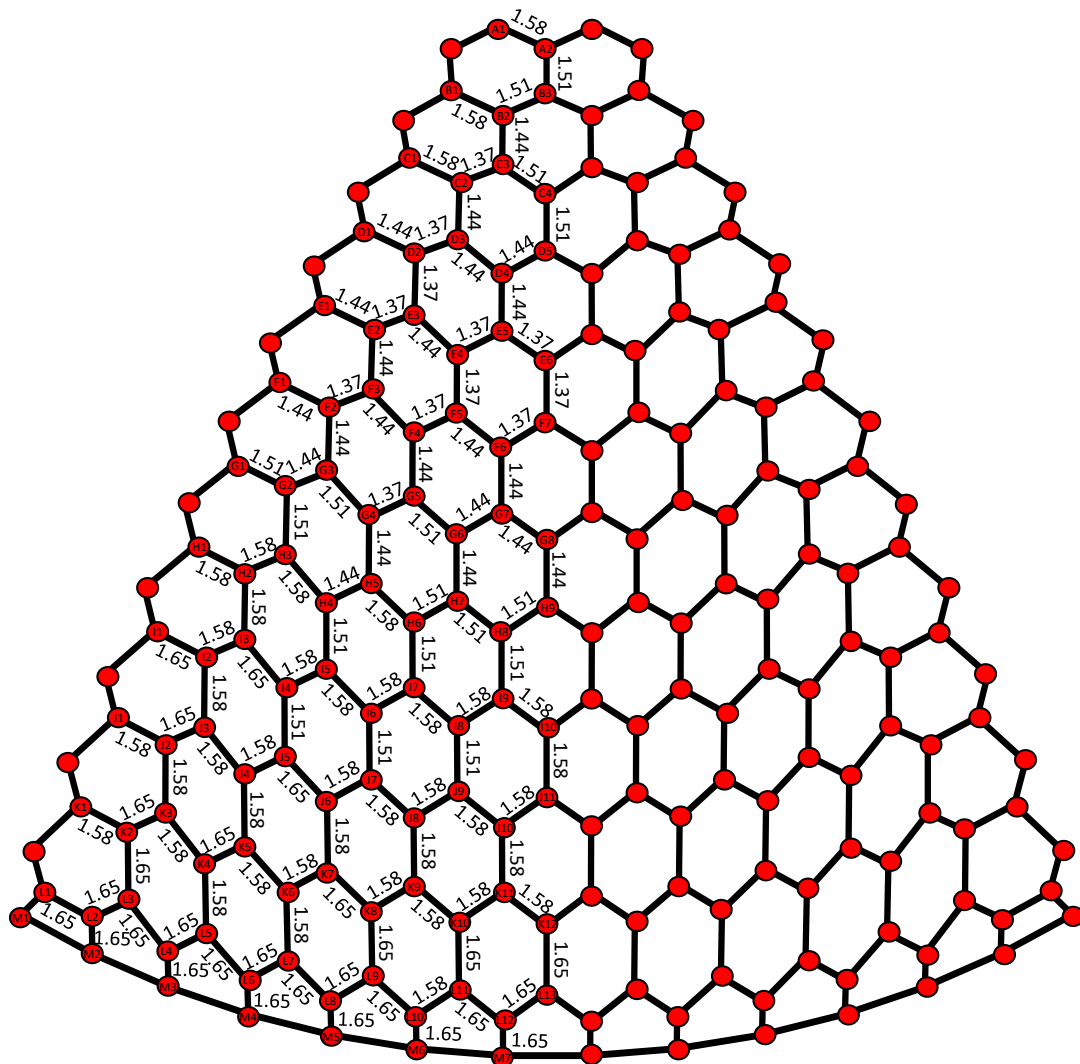
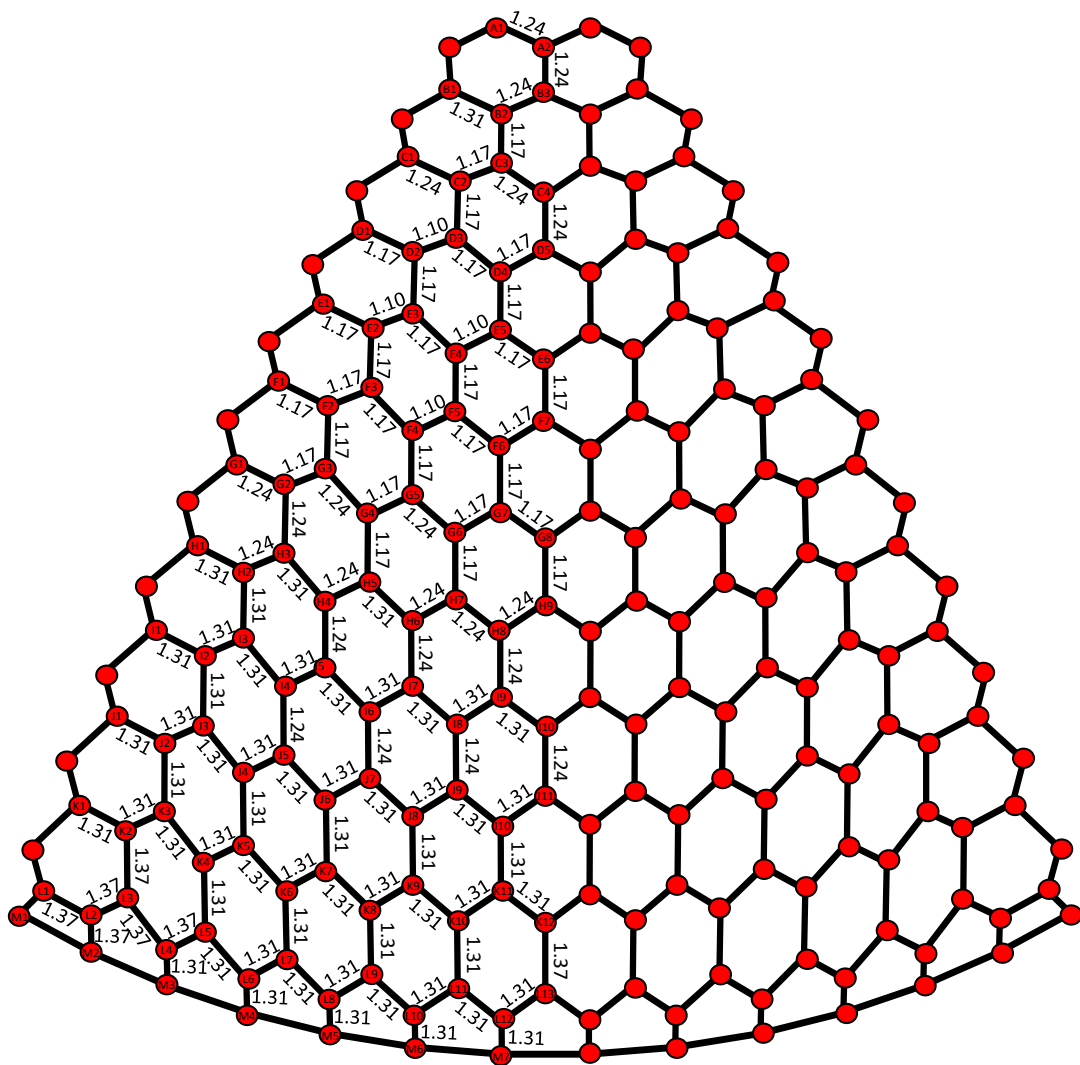


Figure 32: Progressive collapse load for each member based on alternate path analysis for the configuration $V.3^6$ with the span-to-rise ratio of 6



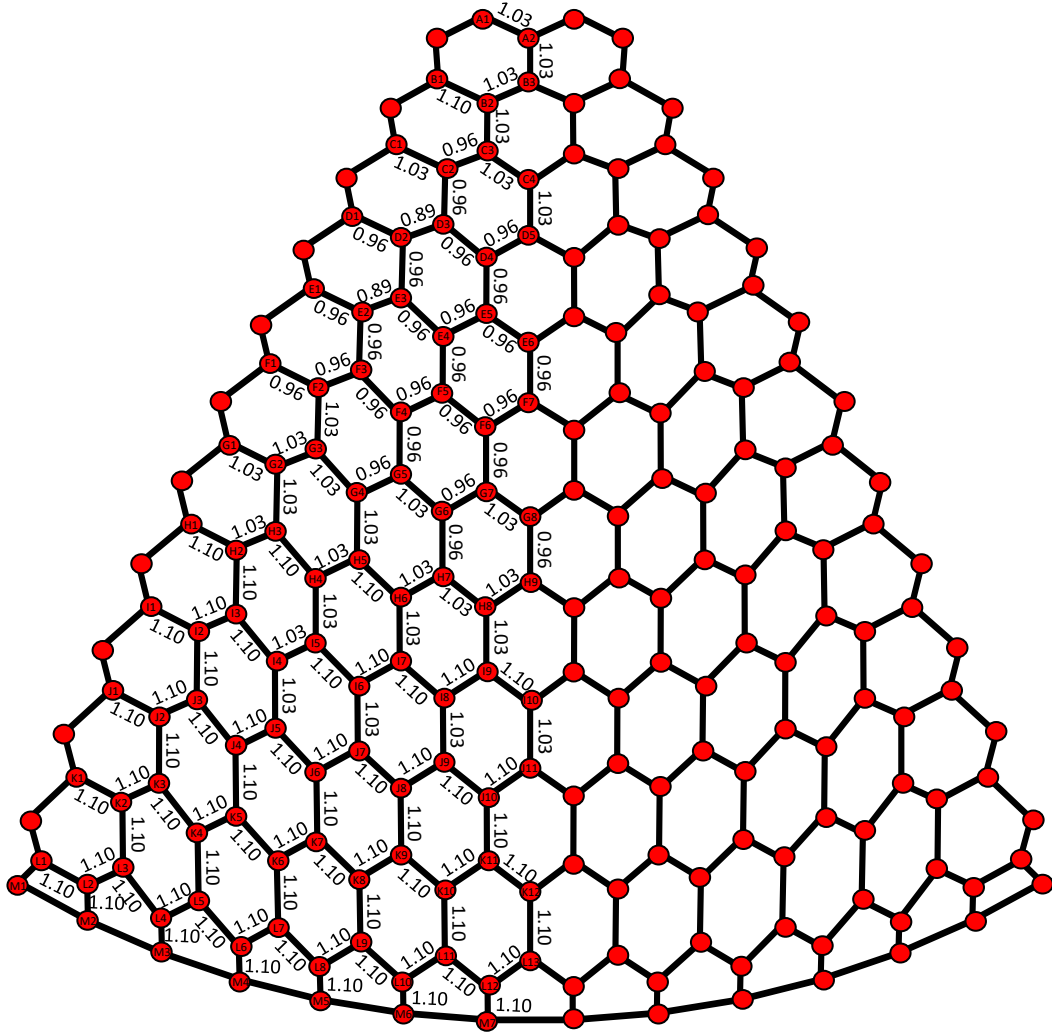


Figure 34: Progressive collapse load for each member based on alternate path analysis for the configuration V.3⁶ with the span-to-rise ratio of 8

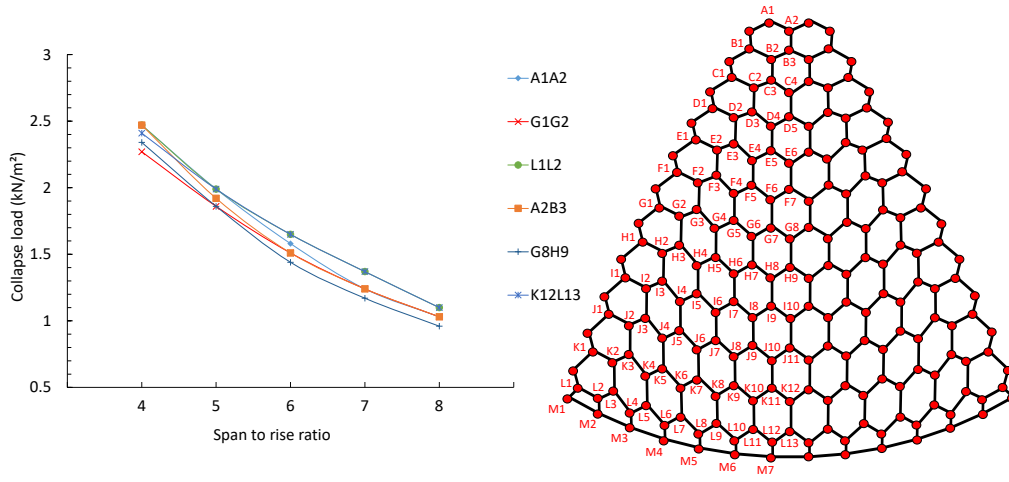


Figure 35: Variation in the collapse load with change in span-to-rise ratio for selected member failure in the configuration $V.3^6$

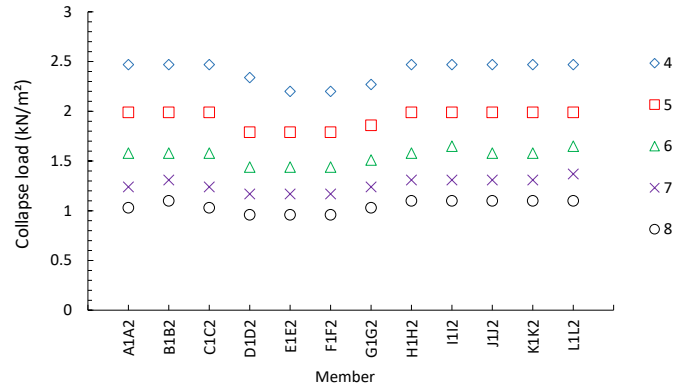


Figure 36: Variation in the collapse load for members from the apex to the supports for the $V.3^6$ configuration. Results are provided for different span-to-rise ratios.

(Table 3). Since the load capacity is measured per unit area, the density of nodes (or members or connections) per unit area also affects the progressive collapse load. Therefore, comparing progressive collapse loads based on lattice domes of similar weights does not accurately represent the structures' actual robustness.

Table 3: Properties of three lattice dome configurations when span-to-rise ratio is 5

Configuration	N_0	N_1	N_2	$\bar{v}_1$	$\bar{f}_1$	Total length (m)
$V.6^3$	169	462	294	5.47	3.00	1582.68
$V.4^4$	313	624	312	3.99	3.92	1448.80
$V.3^6$	936	1406	271	3.00	5.71	1625.60

4. Discussions

The results of the progressive collapse analysis of single-layer lattice domes with different magnitudes of CTPs revealed that the high edge valency of vertices and low edge valency of faces of $V.6^3$ configuration helps to increase the alternate load paths, thereby increasing the collapse resistance. The number of connections and members is also the lowest for the $V.6^3$ configuration among the three lattice domes considered (Table 3). Additionally, an even distribution of members also enhances the robustness of these configurations. Therefore, Kiewitt configurations are always preferred whenever single-layer spatial structures are adopted.

PCR increases with a increase in the edge valency of vertices and a decrease in the edge valency of faces. However, the collapse resistance of $V.3^6$ is higher than that of $V.4^4$, even though the edge valency of vertices for the former is lower (Table 3). The member or node density is much higher in configuration $V.3^6$ compared to $V.4^4$. In addition, radial members play a significant role in load transfer in $V.4^4$. Therefore, they are more sensitive to progressive collapse. Moreover, the member length variation is much higher in $V.4^4$ compared to the other two configurations. Small members are crowded towards the apex region, and longer and fewer members are located toward the support region for $V.4^4$ (Figure 10b). This inefficient distribution of members, again, reduces the PCR.

Comparison between different spatial structural configurations is a complex task due to variation in structural parameters. The three lattice dome configurations have different magnitudes of CTPs. Whenever the values of CTPs vary, the number of vertices (N_0), the number of members (N_1), and the total length of the members used in the structure also change for a particular configuration (Table 3). To ensure a

fair comparison of the PCR of these structures, the present investigation considered total member length as comparable parameters in the three configurations. The total length of the members directly influences the total cost of the structure and provides a basis for comparison between different configurations. Although achieving the same length of the members for all three configurations is challenging, it has been optimised to keep it within the same interval. However, when the total length of the configurations is comparable, other parameters, such as the total number of connections and members, vary (Table 3).

The PCR increases with a decrease in the span-to-rise ratio of the lattice domes. This finding is consistent with previous research by Tian et al. [17]. A lower span-to-rise ratio of the dome promotes arch action, thereby enhancing collapse resistance. In addition, a lower span-to-rise ratio of the structure reduces the likelihood of node buckling and propagation of failure within the structure. Therefore, employing a lower span-to-rise ratio for single-layer lattice domes is advisable when aiming for greater PCR. However, the cost of the structure also increases as the span-to-rise ratio of the structure decreases.

The results from this research primarily indicate the importance of considering Critical Topological Parameters in increasing the alternate load paths for higher robustness in spatial structures. Therefore, a configuration with higher Critical Topological Parameters ensures higher resistance to progressive collapse.

5. Conclusions

Different single-layer lattice dome configurations can be defined on the basis of topological parameters inspired by crystal lattices. Critical topological parameters (CTPs), which directly distinguish between different lattice dome configurations, can be identified on the basis of the interrelations among various structural elements based on crystal lattice formalism. These CTPs play an important role in the resistance to progressive collapse of spatial structures. Therefore, three single-layer lattice dome configurations with different CTP values were subjected to progressive collapse analysis using the APM. Various span-to-rise ratios were also considered to assess the applicability of the results across different structural depths.

The results indicate that a higher edge valence of the vertices enhances progressive collapse resistance by increasing the number of alternate load paths. The Kiewitt configuration was found to have the highest progressive collapse resistance due to its high edge valency of vertices and low edge valency of faces. $V.4^4$ configurations, despite having a higher edge valency of vertices than the $V.3^6$ configurations, exhibited lower progressive collapse resistance due to the high sensitivity of the slender radial

members near the support regions. The number of connections also varies with the configurations, contributing to the load resistance in a structure.

Therefore, this study suggests that CTPs play a major role in increasing alternate load paths and increasing resistance to progressive collapse. A higher edge valency of the vertices can increase the load paths in a structure, enhancing the robustness of single-layer lattice domes when the individual member lengths and number of connections are comparable in a spatial structure.

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