UDC 539.3

# STUDY OF THE BEHAVIOR OF A MESH SHELL ROOF STRUCTURE UPON DEACTIVATION OF INDIVIDUAL ELEMENTS

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The stress-strain state of a mesh shell roof model in the form of a dome was investigated under the failure of individual elements. In the LIRA software package, a spatial finite element model of the shell was created, loaded with a uniformly distributed load applied to the nodes. A numerical approach in a static formulation was employed. The problem was solved in three stages. First, the finite element model of the shell in its intact state was constructed, and the stress-strain state of the undamaged structure was evaluated. Next, one element was removed, and its effect was modeled by applying forces obtained from the first stage to the nodes with opposite signs. The system with the removed element was analyzed, and the behavior of adjacent shell elements and the overall load-bearing capacity of the structure were examined. In the final stage, an additional element adjacent to the first removed element was taken out, and the stress-strain state of the shell was re-evaluated. The numerical approach in the static formulation was consistently applied throughout the analysis.

**Keywords:** progressive collapse, mesh shell, finite element method, stress-strain state.

**Introduction.** Modern construction requires the creation of reliable, economical, and technologically efficient structures using advanced design approaches. The widespread use of lightweight spatial structures, such as mesh shells and domes, is one of the directions for effective construction. Domes are among the oldest spatial roof structures, yet they remain relevant today in terms of economy and efficiency, especially for large spans. Mesh shells have become a common means of shaping energy-efficient buildings and structures and are widely used in both civil and industrial construction. However, due to the large number of lightweight, thin elements and connections in such structures, there is a risk of progressive collapse, where the failure of a single element can lead to partial or complete failure of the entire structure. Structural failure occurs when the system cannot achieve a new state of equilibrium following a local failure.

The importance of analyzing progressive collapse in shell structures has significantly increased due to various accidents in Ukraine, particularly as a result of military actions. Progressive collapse can be caused not only by explosions or fires but also by design errors, non-compliance with manufacturing technology, or operational conditions. Although the impact of local failure in single-layer shells is higher than in frame structures, research on progressive collapse in shell structures began later than for framed buildings [1–4]. Since these structures are widely used in civil construction, they carry high social significance and responsibility in operation, particularly due to the potential for large crowds during public events. Therefore, the issue of progressive collapse is highly relevant and requires measures and methods to improve the design of structures to prevent it.

A literature review has shown that the number of publications on the topic of progressive collapse of shell structures is very limited, highlighting the need for further study of these processes. Theoretical and experimental research in this area is primarily related to the vulnerability analysis of frameless metal shells for special purposes [5, 6] and single-layer mesh roof shells subjected to progressive collapse [7–15]. Researchers have identified factors influencing collapse resistance and have evaluated methods for reinforcing elements of such structures.

In his works, L.-M. Tian extensively examined the behavior of single-layer spatial shells under progressive collapse. Methods for assessing resistance to progressive collapse and studies on the influence of the rise-to-span ratio of domes were proposed in [7–11]. A method was developed using a quantitative assessment index, the “collapse resistance reserve factor,” based on incremental dynamic analysis, and the importance of accounting for initial geometric imperfections using the method of successive modal imperfections was confirmed [7]. To verify the method, three typical shells with positive, zero, and negative Gaussian curvature were analyzed. For performing nonlinear analysis in

[8], the authors considered the first 10 buckling modes simultaneously, which allowed for the identification of critical elements whose failure could trigger progressive collapse.

Additionally, the authors developed a new dynamic analysis method: based on the "birth" and "death" algorithm for elements, an equivalent unloading method considering construction effects was proposed. This approach accounts for the redistribution of internal forces at each construction stage and gradual equivalent unloading during the removal of critical elements. The influence of the rise-to-span ratio was investigated using the Kiewitt-6 dome under scenarios of unevenly distributed snow loads [9], simultaneous or sequential element failures [10], and uniformly distributed loads [11].

Yan et al. [12] conducted nonlinear dynamic analyses using the alternative load path method for four types of single-layer mesh domes to identify the distribution of critical elements. In another study, Zhang et al. [13] performed a sensitivity analysis of Kiewitt domes to progressive collapse by removing elements while considering various geometric parameters. Xu et al. [14] assessed the resistance of single-layer mesh domes with bolted and welded joints to progressive collapse. Parametric analysis was used to evaluate the influence of joint stiffness, grid configuration, and type of initial failure. The results showed that dome configurations with rectangular grids exhibited lower resistance to progressive collapse than those with triangular grids. Additionally, two modes of failure propagation were identified: radial and circumferential. A comparison of the progressive collapse resistance of different single-layer lattice dome structures—Kiewitt domes and geodesic domes—was carried out in [15]. It was found that the Kiewitt Lamella dome demonstrated better resistance to progressive collapse than the geodesic dome.

The vast majority of experimental studies on the progressive collapse of mesh domes are based on the alternative load path method [9–11, 15]. Experimental results aimed at identifying mechanisms to prevent the collapse of spatial mesh structures through substructure investigations are also presented in [16–18]. Experimental studies are complex, costly, and labor-intensive, which makes numerical methods widely used due to the availability of high-performance software for accurate and rapid data analysis.

Analytical studies serve as a supplementary approach for conducting experimental and numerical research on shell structures and can be a powerful tool for understanding their behavior under collapse. In this context, the problem of progressive collapse in mesh shells is addressed by identifying the critical elements of the structure.

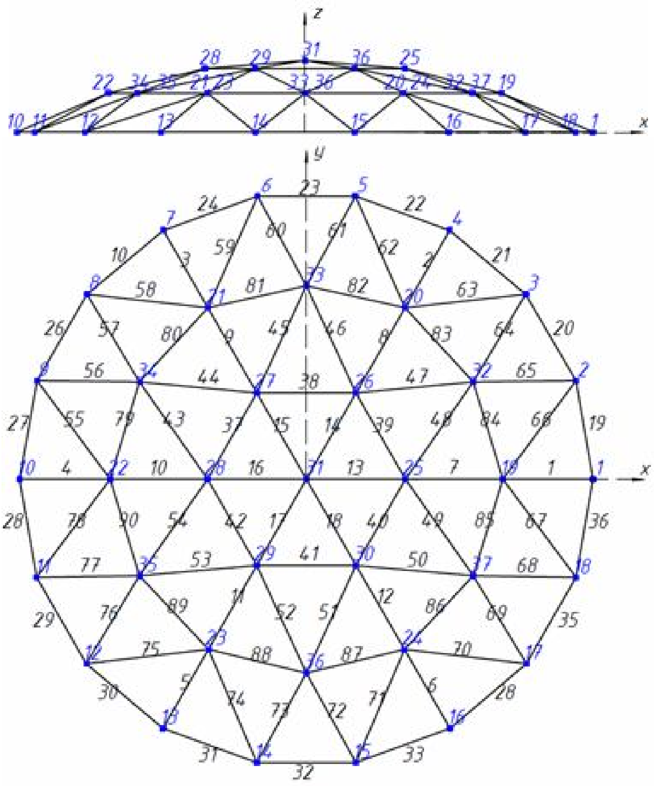
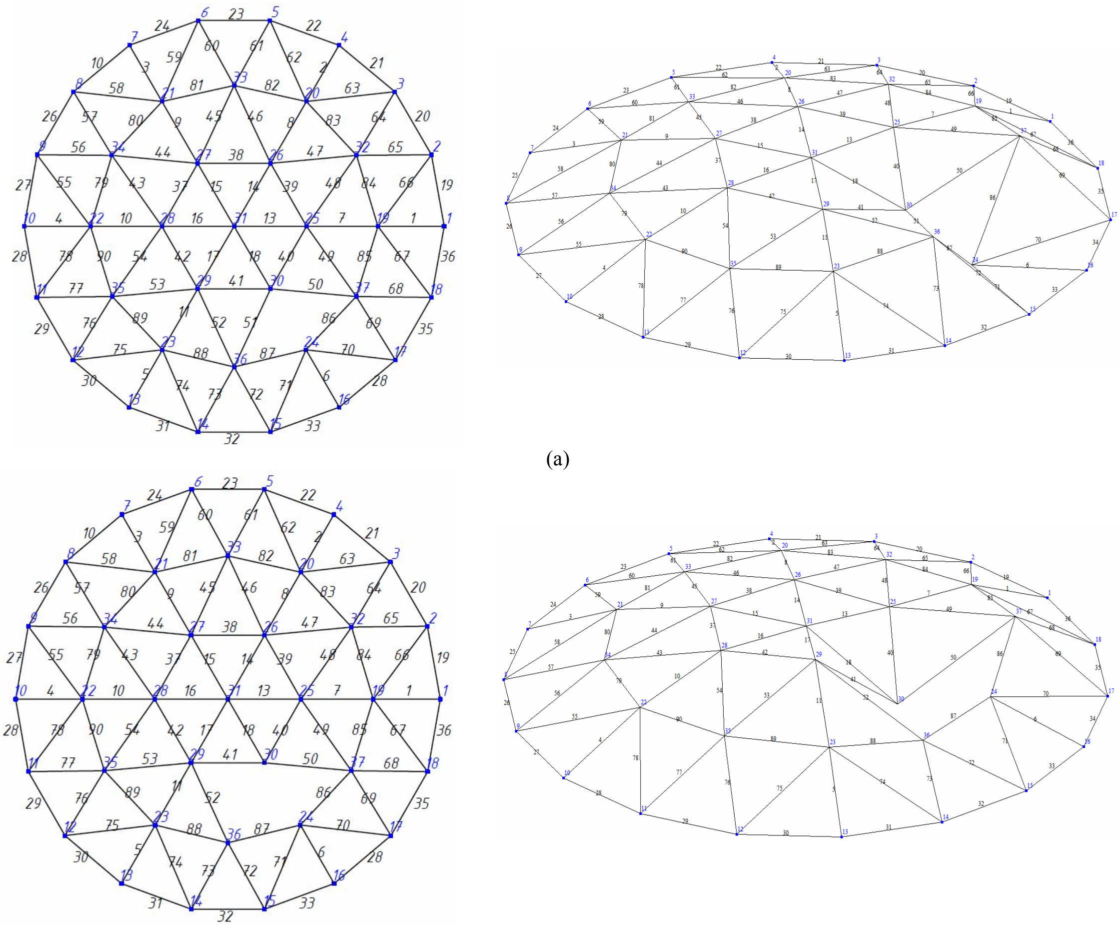


Fig 1. Computational scheme of the shell

**1. Finite element model of dome structure**. To solve the problem, the authors used a model of a single-layer mesh dome for which experimental data are available [11]. In the LIRA software package, a finite element model of a dome with a diameter of 2.4 m and a rise of 0.3 m was constructed. The model consists of 37 nodes and 90 elements. The steel tubular members were modeled as beam finite elements with a cross-section diameter of 10 mm and a wall thickness of 1 mm. All shell nodes are rigid by default, and the support nodes of the lower chord are fixed. The mechanical properties of the structural elements are as follows: $E = 2{,}14 \cdot 10^7 \, \text{kPa}$, $\rho = 78{,}5 \, \text{kN/m}^3$. A concentrated load of 0.4 kN is applied to each node. The computational scheme of the shell is shown in Figure 1.

A static analysis was performed in the linear formulation for the undamaged structure. Subsequently, the analysis was carried out with the sequential removal of elements. The stress-strain state of the mesh shell was investigated when first one and then a pair of elements were taken out of service. The effect of the removed element was compensated by equivalent forces applied to the nodes with opposite signs. Initially, member No. 12 of the second ring was removed to interrupt the load transfer path between the first and third rings of the dome, followed by the removal of the adjacent diagonal member No. 51.

Computational and deformed configurations of the shell are shown in Figure 2.



(a)

(b)

Fig. 2. Computational and deformed configurations of the shell:
(a) after removing element 12 from service; (b) after removing elements 12 and 51 from service

To assess the impact of removing elements 12 and 51 on the stress-strain state of the shell, the nodal displacements and forces in the elements near the removed members along the load transfer path were analyzed. Table 1 presents the nodal displacements of the model in the undamaged state (state 1), after the removal of a single element (state 2), and after the removal of a pair of adjacent elements (state 3).

The removal of member No. 12 caused the greatest increase in vertical displacements in adjacent nodes No. 24 and No. 30, with vertical displacements of -3.33 mm and -2.278 mm, respectively. The removal of element No. 51 affected the vertical displacements of nodes No. 30 and No. 36, which were -11.862 mm and -3.335 mm, respectively, with the maximum uplift observed at node 37, reaching 3.5 mm. Displacements in the other nodes were significantly smaller.

Furthermore, after the removal of element No. 12 disrupted the local load transfer path, the adjacent elements No. 50 and No. 51 of the second ring took on compressive forces. The increase in forces in these elements was 116% of the force in the undamaged state. In the third ring, the highest compressive forces were observed in elements No. 68 and No. 73, with $N$ = -2.405 kN. It is also worth noting that element No. 6 of the third ring, which was connected to the removed member, changed from compression $N$ = -0.682 kN to tension $N$ = 1.604 kN.

Table 1

Nodal displacements in elements adjacent to the removed members

| Node No. | State 1 | | | State 2 | | | State 3 | | |
|---|---|---|---|---|---|---|---|---|---|
| | $X$, mm | $Y$, mm | $Z$, mm | $X$, mm | $Y$, mm | $Z$, mm | $X$, mm | $Y$, mm | $Z$, mm |
| 23 | 0.051 | 0.088 | -0.355 | -0.067 | -0.1 | 0.089 | -0.327 | -0.112 | 0.403 |
| 24 | -0.051 | 0.088 | -0.355 | -0.663 | 0 | -3.33 | -0.517 | 0.288 | -2.638 |
| 25 | -0.054 | 0 | -0.415 | 0.029 | -0.107 | -0.649 | -0.509 | -0.697 | -3.016 |
| 28 | 0.054 | 0 | -0.415 | 0.115 | -0.088 | -0.612 | -0.405 | -0.144 | 0.549 |
| 29 | 0.027 | 0.046 | -0.414 | 0.078 | -0.078 | -0.649 | -1.152 | -0.548 | 3.271 |
| 30 | -0.027 | 0.046 | -0.414 | 0.033 | -0.057 | -2.278 | -3.308 | 1.076 | -11.862 |
| 31 | 0 | 0 | -0.459 | 0.086 | -0.149 | -1.058 | -0.277 | -0.592 | -2.424 |
| 35 | 0.084 | 0.049 | -0.421 | 0.06 | -0.018 | -0.316 | -0.472 | -0.165 | 0.853 |
| 36 | 0 | 0.098 | -0.421 | -0.21 | -0.548 | 0.992 | -0.541 | -0.699 | -3.335 |
| 37 | -0.084 | 0.049 | -0.421 | 0.579 | -0.092 | 0.992 | 1.625 | -0.671 | 3.502 |

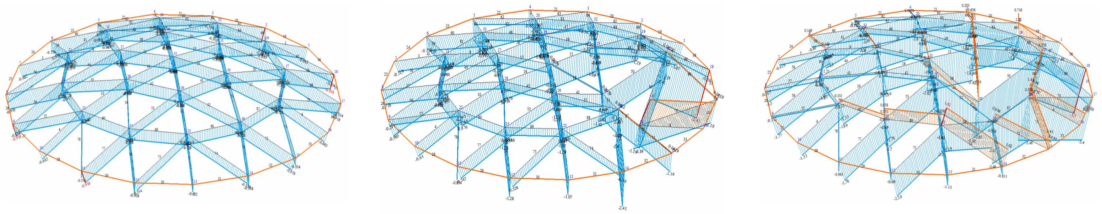
After the removal of the two adjacent elements, namely members No. 12 and No. 51, the greatest increase in compressive force was observed in the adjacent element No. 41, amounting to 308% of the force in the undamaged state. The most compressed elements in the third ring were No. 68 and No. 70, with $N$ = -3.505 kN and $N$ = -3.397 kN, respectively, as well as the member adjacent to the removed element, No. 87, with $N$ = -3.457 kN. The largest changes in element forces are summarized in Table 2.

Table 2

Axial forces in elements adjacent to the removed members

| Element No. | State 1, $N$, kN | State 2, $N$, kN | State 3, $N$, kN |
|---|---|---|---|
| 6 | -0.682 | 1.604 | 1.164 |
| 12 | -0.869 | - | - |
| 18 | -0.814 | -0.098 | 0.669 |
| 41 | -0.768 | -0.649 | -2.654 |
| 50 | -0.653 | -1.413 | -2.924 |
| 51 | -0.653 | -1.413 | - |
| 68 | -0.918 | -2.405 | -3.504 |
| 70 | -0.554 | -1.185 | -3.397 |
| 73 | -0.918 | -2.405 | -0.811 |
| 87 | -0.728 | -1.194 | -3.457 |

Figure 3 below shows the axial force diagrams in the shell elements for the three states.



(a) (b) (c)

Fig. 3. Axial force *N* diagrams in the shell elements: (a) in the undamaged state; (b) after removing element 12 from service; (c) after removing elements 12 and 51 from service

**Conclusion.** The presented numerical approach made it possible to investigate the impact of element removal on the distribution of deformations and the redistribution of forces in elements adjacent to the removed members and in elements located within the damaged zone. Increases in forces of 116% and 308% were observed in individual elements. Moreover, the nature of the stress-strain state in certain elements changed: prior to the removal, the members were in compression, whereas after removal, tension appeared in these elements. Such changes indicate the risk of progressive collapse. This work represents one of the authors' first studies on the analysis of shell stability against progressive collapse. The next step will involve investigating the dynamic behavior of the mesh shell.

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*Костіна О.В., Самойленко Б.К.*

**ДОСЛІДЖЕННЯ ПОВЕДІНКИ СІТЧАСТОЇ ОБОЛОНКИ ПОКРИТТЯ ПРИ ВИКЛЮЧЕННІ З РОБОТИ ОКРЕМИХ ЕЛЕМЕНТІВ**

Досліджено напружено-деформований стан моделі сітчастої оболонки покриття у вигляді купола при відмові окремих елементів. У програмному комплексі ЛІРА створена просторова модель оболонки, навантажена рівномірно розподіленим навантаженням шляхом прикладання його до вузлів. Модель містить 37 вузлів і 90 елементів. Сталеві трубчасті стержні моделювалися балочними скінченними елементами з поперечeним перерізом діаметром 10 мм і товщиною стінки·1 мм. Усі вузли оболонки жорсткі за замовчуванням, опорні вузли нижнього поясу жостко затиснені. Застосований чисельний підхід у статичній постановці. Задачу розв'язано у три етапи. Спочатку сформована скінченно-елементна модель оболонки до відмови елементів, оцінено напружено-деформований стан непошкодженої конструкції. Потім видалено один елемент із моделюванням його дії зусиллями, що отримані на першому етапі розрахунку, і прикладеними до вузлів з протилежними знаками. Виконано розрахунок системи з видаленим елементом і проаналізовано поведінку сусідніх елементів оболонки і несучу здатність конструкції в цілому. На останньому етапі видалено іще один елемент поруч із першим. При цьому враховано стан конструкції після втрати першого елемента. Знову досліджено напружено-деформований стан оболонки. Для виявлення впливу на напружено-деформований стан оболонки виключення з роботи окремих елементів проаналізовані переміщення вузлів та зусилля в елементах, що знаходяться близько до видалених на шляху передачі зусиль. Виявлені значні зміни в напружено-деформованому стані великої кількості елементів. В окремих елементах спостерігалося збільшення зусиль на 116 і 308%. При цьому в окремих елементах змінився характер напружено-деформованого стану: до вилучення елемента стержні працювали на стиск, після вилучення в цих елементах спостерігався розтяг. Такі зміни свідчать про небезпеку прогресуючого обвалення.

**Ключові слова:** прогресуюче обвалення, сітчаста оболонка, метод скінченних елементів, напружено-деформований стан.

*Kostina O.V., Samoilenko B.K.*

**STUDY OF THE BEHAVIOR OF A MESH SHELL ROOF STRUCTURE UPON DEACTIVATION OF INDIVIDUAL ELEMENTS**

The stress–strain state of a dome-shaped grid shell roof model under the failure of individual elements has been investigated. A spatial model of the shell was created in the LIRA software package and loaded with a uniformly distributed load applied to the nodes. The model contains 37 nodes and 90 elements. The steel tubular bars were modeled using beam finite elements with a circular cross-section of 10 mm in diameter and a wall thickness of 1 mm. All shell nodes were assumed to be rigid by default, and the support nodes of the lower ring were fixed. A numerical approach in the static formulation was applied. The problem was solved in three stages. First, a finite element model of the intact shell was created and its stress–strain state was analyzed. Then, one element was removed, and its effect was simulated by applying to its end nodes forces equal in magnitude and opposite in direction to those obtained in the first stage. The system with the removed element was recalculated, and the behavior of the neighboring elements and the overall load-bearing capacity of the structure were analyzed. At the final stage, another element adjacent to the first was removed, taking into account the state of the structure after the failure of the first element. The stress–strain state of the shell was analyzed again. To determine the effect of excluding individual elements from the structure on the stress–strain state, the nodal displacements and forces in the elements located near the removed ones along the load-transfer path were examined. Significant changes in the stress–strain state of a large number of elements were detected. In some members, internal forces increased by 116% and 308%. Moreover, in certain elements the nature of the stress–strain state changed: before the removal, the bars were in compression, whereas after the removal they experienced tension. Such changes indicate the potential danger of progressive collapse.

**Keywords:** progressive collapse, grid shell, finite element method, stress–strain state.

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***Костіна О.В., Самойленко Б.К.*** **Дослідження поведінки сітчастої оболонки покриття при виключенні з роботи окремих елементів** // Опір матеріалів і теорія споруд: наук.-тех. збірн. – К.: КНУБА, 2025. – Вип. 115. – С. 143-149.
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Табл. 2. Іл. 3. Бібліогр. 18 назв.

UDC 539.3
*Kostina O.V., Samoilenko B.K.* **Study of the behavior of a mesh shell roof structure upon deactivation of individual elements** // Strength of Materials and Theory of Structures: Scientific-and-technical collected articles. – K.: KNUBA. 2025. – Issue 115. – P. 143-149.
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Tab. 2. Fig. 3. References 18 items.

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