

Article

Comparative Selection of Staggered Jacking Schemes for a Large-Span Double-Layer Space Frame: A Case Study of the Han Culture Museum Grand Hall

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Abstract

Focusing on the construction of a 58-m-diameter double-layer steel space frame dome at the Han Culture Museum Assembly Hall, this study addresses scheme selection and safety control challenges in staggered jacking of large-span spatial structures. A three-dimensional finite element model in MIDAS Gen simulated the three-stage jacking process to compare three temporary support layouts. Numerical evaluation metrics included maximum vertical displacements, peak internal forces, the proportion of members undergoing stress state transitions, and spatio-temporal evolution of stress concentrations. Scheme B demonstrated superior performance, reducing peak vertical displacement by 44% under critical conditions, lowering peak stresses, and enabling more uniform internal force redistribution—effectively mitigating tension–compression cycling and buckling risks. Crucially, only nodal displacements and support elevations were monitored in situ using a 3D system based on magnetic prisms and total stations; no strain or force measurements were conducted due to practical constraints during construction. Monitoring data show good agreement with simulated displacements and support elevations under Scheme B, validating the model's deformation response. However, localized deviations—including a 29 mm deflection discrepancy and elevation errors up to 28 mm—reveal the influence of uneven boundary conditions, with potential implications for long-term structural behavior. The findings confirm that numerical predictions of deformation are reliable, while internal forces remain unvalidated by field data. The integrated approach of “scheme comparison–construction simulation–full-process displacement monitoring” proves effective for safety control and decision-making in complex jacking operations, offering a transferable framework for similar large-span double-layer space frame projects.



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Keywords: staggered jacking; space frame structure; construction monitoring; scheme comparison; steel structure

1. Introduction

With modern building structures developing toward larger spans and more complex spatial forms, dome structures, by virtue of their double-curvature load-transfer characteristics, have been widely used in long-span buildings such as stadiums and airport terminals. According to differences in structural topology and construction methods [1,2], modern dome structures can be broadly classified into continuous shell systems [3–5], discretely stiffened systems [6,7], and space grid structures [8]. Among these, space grid structures, by discretizing the system into bar members, achieve a high degree of synergy between architectural form and structural efficiency. They not only allow for controllable construction accuracy and reduced steel consumption but also maintain overall stability even when an individual node fails, exhibiting strong damage tolerance; therefore, they have been widely used in long-span buildings [9,10]. Regarding the selection of structural schemes for large-span domes, the comparative study conducted by Du et al. [11] using the Qinyang Gymnasium as a case example has demonstrated that double-layer latticed shells exhibit favorable overall advantages in terms of structural reliability, material economy, and technical feasibility.

The erection of large-span space grid structures commonly adopts methods such as overall jacking [1,12,13], block hoisting [14,15], high-altitude sliding [16], and high-altitude in situ assembly [17]. The choice of construction scheme is influenced by factors such as span, site conditions, and economic considerations. Among these methods, the jacking technique features high construction efficiency and good quality control, and has been widely applied in projects where ground assembly conditions are available. With the continuous increase in structural span and geometrical complexity, the focus of construction analysis for large-span space grid structures has shifted from conventional static analysis to the investigation of time-varying structural systems [18–20].

Wang et al. [21], based on an overall lifting project of a large-span steel latticed shell, established a finite element model of the entire construction process using Midas Gen and demonstrated that the finite element method can effectively predict the internal force distribution and deformation patterns of the structure during the lifting process. Sun et al. [22], through static loading tests on a K6 reticulated shell model combined with finite element analysis, found that joint performance has a non-negligible influence on structural stability and explicitly pointed out that axial slip stiffness should be taken into account in stability design. Yang et al. [23] carried out simulations for a steel arch bridge constructed using the incremental launching method. By shifting the positions of the supports to reproduce the launching stages, the study showed that accurately modeling the construction process can significantly improve the accuracy of the numerical simulation. The above studies indicate that, in complex construction processes, combining numerical simulation with on-site monitoring is a reliable approach to ensuring structural safety and construction accuracy [24].

While these studies confirm the value of integrating simulation and monitoring in construction-stage analysis, several limitations remain. First, most existing works focus on validating a single pre-determined erection scheme rather than comparing alternative temporary support configurations to identify an optimal layout. Second, evaluation criteria often rely on global metrics (e.g., maximum displacement), neglecting member-level risk indicators such as stress state reversals or buckling-prone zones, which are critical for double-layer grids under asymmetric jacking. Third, in spatially constrained urban sites, where jacking must be executed in staggered phases with limited equipment access, a decision-support framework that links parametric simulation, risk quantification, and real-time displacement feedback is still lacking.

Under the background of urban renewal and the renovation of existing building complexes, the available construction workspace is often strictly constrained, and large lifting equipment is sometimes difficult to bring into the site. This paper takes as its object of study the 58 m-diameter double-layer latticed dome with welded spherical joints forming the roof of the Grand Hall of the Han Culture Museum, and conducts a systematic investigation into the optimization of its staggered jacking construction scheme. A finite element model that accounts for the entire construction process is established, and three representative temporary support schemes are subjected to parametric simulations and comparative analysis. By comparing the peak internal forces, displacement control, and the number of members whose stress state changes, the optimal support scheme is selected. The monitoring data are systematically compared with the simulation results to verify the effectiveness of the optimized scheme, and the key stages of the structural behavior during construction are summarized. The conclusions obtained in this study are intended to provide a solid theoretical basis and a transferable practical paradigm for the safe construction of large-span steel structures in similarly complex environments.

2. Project Overview

The dome of the Grand Hall of the Han Culture Museum is a double-layer spherical latticed shell with a diameter of 58 m. The rise of the space grid is 9.8 m, the elevation of the support spheres is 5.4 m, and the total building height is 15.2 m. The structural system mainly consists of two units: an external welded-sphere steel space grid and an internal reinforced concrete frame. As shown in Figure 1, the steel latticed shell has a thickness of 2.3 m and a rise of 9.8 m. The supporting system consists of 24 reinforced concrete columns arranged in a ring, with the corbel elevation at 4.75 m.

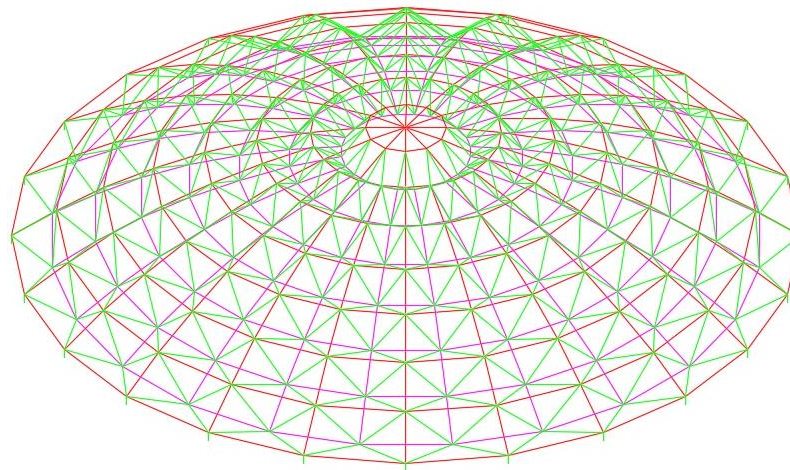


Figure 1. Schematic diagram of the spherical dome structure.

The project is located at the central part of the overall building, where large lifting equipment cannot be brought close for use, and the construction schedule is also very tight. Taking advantage of the sunken stage void, a steel assembly platform is erected, and, in combination with jacks and other lifting equipment, the structure is lifted in a staggered manner from the inner ring to the outer ring and assembled in situ.

3. Jacking Construction Scheme

3.1. Jacking Construction Technique

The jacking supports are arranged along the inner ring of the annular concrete grandstand, where the completed grandstand is used as the assembly platform, and a construc-

tion method combining simultaneous jacking and assembly is adopted. This method can effectively avoid the risks associated with high-altitude operations during the erection of the space grid, ensuring both installation accuracy and operational safety, while also enabling overlapping construction with the surrounding civil works, thereby significantly shortening the overall construction period.

According to the study by Liu [12], the height difference between jacking frames and the jacking speed are the key factors affecting the quality of overall jacking construction. Therefore, the overall structure is divided into seven annular grid rings that are progressively extended from the inside to the outside, and staged jacking operations are carried out by using the lower jacking platforms in combination with hydraulic jacks.

As shown in Figure 2, due to the limited working area of the circular jacking platform, the installation range of the first-stage jacking supports is restricted to the region of the first and second grid rings. As shown in Figure 3, the construction stages are denoted as R1 to R7, where R_n corresponds to the installation of the n -th concentric ring of the space grid structure; the entire dome consists of seven rings assembled. If the structure is continuously lifted only by the supports beneath the second grid ring, the upper chord members may experience excessive tensile stress and suffer damage when the fifth to seventh grid rings are subsequently installed. Therefore, when construction proceeds to the fourth grid ring, additional jacking supports need to be installed around the perimeter. With this change in boundary conditions, the entire jacking process is correspondingly divided into three stages.

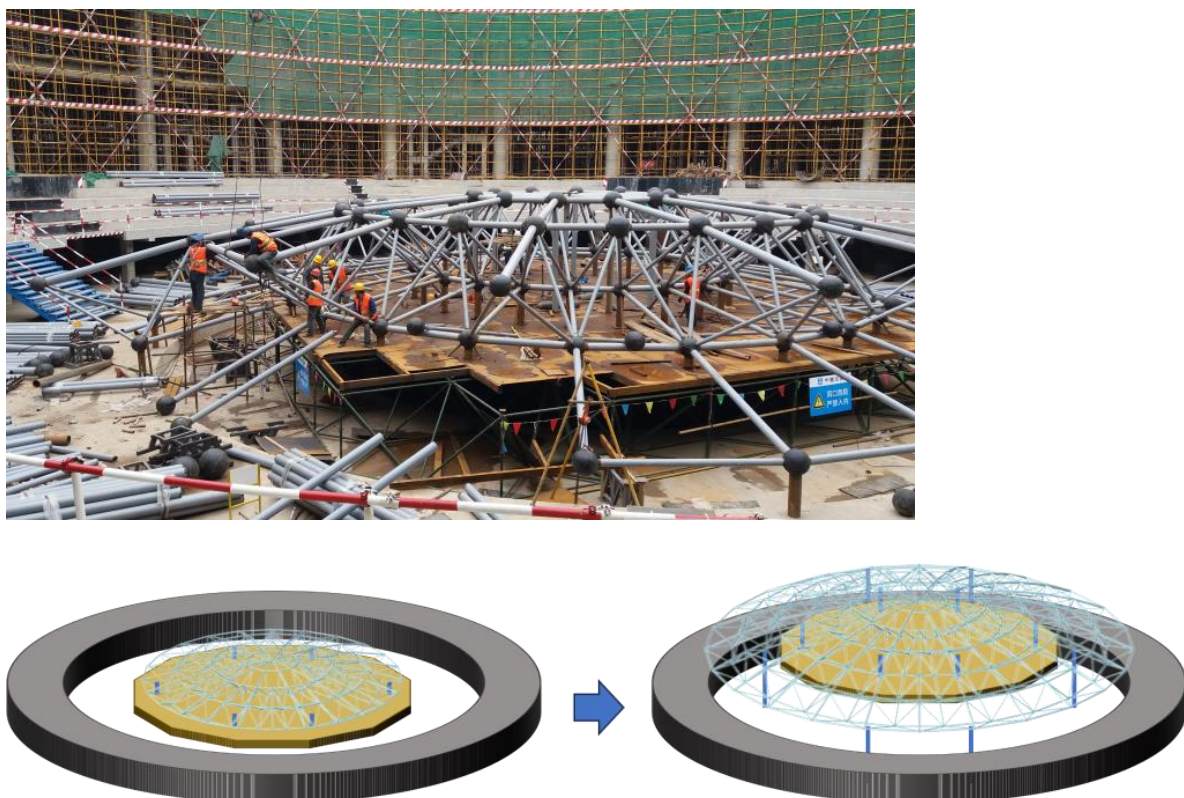


Figure 2. Schematic diagram of the staggered jacking process.

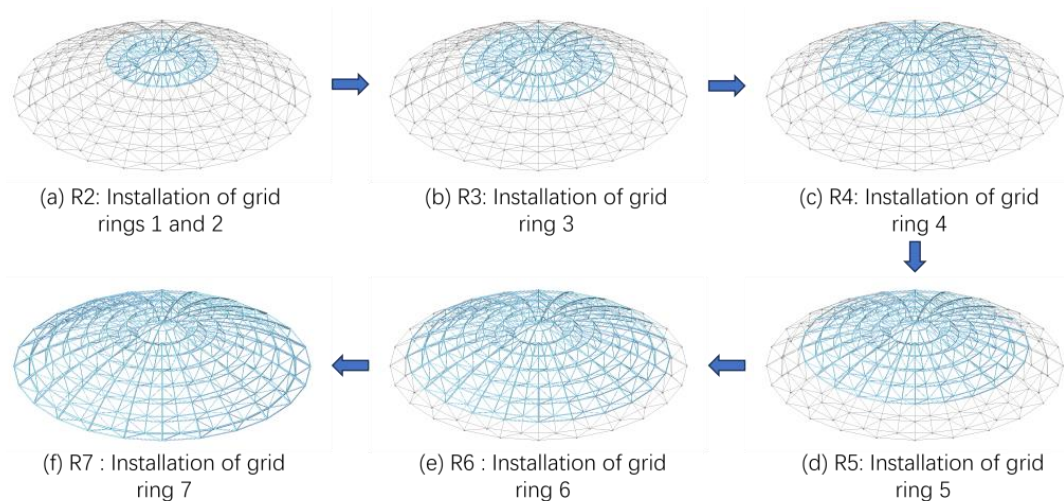


Figure 3. Jacking construction procedure.

3.2. Construction Scheme

The overall structural configuration installed in the first stage is shown by the blue portion in Figure 4. After the first and second grid rings shown in Figure 3a are installed, six jacking support frames are uniformly arranged at the lower-chord spherical joints of the second grid ring. After jacking to the predetermined height, the third grid ring is welded in place, and the structural configuration at this stage is as shown in Figure 3b. The jacking operation is then repeated to complete the welding of the fourth grid ring, as illustrated in Figure 3c.

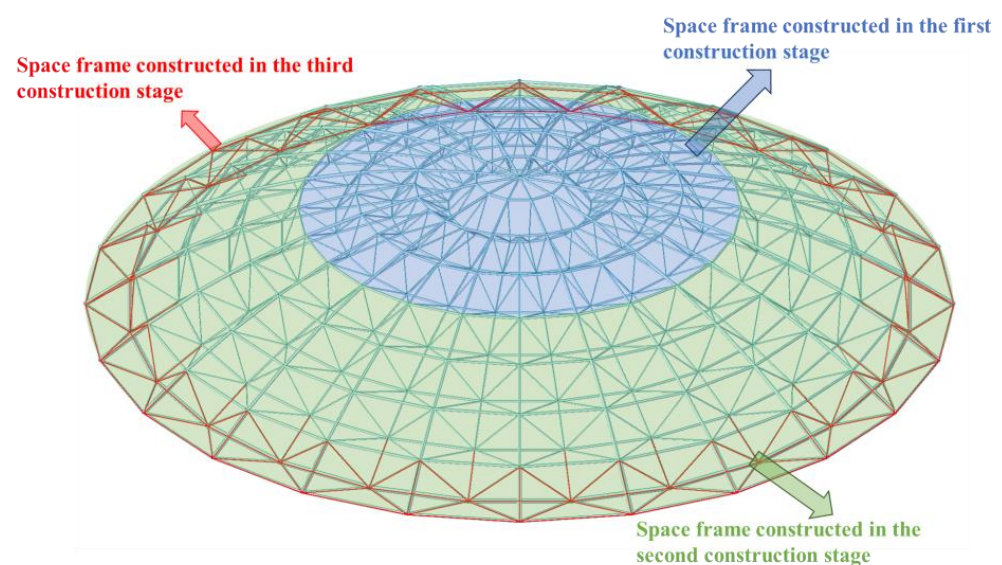


Figure 4. Division of Overall Construction Phases.

At the second stage, the fifth to seventh grid rings are installed, with the installation area shown as the green region in Figure 4. For the jacking installation of the grid structure at this stage, three alternative support schemes are proposed for comparative evaluation. By comparing the mechanical performance of each scheme during construction through finite element simulations, the scheme with the smallest maximum stress, the smallest maximum vertical displacement of nodes, and the fewest members experiencing a change in stress state is selected.

In Scheme A, the support arrangement is shown in Figure 5. On the basis of retaining the six jacking supports from the first stage, an additional six jacking supports are installed

beneath the fourth grid ring, with the new supports arranged in a staggered pattern relative to the original ones.

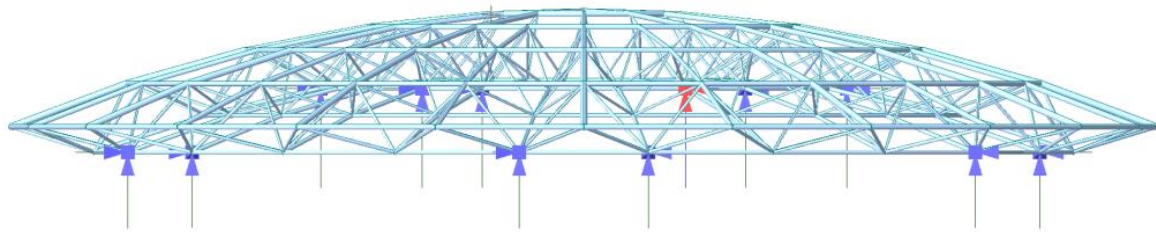


Figure 5. Boundary Conditions for Scheme A.

As shown in Figure 6, in Scheme B, twelve jacking support frames are uniformly arranged at the lower-chord nodes of the already installed fourth grid ring, while the six supports originally located beneath the second grid ring are removed.

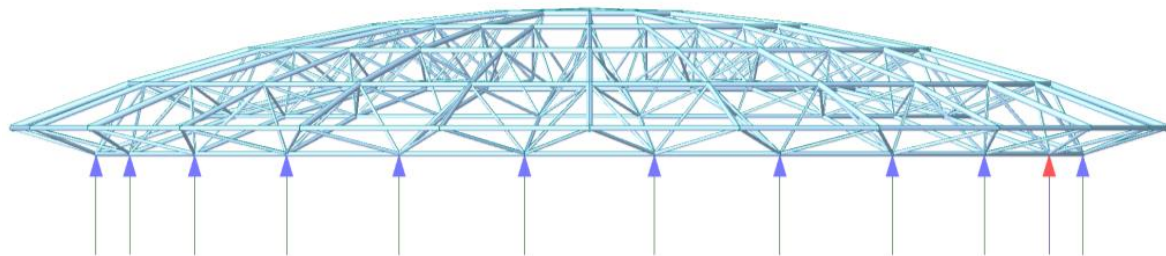


Figure 6. Boundary Conditions for Scheme B.

As shown in Figure 7, in Scheme C no additional supports are installed; the jacking operations at the second stage are carried out entirely by the six supports arranged at the lower-chord nodes of the fourth grid ring.

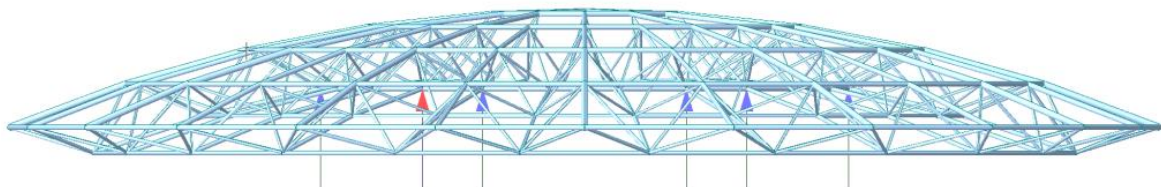


Figure 7. Boundary Conditions for Scheme C.

At the third stage, as indicated by the red portion in Figure 4, after the space grid is jacked as a whole to the appropriate height, the upper chord members and web members of the first ring are installed to complete the structural closure and final forming.

3.3. Numerical Simulation Analysis

During the jacking process, changes in the boundary conditions lead to a redistribution of internal forces in the structural members. The same member may be subjected to different internal actions such as tension, compression, and bending at different stages of construction. To ensure construction safety and structural reliability, MIDAS GEN is employed to establish a staged finite element model and to perform construction mechanical simulations for the entire staggered jacking process, thereby providing a basis for the selection of the construction scheme.

3.3.1. Element Types and Material Properties

The space grid structure mainly consists of slender members, and the welded spherical joints are treated as pin joints, so that the members are primarily subjected to axial forces and the influence of bending moments is neglected. The cross-sectional properties of all members are summarized in Appendix A.

The space grid structure mainly consists of slender members. Although the joints are fabricated by welding, the spherical geometry and relatively small joint rigidity compared to the member stiffness result in limited moment transfer. Therefore, the welded spherical joints are idealized as pin connections—a common simplification in the analysis of spatial lattice shells, as the members are predominantly subjected to axial forces and bending moments at the joints are negligible. In Figure 8, the prominently rendered members are fabricated from Q355 steel; all other members shown in the figure are Q235. Material properties are summarized in Table 1.

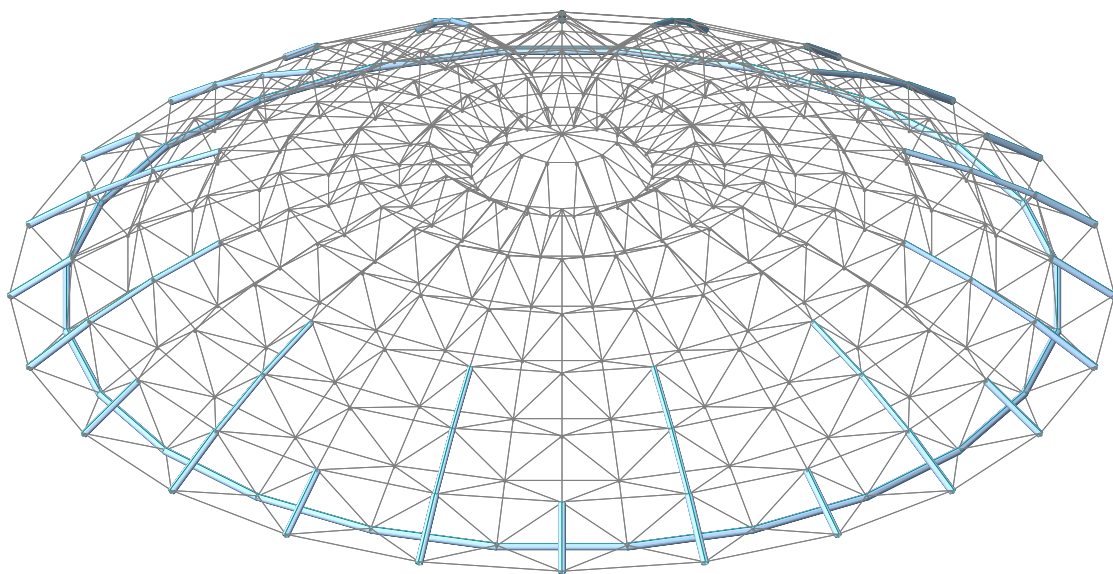


Figure 8. Schematic Diagram of Q355 Steel Members.

Table 1. Material Properties.

Material	Density (kg/m ³)	Elastic Modulus (GPa)	Poisson's Ratio	Yield Strength (MPa)
Q235	7850	206	0.3	235
Q355	7850	206	0.3	355

As shown in Figure 9, a support is installed between the spherical node and the hydraulic jack to prevent lateral displacement and rotation during jacking. Therefore, fixed constraints are applied at the jacking support locations in the numerical model for each construction stage, as indicated in Figures 5–7.



Figure 9. Field photograph of the support point during jacking.

3.3.2. Load Specifications

The Code for Construction of Steel Structures [25,26] stipulates that jacking of large structural components shall be performed in wind-free conditions. Accordingly, this study simulates only the jacking process, neglecting live and environmental loads. In the construction simulation analysis, only the construction dead load is considered, while design-stage actions such as wind and earthquake effects are ignored. The dead load consists of the self-weight of the space grid and the concentrated loads at the catwalk locations. To account for the mass of the spherical joints, the self-weight of the grid is increased by a factor of 10%. During the second construction stage, the weight of the catwalks is applied as nodal forces at the lower chord nodes, with a load of 5 kN at each of 48 nodes. However, wind or seismic effects may become significant if jacking is prolonged or performed under adverse conditions. Therefore, the results are applicable to dead-load-controlled scenarios, and a full load combination should be used in the final design.

Although wind loading is excluded from the present analysis due to regulatory requirements that jacking operations be conducted only under wind-free conditions, it remains a critical action for the completed structure under service and extreme limit states. The treatment of wind effects on spatial roof systems has been extensively investigated in the recent literature. Rizzo et al. [27] provide detailed statistics of wind pressure peak factors for curved roof geometries relevant to dome-like forms. Peng et al. [28] evaluate multiple methods for estimating peak wind loads on large-span buildings, offering guidance on load estimation accuracy. Gavanski et al. [29] discuss spatial correlation and peak factor considerations specific to lattice and spatial frameworks. Li et al. [30] present a probabilistic framework for wind-induced responses of domes and shells, including pressure coefficient derivation and load combination strategies. Together, these studies form a solid basis for wind load modeling in the final design stage, even though such loads are intentionally omitted in this study to reflect compliant construction practice under negligible wind conditions.

4. Comparison of Schemes Based on Numerical Results

4.1. Maximum Vertical Displacement and Maximum Stress

The extreme values of member stresses at each construction step are shown in Figure 10. In all three schemes, the member stresses at every stage remain below the yield strength of the material. In Scheme C, the peak tensile and compressive stresses at all stages are significantly higher than those in Schemes A and B, indicating that the addition of a supporting system during the jacking of large-span space grids is necessary.

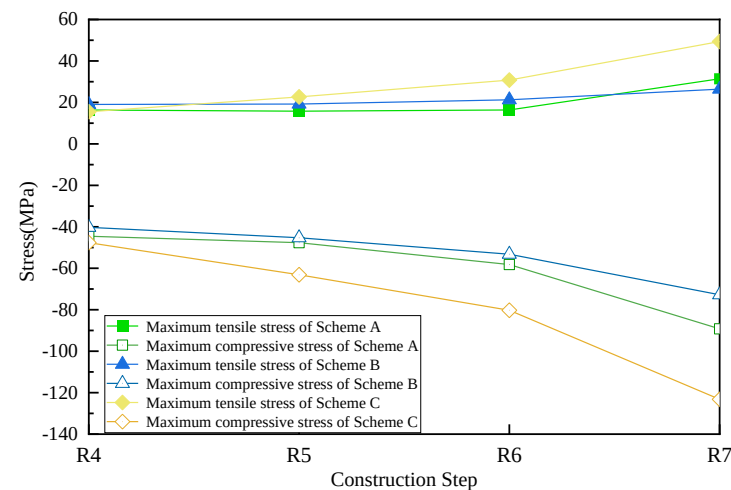


Figure 10. Maximum Stress Throughout the Entire Process for Each Scheme.

Although in steps R4 and R5 the tensile stress of some members in Scheme A is slightly lower than that in Scheme B, in steps R6 and R7, where the internal forces are most concentrated, the maximum tensile and compressive stresses in Scheme B are both lower than those in Scheme A. For steel space grid structures, the peak internal force under the most unfavorable working condition is the key factor controlling the design. The advantage of Scheme B in terms of peak stress indicates that it provides a higher safety margin.

The displacement data, as shown in Figure 11. The displacement data indicate that throughout the construction process, the displacement and stress in Scheme A increase in a nonlinearly accelerated manner as the steps progress, and exhibit a sharp surge in the final R7, which suggests that Scheme A involves potential risks.

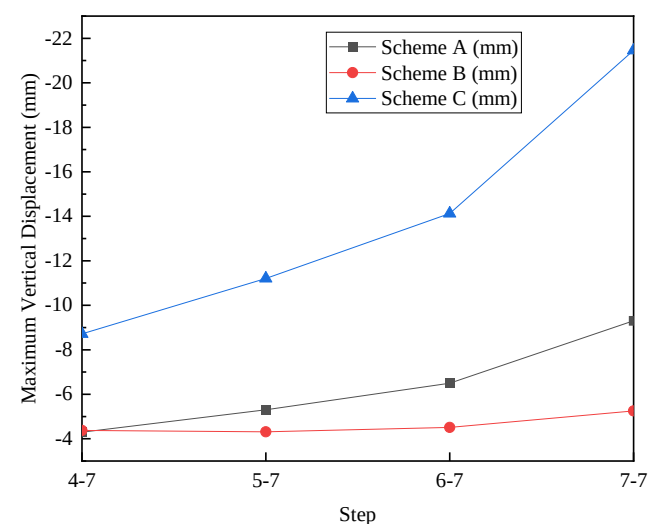


Figure 11. Maximum Vertical Displacement Throughout the Entire Process for Each Scheme.

In Scheme B, the evolution of displacement and stress remains stable, and the maximum vertical displacement is significantly smaller than that in Scheme A, with the deformation reduced by approximately 44%. Starting from R6, the difference between the two schemes increases sharply. The maximum displacement in Scheme A grows rapidly as the jacking progresses, whereas the displacement growth in Scheme B is effectively controlled, indicating that the supporting system in Scheme B has greater stiffness and higher effectiveness.

4.2. Proportion of Members with Changed Stress States at Each Construction Stage

Frequent changes in the stress state of members usually indicate that there is redundancy or conflict in the structural load-transfer path, which will cause local members to be subjected to repeated tension–compression stress reversals during construction. This phenomenon not only affects construction quality but also poses a potential threat to structural safety. To quantitatively evaluate this effect, the number of members experiencing a change in stress state during the jacking process is systematically counted for each scheme.

For the member stress results of each scheme at each construction stage obtained from the numerical simulations, a MATLAB program is developed to perform data processing and analysis. The program first reads and preprocesses the raw stress data and identifies all structural elements and construction stages. It then uses a two-level looping algorithm to traverse, in turn, the three schemes, all construction stages, and all elements, systematically detecting abrupt changes in the sign of stress and performing classified statistics. The program flowchart is shown in Figure 12. Finally, it outputs detailed statistical tables and visualization charts, and saves the complete results to an output file.

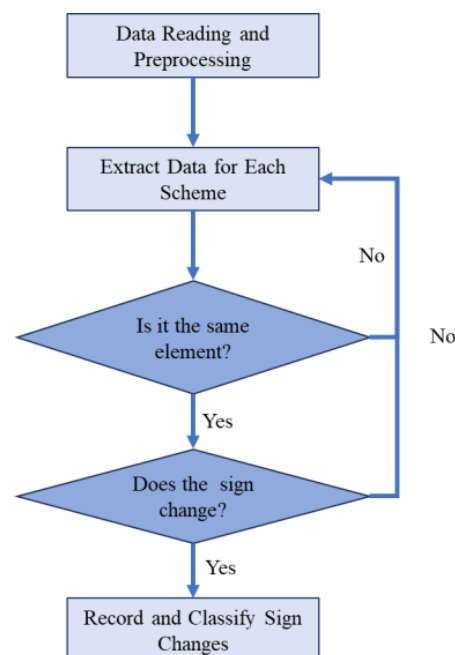


Figure 12. Flowchart of the Data Processing Program.

After excluding the influence of the common system transformation at the initial stage, a quantitative analysis was performed on the changes in stress sign during the subsequent construction stages. As shown in Figure 13a, during stages R4–R5 and R5–R6, the number of members experiencing a change in stress state in Scheme A is significantly greater than that in Scheme B. As shown in Figure 13b, across all three stages, the total number of members with a change in stress state in Scheme A is also higher than that in Scheme B.

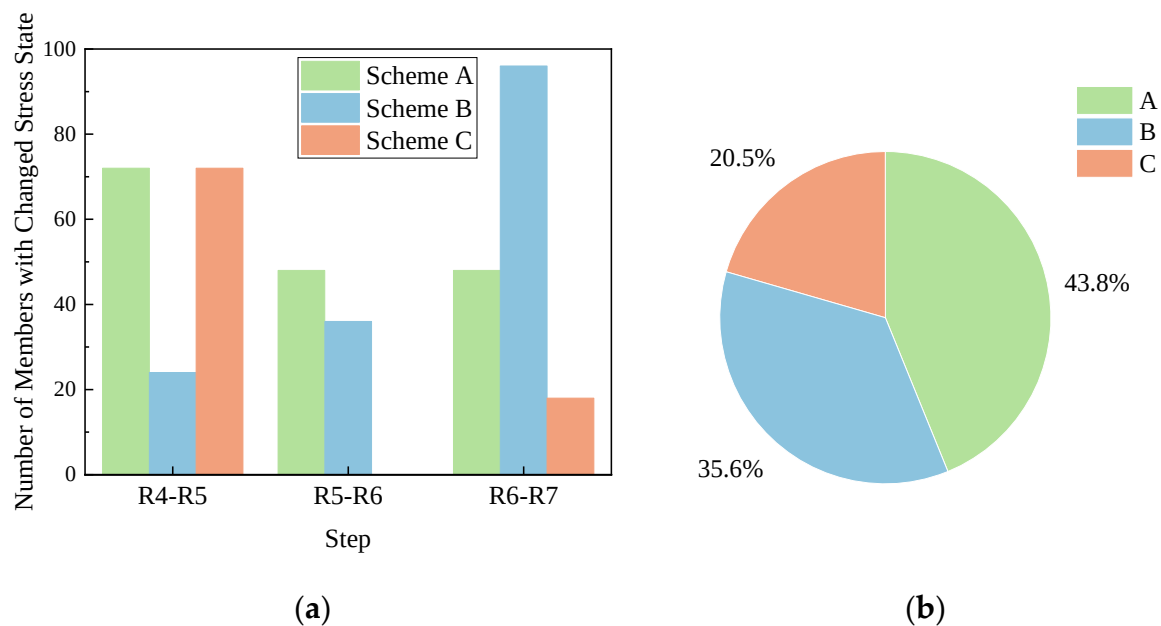


Figure 13. Proportion of Members with Changed Stress States. (a) Stage-wise numbers for Schemes A, B, and C during R4–R5, R5–R6, and R6–R7. (b) Cumulative totals represented in a pie chart for Schemes A, B, and C over the three stages.

Taken together, Scheme B outperforms Scheme A in three aspects, namely controlling the uniformity of internal force redistribution, avoiding unfavorable tension–compression stress reversals, and limiting displacements during construction. Scheme B is therefore selected as the final construction scheme.

4.3. Stress Evolution of Critical Members Throughout the Entire Process

Figure 14 presents the stress contour plots of the space grid structure during the ring-by-ring welding and jacking process. Figure 14a shows the stress distribution after the welding of the fourth ring is completed. Figure 14b illustrates the structural stress state after additional jacking supports are installed, during which the boundary condition changes from six supports located beneath the sixth ring to twelve supports located beneath the fourth ring. Figure 14c–e sequentially show the stress evolution after the welding and jacking of the third, second and first rings, respectively, ultimately forming a complete load-resisting system.

Throughout the entire construction process, a clear pattern of stress distribution can be observed. At the first stage, the maximum stress is concentrated in the inclined web members near the supports of the second ring, and these members are predominantly subjected to significant compression. At the second stage, as the support conditions change and the space grid is installed, the stress concentration zone shifts to the vicinity of the supports in the fourth ring. The inner-ring web members are subjected to compression, whereas the adjacent outer web members are in tension. After the jacking is completed, the web members of the fourth grid ring become the primary stress-bearing components.

The maximum stress is consistently concentrated in the regions near the jacking supports. Based on the stress conditions at each stage, R5 and R4 are defined as the critical loading conditions for the first and second stages, respectively. The members located at the supports of the sixth and fourth grid rings are selected as the objects of study to analyze their stress evolution over the entire construction process.

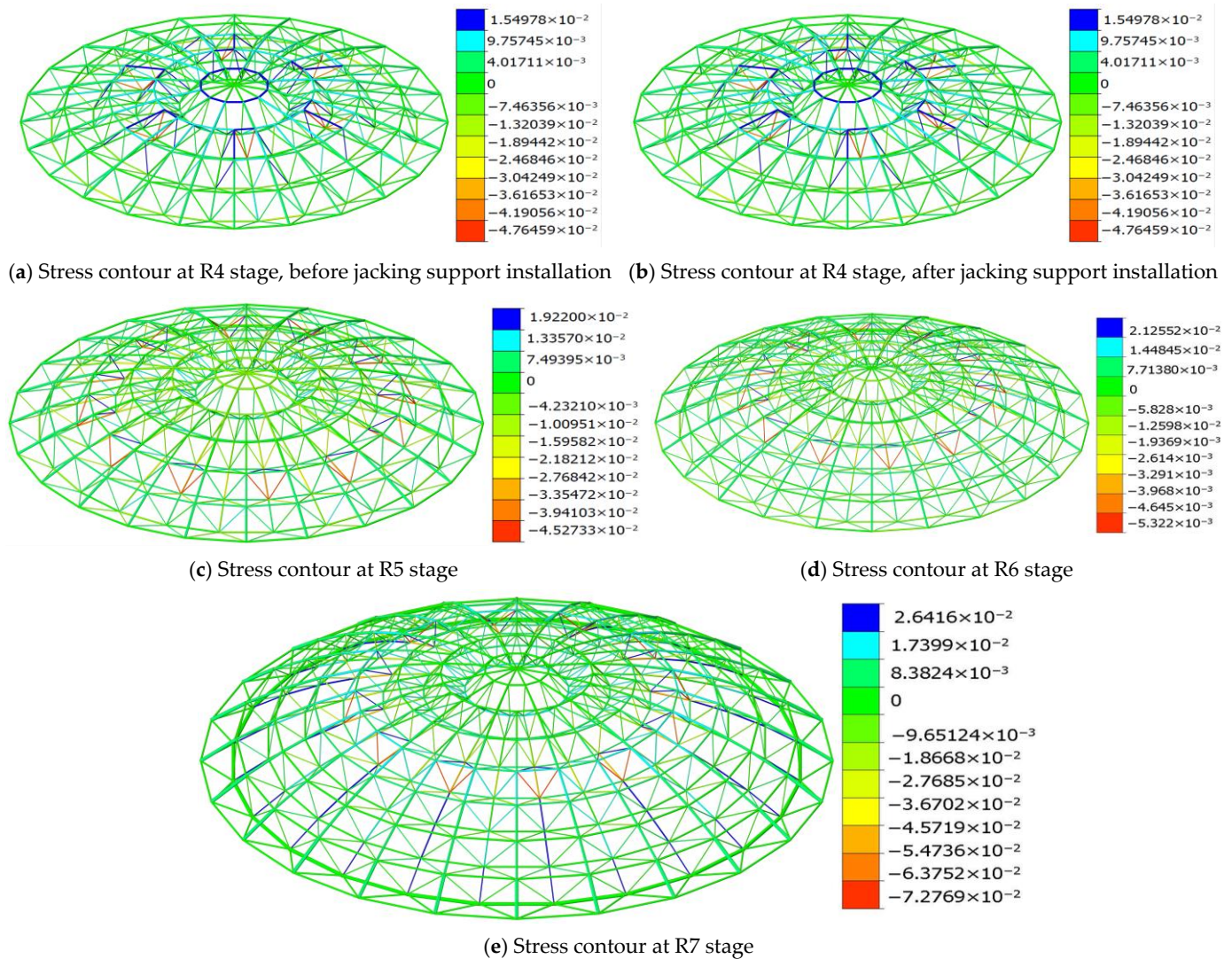


Figure 14. Overall Stress Contour of the Structure (kN/m²).

Based on the structural symmetry, only the key members on one side of the support are analyzed. At the support of the second ring, the inner web member 845 and its adjacent member 846, as well as the outer web member 849 and its neighboring member 850, are selected, as shown in Figure 15a. At the support of the fourth ring, four inclined web members and the corresponding upper chord member form a pyramidal load-transfer unit. The focus is placed on the inner web members 855 and 856 and the left symmetric web members 861 and 865, as shown in Figure 15b.

Figure 16 shows the stress states of the members at the support of the second grid ring under all working conditions. It can be seen from the figure that the member stresses undergo abrupt changes when the support condition is altered. In the first three working conditions, the web members 845 and 849 that are directly connected to the support are both in compression, whereas after the supporting system is modified, their stress state changes from compression to tension. This transition indicates that the change in the supporting system significantly affects the load-transfer path, leading to a change in the force-resisting behavior of the web members near the supports.

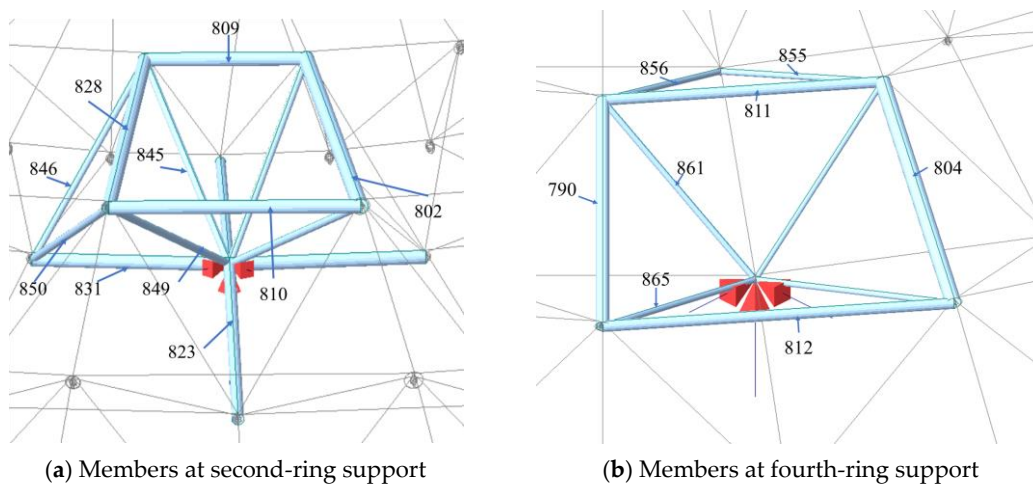


Figure 15. Schematic Diagram of Members Connected to the Support.

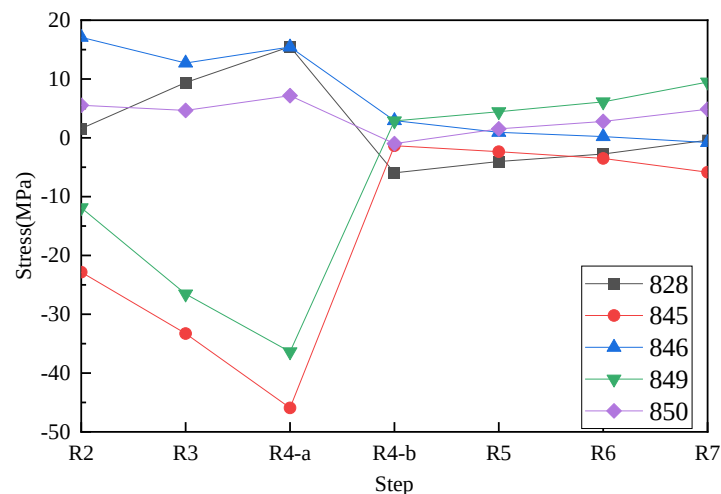


Figure 16. Stress evolution of critical members at the second-ring support throughout the entire construction process.

Figure 17 shows the stress states of the members at the support of the fourth grid ring over the entire construction process. During the erection of the space grid, the region of stress concentration in the structure is successfully transferred from the supports of the sixth ring to those of the fourth ring. As the space grid is progressively installed and jacked, the internal forces in most members continue to increase, and this region becomes the primary load-bearing zone at the middle and late stages of construction. When the support conditions change, only members 811, 812, 856, and 855 experience a change in stress state, while the other members remain stable.

4.4. Proportion of Members with Changed Stress States at Each Construction Stage

To preliminarily assess the overall stability of the structure, a linear buckling analysis was conducted, and the first twenty buckling modes were extracted. The results are presented in Table 2. The eigenvalue corresponding to the first buckling mode, namely the critical load factor, is 5.23. This indicates that under the current jacking load condition, the critical load for global instability of the structure is approximately 5.23 times the actual applied load. The result demonstrates that the structure possesses a considerable safety margin against global buckling under the studied jacking scenario, and the likelihood of global buckling is low.

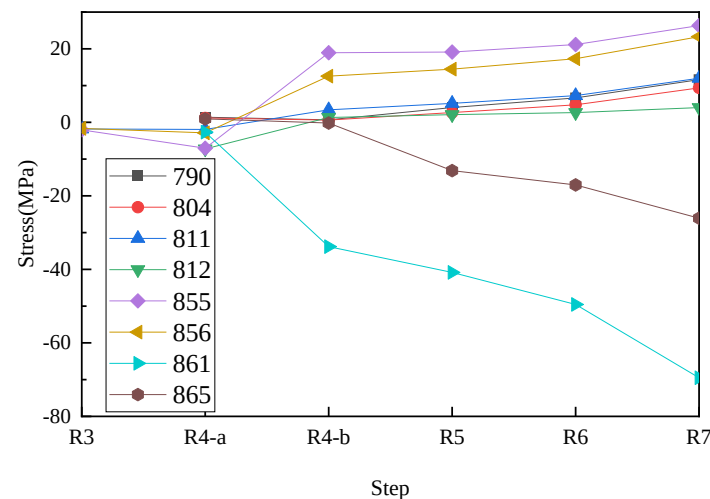


Figure 17. Stress evolution of critical members at the fourth-ring support throughout the entire construction process.

Table 2. Results of Linear Buckling Analysis.

Mode	Eigenvalue	Tolerance
1	5.231908	0.0000
2	9.885741	0.0000
3	17.268686	0.0000
4	18.053742	0.0000
5	18.053742	0.0000
6	20.587083	0.0000
7	20.587083	0.0000
8	25.474009	0.0000
9	25.474009	0.0000
10	35.206405	0.0000
11	35.206405	0.0000
12	41.179249	0.0000
13	41.179249	0.0000
14	150.386791	5.2570×10^{-89}
15	150.386791	3.0782×10^{-89}
16	176.521376	8.9909×10^{-63}
17	176.521376	1.2754×10^{-62}
18	191.075212	4.7365×10^{-58}
19	191.075212	8.5190×10^{-47}
20	195.868943	3.6804×10^{-42}

5. Monitoring Results and Their Comparison with Numerical Simulation Results

5.1. Layout of Monitoring Points

Considering that, during the jacking process, the stress state of members changes with their position, using bonded strain gauges to measure member stresses would significantly increase the difficulty of construction. Therefore, only the nodal coordinates are monitored. Due to the complex spatial configuration of this project, measurement and positioning are difficult. If conventional total station surveying methods are used, it is very difficult for the operators to set up prisms above the structural members. To address this technical challenge, a new type of measuring device was designed, as shown in Figure 18. The bottom of the device consists of a hollow steel tray structure with built-in strong magnets, which can be firmly attached to the surface of the spherical node and are free to rotate

together with the node. Four circular levels are uniformly arranged around the tray to ensure that the prism remains precisely in a vertical position.

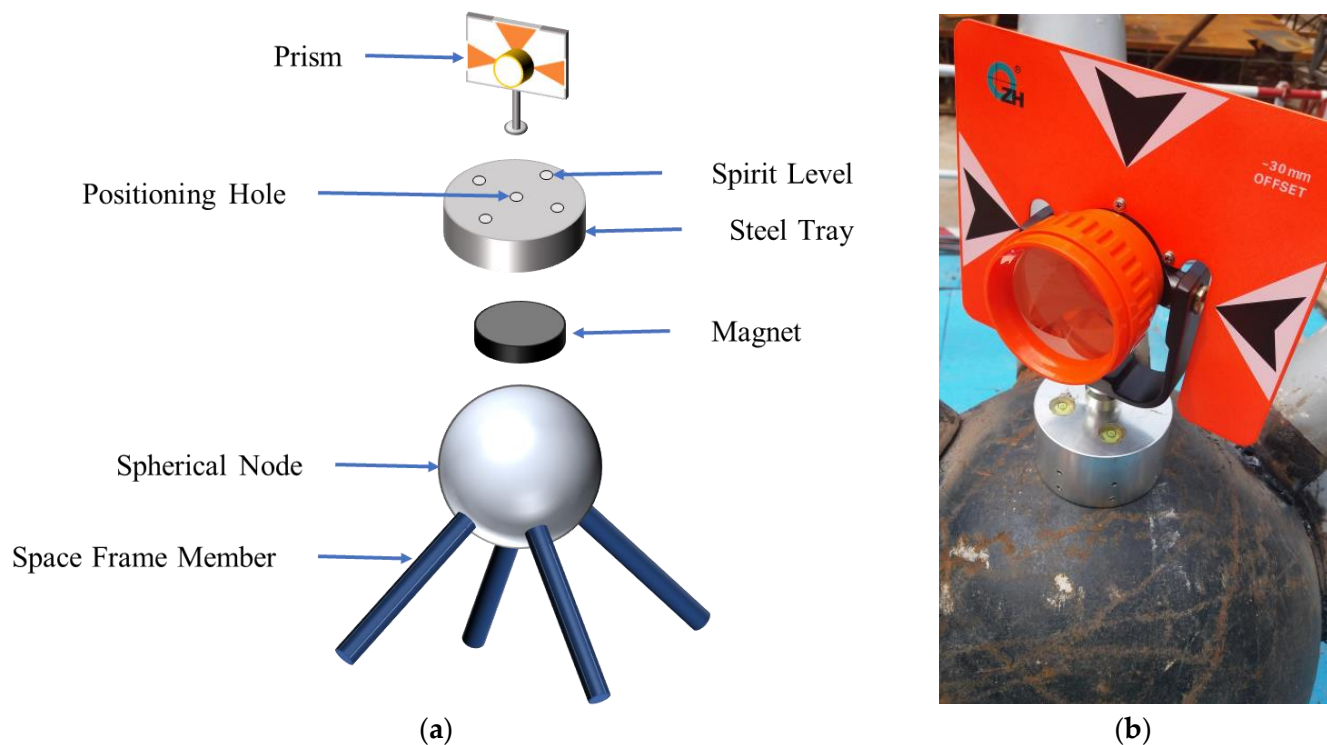


Figure 18. Schematic Diagram of the Measurement Instrument. (a) Schematic of the measurement instrument. (b) Field deployment of the measurement instrument.

During the ring-by-ring assembly and jacking of the space grid, four reference observation points are arranged in the surrounding concrete grandstand area to serve as the spatial benchmarks for the entire process. Test points are arranged at the bottom and top spherical nodes of the space grid. The layout of the monitoring points is shown in Figure 19. As shown in Figure 20, during construction, a total station is used to monitor in real time the displacement of the support measurement points, which together with the four reference observation points form a closed survey loop. By comparing the monitoring data with the numerical simulation results, the overall configuration of the space grid and the accuracy of synchronous jacking are effectively controlled, thereby ensuring safety during the transformation of the structural system. Onsite photographs of the observation point (a) and target point (b) are shown in Figure 21, illustrating the actual implementation of the monitoring process.

5.2. Displacement Monitoring Results and Comparison with Numerical Simulation Results

After the completion of a series of complex construction procedures, including ring-by-ring welding, jacking, and system transformation, the overall deformation state of the space grid structure agrees well with the numerical simulation results, indicating that the construction control measures are effective and that the global mechanical performance of the structure meets the design expectations. The monitoring data show that, except for the pair of measurement points 3–7, which exhibits an abnormal deflection deviation of 29 mm, the deviations of all other pairs of measurement points are controlled within 4 mm, and the deflections at all measurement points remain within the safety threshold. The detailed measured deflection values are listed in Table 3.

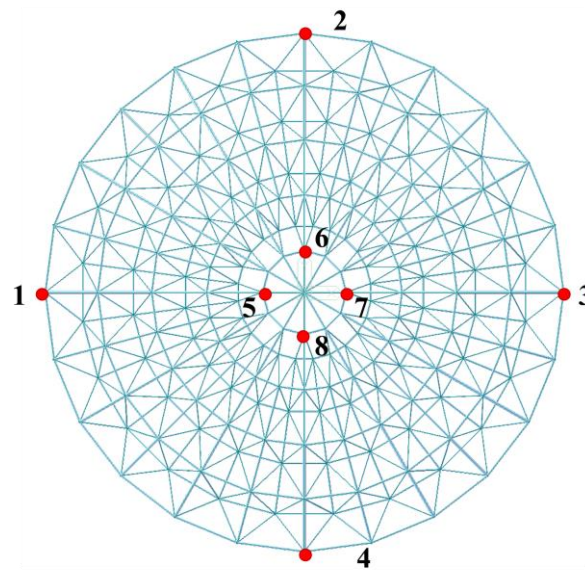


Figure 19. Layout of Deflection Monitoring Points.

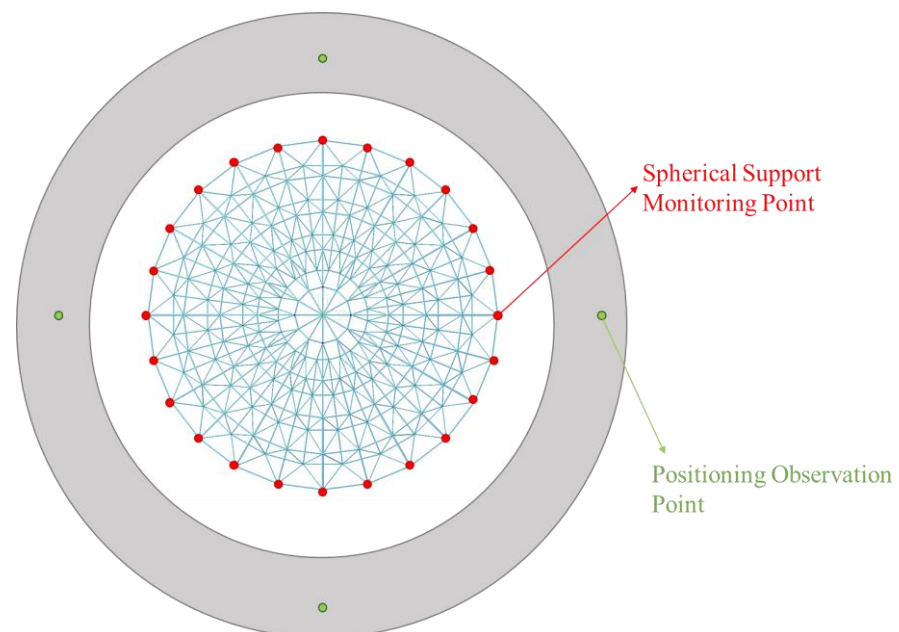


Figure 20. Layout of Positioning Observation Points for Space Frame Nodes.



(a) Observation point



(b) Target point

Figure 21. Onsite Monitoring Photograph.

Table 3. Measured Deflection Values of the Space Frame.

Number	Elevation (m)	Actual Height Difference (m)	Simulation Result (m)	Deflection Deviation (mm)
2 6	−1.334 6.664	7.998	8.002	−4
3 7	−1.367 6.664	8.031	8.002	29
1 5	−1.337 6.664	8.001	8.002	1
4 8	−1.334 6.664	7.998	8.002	−4

This localized discrepancy is likely due to slight jacking asynchrony, which was corrected during on-site alignment adjustments prior to structural closure. Post-construction inspections confirmed that members near points 3–7 are properly positioned and within allowable tolerances, confirming that the deviation was a transient construction-stage anomaly with no significant impact on long-term structural performance. The comparison between actual and simulated elevations of spherical nodes is provided in Table 4.

Table 4. Comparison of Actual Installation Elevation and Simulated Elevation of Spherical Nodes.

Monitoring Point	Measured Value (m)	Elevation from Numerical Analysis (m)	Deviation (mm)
1	5.674	5.675	1
2	5.675	5.675	0
3	5.68	5.675	−5
4	5.658	5.675	17
5	5.679	5.675	−4
6	5.655	5.675	20
7	5.691	5.675	−16
8	5.654	5.675	21
9	5.675	5.675	0
10	5.647	5.675	28
11	5.69	5.675	−15
12	5.674	5.675	1
13	5.682	5.675	−7
14	5.679	5.675	−4
15	5.667	5.675	8
16	5.659	5.675	16
17	5.677	5.675	−2
18	5.648	5.675	27
19	5.673	5.675	2
20	5.661	5.675	14
21	5.668	5.675	7
22	5.665	5.675	10
23	5.676	5.675	−1
24	5.664	5.675	11

After the jacking construction of the space grid structure is completed and all loads are transferred to the supports, the elevations of the supporting system are overall well controlled. The field measurement data indicate that the deviation values of the vast majority of measurement points are concentrated within the range of ± 10 mm, and the initial positioning state of the structure complies with the relevant code requirements. At

the same time, the settlements of the supports exhibit a certain spatial distribution pattern, among which a few supports such as No. 10 and No. 18 show slightly larger deviations, indicating a slight asynchrony in the lower jacking system. Therefore, the supports with deviations exceeding 15 mm need to be precisely fine-tuned in order to ensure that the final configuration of the structure meets the design requirements.

After the completion of the steel structure grid construction, the roof is then installed on top. Figure 22 shows the on-site image of the overall completion of the Han Culture Museum auditorium.



(a) Photo of completed steel space frame installation



(b) Photo of fully completed Great Hall building

Figure 22. Completion of Space Frame Lifting and Construction.

6. Conclusions

Taking the grand hall of the Han Culture Museum as the engineering case, this study carries out full-process monitoring of member stresses, support settlements and global displacements of the grid structure. Seven key construction conditions are selected from the entire erection and jacking process for focused analysis, and three different support schemes are compared by means of numerical simulation. The main conclusions are as follows:

- (1) By comparing the monitored and simulated member stresses at each construction stage, it is found that, for all schemes, the member stresses remain below the material yield strength. Among them, Scheme B achieves the best control of peak stresses under critical conditions and provides the largest structural safety margin, thereby confirming the necessity and effectiveness of adding auxiliary supports during the jacking process.
- (2) In terms of deformation control, Scheme B exhibits the most favorable performance, with a maximum vertical displacement of -5.25 mm, which is about 44% lower than that of Scheme A. The evolution of displacement is smooth and essentially linear, suggesting that the supporting system has a high stiffness and a well-defined force transfer mechanism, thus providing reliable assurance for both the global stability of the structure and the safety of the excavation during construction.
- (3) During construction, the region of stress concentration in the structure gradually shifts from the supports of the sixth ring to those of the fourth ring. In the course of the system transformation, the stresses in some members (such as web members No. 845 and No. 849) exhibit abrupt changes from compression to tension, which is a typical manifestation of load path reconstruction. Field measurements indicate that this process is globally controllable and does not trigger any structural instability.
- (4) In some local regions, such as the 3–7 measurement-point pair with an abnormal deflection of 29 mm, and at several supports where the elevation deviation is relatively

large (up to 28 mm), discrepancies in local support conditions are indicated. It is recommended that the bearing conditions in these areas be re-examined and finely adjusted where necessary, and that they be designated as priority zones for long-term structural health monitoring, so as to ensure the durability of the structural performance in service.

In summary, this study systematically reveals the mechanical evolution and critical control aspects of large-span double-layer space frames during staggered jacking construction by integrating full-process monitoring with numerical simulation. It verifies that a well-designed temporary support system plays a decisive role in ensuring structural safety and construction controllability. The proposed methodology and technical approach can serve as a reference and practical paradigm for jacking construction of large-span space frame roofs under similarly complex site conditions. Although demonstrated on a single case, the framework is adaptable to other long-span rigid spatial structures, such as double-layer latticed shells or large-span space grids, provided that local constraints, support conditions, and construction sequences are appropriately addressed.

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Appendix A

Appendix A.1 Member Section Properties

As shown in Figure A1, the members of the space grid structure are grouped by color according to their cross-sectional dimensions. Each color represents one section type labeled S1 through S14. The corresponding outer diameter, wall thickness, and sectional properties for all member types are listed in Table A1.

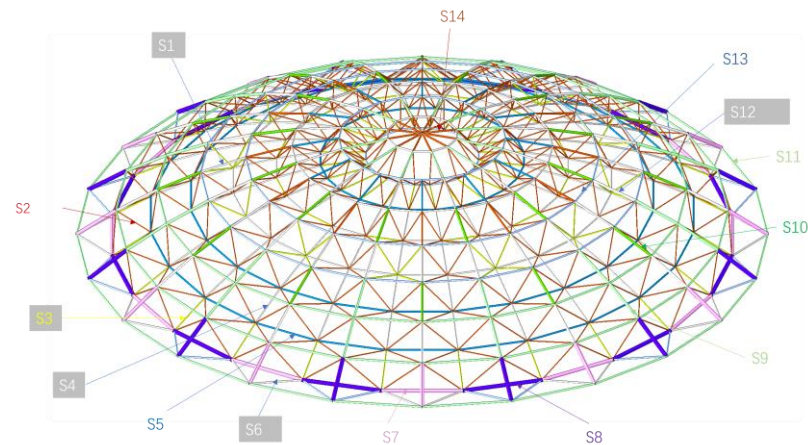


Figure A1. Section distribution of members in the space grid structure.

Table A1. Member Cross-Sections.

Section Type	Diameter (mm)	Thickness (mm)
S1	180	14
S2	88.5	4
S3	114	4
S4	140	4
S5	159	6
S6	159	8
S7	325	16
S8	351	18
S9	245	14
S10	219	12
S11	180	12
S12	219	14
S13	159	10
S14	219	12

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