



# Article Experimental Investigation and Numerical Simulation of a Levy Hinged-Beam Cable Dome

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**Abstract:** According to existing rigid roofing projects, a new structure called the Levy hinged-beam cable dome is proposed. By replacing the upper flexible cables with hinged beams, rigid plates can be installed overhead. To fulfill the requirements of integral tow-lifting construction, the setting criteria for the temporary hinged joints on ridge beams were presented. An 8-m diameter specimen was manufactured and monitored to investigate the structural configurations during the accumulative traction-hoisting construction process. Finally, the specimen was tested under full-span and half-span loading conditions, while a numerical model was built to verify the experimental values. The results show that in the early stages of traction-hoisting, the structure establishes the overall prestress and finds its internal force balance, while the entire structure is in a shape of " $\omega$ ". As the component's internal force increases during the construction steps, and the local deformations of the hinged beams gradually decrease, with the entire structure changing from " $\omega$ " to "m", and finally reach their designed states. Under full-span loads, large local deformations occurred at the HB-3 hinges, while the bending stresses of these hinged beams were relatively small. Under half-span loads, the loading part exhibits a downward appearance, while the unloading part exhibits upward deflection.

**Keywords:** levy hinged-beam cable dome; accumulative traction-hoisting construction; static test; numerical simulation

# 1. Introduction

In recent years, large spatial structures have been used in many stadiums [1,2], convention centers [3], the waiting halls of airports, and other public buildings. In addition to the requirements of diverse architectural functions and a beautiful appearance, modern large spatial structures need to fulfill the demand of a larger spanning ability, a higher utilization of the material mechanical properties, and lower construction costs. Therefore, research on light cable-stayed structures [4] needs to be actively carried out. Cable domes, based on the idea of tensegrity systems [5,6], could meet this trend. With the advantages of reasonable load-bearing systems, nonsupport construction technology, and the utilization of high-strength materials, a cable dome is an excellent spatial system and is considered a main form for future super-span structures. Therefore, cable domes have aroused widespread attention from the engineering and academic communities. The first cable dome was designed by Geiger [7] for the Seoul Olympics gymnastics in 1986. After that, cable domes have been extensively used in various large-span projects.

A conventional cable dome consists of flexible tensile cables and compressed struts [8]. Different from latticed shells [9] or spatial grids [10], initial pretension forces in the cables builds the out-of-plane stiffness of the entire structure to bear external loads and to limit the deformation [11]. According to the composition of materials, roofing forms can be divided into two types: flexible roofs with pretensioned membranes [12] and rigid roofs



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**Copyright:** © 2021 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (https:// creativecommons.org/licenses/by/ 4.0/). composed of glass plates [13] or other rigid plates. To make the appearance light and concise, the distance between the adjacent components of a cable dome is always designed to be large, which is a large challenge for increasing the spanning ability of roofing plates. Therefore, most existing projects use pretensioned membranes [14] as their roofing materials. However, the poor thermal insulation, poor environmental pollution resistance, and relatively higher price limit the application of pretensioned membranes under heavy loading conditions and thus limit the promotion of cable domes.

Rigid roofing components, such as glass panes [15], steel plates [16], reinforced concrete slabs [17], and composite sheets [18], have been widely used in steel frames. However, these components all have a small spanning capacity and need secondary structures to transfer the heavy roof load to the main structures [19], which places relatively higher demands on the bearing capacity of the secondary structures. Furthermore, for conventional cable domes, the connecting nodes between the main structure and the secondary structure should be placed on the top of the struts, not on the ridge cables, to avoid possible large local deformations, which is a major challenge for the construction industry.

Extensive research has been conducted to lay rigid plates on tensile structures. A suspended dome [20] is a hybrid structure, with a single-layer reticulated steel shell for laying rigid roofing plates and a cable-strut system for improving the structure's out-of-plane stiffness. A beam string structure [21] is a self-balancing space structure which is composed of a compression-bending beam, a tension string, some transverse braces, and several vertical struts to connect the beams. By combining a cable dome and a single-layer reticulated steel shell, Dong et al. [22] proposed a composite structure and applied it to the roof of the New District Science and Technology Exchange Center in Wuxi. Li et al. [23] employed a new hybrid structure consisting of a single-layer lattice shell and flexible cables. The newly proposed rib-type rigid cable dome [24] gives a scheme similar to the suspended dome, which replaces the flexible ridge cables with rigid rods, which are rigidly connected, to bear the heavy weight of the upper rigid roofing system. However, the upper rigid components of these structures are subjected to compression and bending under vertical loading. To improve the stability, the sectional areas of these rigid components are large, which significantly increase the self-weight of the main structure.

More recently, Ding et al. [25] proposed a Geiger-type ridge-beam cable dome, which changes the ridge cables of a conventional cable dome to rigid beams. However, to fulfill the requirements of integral tow-lifting construction, the ridge beams are articulated at both ends and the middle nodes, which leads to large local deformations under vertical loads. In addition, because the span of large-span roofs generally reaches up to tens or even hundreds of meters [26] and the span of the rigid roof panels is generally small, it is difficult to lay rigid roof panels directly between the adjacent tension beams.

Based on the idea of existing designs, a Levy hinged-beam cable dome, which is suitable for rigid roofing systems with a relatively larger sectional stiffness of the upper components, lower pretension forces, and smaller local deformations, is proposed in this paper. First, the characteristics of the Levy hinged-beam cable dome are introduced with the setting criteria for temporary hinged joints on the ridge beams. Then, an 8 m specimen is built and monitored to investigate the structural configurations during the different construction stages. Finally, by tests under a full-span load case and a half-span load case, the static behavior of the structure was obtained and compared with the results of a finite element analysis.

#### 2. Characteristics of the Levy Hinged-Beam Cable Dome

If the ridge cables of a typical Levy cable dome are replaced by rigid round steel pipes with hinged joints set on all of the two ends, then a Levy hinged-beam cable dome is generated. A Levy hinged-beam cable dome is composed of oblique cables, ring cables, struts, outer ridge hinged beams, and an outer compression beam, as shown in Figure 1a,b.

During the integral tow-lifting construction process, pretension forces are built in all the hinged beams, while temporary hinged joints are also set on the mid-span points of several hinged beams. These hinged beams are thus divided into two segments to release the freedom of the rotational degrees, generating a catenary system to improve the rotational deformation capacity. In this case, feasible setting criteria for the mechanism hinges on the hinged beams is necessary. After finishing the construction process, the temporary hinged joints of the ridge beams could be transferred to rigid ones, and the transformation process is illustrated in Figure 2.



Figure 1. Levy hinged-beam cable dome. (a) Axonometric drawing; (b) Sectional drawing.



**Figure 2.** Flowchart of changing the mid-span connection of a ridge beam from hinged to rigid. (**a**) Connect the two segments with a screw bolt; (**b**) Start the integral tow-lifting construction with the free rotation of the two segments; (**c**) After the integral tow-lifting construction, the two segments behave in a straight line; (**d**) Install the other screw bolt to change the hinged joint to a rigid one.

In normal uses, due to the large sectional stiffness of ridge hinged beams, Levy hingedbeam cable domes have a better structural stiffness and bearing capacity than conventional Levy cable domes. The anti-bending performance of the hinged beams greatly improves the local bearing capacity of the entire structure; thus, a rigid roofing system could be laid on them. Moreover, the hinged beams could bear both tension forces and bending moments, which makes them able to withstand heavy external loads with a relatively lower prestress level than conventional Levy domes.

# 3. Simplified Equations for the Optimal Distribution Mode of the Temporary Hinges of the Hinged Beams

As stated above, temporary hinges should be set on several ridge beams to implement integral tow-lifting construction. For a Levy hinged-beam cable dome with *n* rings of hinged beams, the mid-span deformation ratios of adjacent hinged beams (e.  $\Delta_1/\Delta_2$ ,  $\Delta_2/\Delta_3, \ldots, \Delta_{i-1}/\Delta_i, \ldots$ , and  $\Delta_{n-1}/\Delta_n$ ) should be compared to find the largest mid-span deformation,  $\Delta_m$ , as calculated in Equations (1) and (2). After that, temporary hinged joints are set in the mid-span of the hinged beams in the *m*-th ring, while the middle points of the other hinged beams remain rigid:

If *i*=2,

$$\frac{\Delta_1}{\Delta_2} = \frac{\sin \alpha_1 (\cot \alpha_1 + \cot \beta_1)}{2 \cos \alpha_2 \cos \varphi_{12}} \cdot \frac{L_1^2}{L_2^2}$$
(1)

If  $i \geq 3$ ,

$$\frac{\Delta_{i-1}}{\Delta_i} = \frac{\cos \alpha_{i-1} \cos \varphi_{i-1,i-2} (1 + \tan \alpha_{i-1} \cot \beta_{i-1})}{\cos \alpha_i \cos \varphi_{i-1,i}} \cdot \frac{L_{i-1}^2}{L_i^2}$$
(2)

where  $\Delta_i$  is the mid-span deformation of the hinged beams under vertical load in the *i*th ring, numbered from the inner strut/rigid ring;  $L_i$  is the distance between the two ends of the hinged beams in the *i*-th ring;  $\varphi_{i-1,i}$  is the horizontal angle between the member (e.g., node *i*-1 to node *i*) and the straight line from the middle point of the entire structure to node *i*-1;  $\alpha_i$  is the vertical angle between the horizontal plane and the hinged beams in the *i*-th ring; and  $\beta_i$  is the vertical angle between the horizontal plane and the oblique cables in the *i*-th ring. These symbols are displayed in Figure 3a,b.



Figure 3. Calculation graphs for the Levy hinged-beam cable dome. (a) Plane graph; (b) Profile graph.

#### 4. Design of the Test Model

# 4.1. Supporting System

To study the construction variation and static response of the Levy hinged-beam cable dome, an experimental model was designed with an 8-m diameter specimen, with eight pieces in the radius, and three pieces in the ring. The experimental model consists of hinged beams (HBs), oblique cables (OCs), ring cables (RCs), struts (STs), an inner rigid ring (IRR), and a supporting platform (SP). The plane, elevation, and photograph of the specimen are illustrated in Figure 4a–c, respectively. The rise-span ratio of the specimen is 0.1.

The SP consists of an outer compressive ring beam and columns. The outer compressive ring beam is the main supporting member of the new structure. The self-balanced internal force of the structure is transmitted to the ring beam through the upper HBs and the bottom OCs. The outer compressive ring beams should possess sufficient rigidity to bear the reaction forces. The cross section of the ring beam is 220 mm  $\times$  220 mm  $\times$  14 mm  $\times$  14 mm, and each segment of the ring beam is connected to the columns by four  $\Phi$ 20 screw bolts. To effectively connect the ring beam with the outer HBs and the outer Ocs, two ear plates are set at the top of each steel column. The HB-1s and the OC-1s are connected to the gusset plates, while each gusset plate is connected to the ear plates via a long screw connector and a pin, as shown in Figure 5a,b.



Figure 4. Test model of a Levy hinged-beam cable dome. (a) Plane view; (b) Elevation view; (c) Specimen photograph.



Figure 5. Details of the boundary nodes. (a) Support node; (b) Gusset plate.

# 4.2. Structural Members and Joints

The arrangement of the HBs is the main innovation of the Levy hinged-beam cable dome structure. All the HBs and struts used in this specimen are Q345B steel tubes with a section size of  $\Phi$ 32 × 3.5. Three kinds of hinged beams are contained in this structure, the inner hinged beam (HB-3), the middle hinged beam (HB-2), and the outer hinged beam (HB-1), as shown in Figure 6a,b.

According to the force transmission performance and the boundary conditions, 8 mm and 12 mm high-strength steel wire ropes are used for the OCs and RCs, respectively. Each steel wire rope has an adjustment sleeve at two ends and an adjustable length range of  $\pm 10$  mm, as displayed in Figure 7a,b.

The connecting joints of the struts can be divided into two kinds: the lower joints to connect the OCs and RCs and the upper joints to connect adjacent HBs. Steel tubes are welded with the ring beams to form the IRR. The upper and lower ring beams are evenly arranged with eight ear plates along the circumferential direction, and the HB-3s, OC-3s and IRR are connected via ear plates. The central axes of the HBs, cables, and STs are

designed to intersect at one point to reduce errors between the specimen and the simulation model. The details are illustrated in Figure 8, Figure 9, Figure 10.



(a)

Figure 6. Photographs of the HBs. (a) HB-3; (b) HB-1/HB-2.



(a)

(b)

Figure 7. Photographs of the cables. (a) OC; (b) RC.



(a)



Figure 8. Photographs of the struts. (a) ST-1; (b) ST-2.



(b) Figure 9. Photographs of the connecting joints. (a) Lower joint; (b) Upper joint.



Figure 10. Photographs of the IRR.

Details of specimen are shown in Table 1.

Table 1. Parameters of specimen components.

Name	Туре	Yield Strength/MPa	Specification/mm
HB	Steel tube	300	$\Phi 32  imes 3.5$
ST	Steel tube	300	$\Phi32 imes3.5$
IRR	Steel tube	300	$\Phi32 imes3.5$
RC-1	Wire rope	928	12
RC-2	Wire rope	928	12
OC-1	Wire rope	928	8
OC-2	Wire rope	928	8
OC-3	Wire rope	928	8

# 4.3. Measurement System and Measuring Point Arrangement

The stress and displacement of the structural nodes are the two main data points measured in this paper. Resistance strain gauges are used to measure the internal forces of the components, and strain gauges are attached to the front and back sides of each measuring component to eliminate errors caused by eccentric forces and temperature during the test. The internal forces of the RCs are measured by tension sensors, the cable forces of the Ocs are measured by a tensiometer, the nodal displacements are monitored by a total station, and a DH3816 static strain test system is adopted as the test data acquisition instrument. These devices are illustrated in Figure 11a–d.

Strain measuring point: 16 points on the HBs, of which eight are for HB-1s, four are for HB-2s, and four are for HB-3s. Axial force measuring point: 16 points on the OCs, including eight on OC-1s, four on OC-2s, and four on OC-3s. Eight points on the RCs, including four RC-1s, and four RC-2s. Displacement measuring point: 16 measuring points in total, which are four points on the top of the ST-1s, ST-2s, and IRR. These measuring points are listed in Figure 12.



Figure 11. Test and measurement instrument. (a) DH3816 static strain test system; (b) Tension sensor; (c) Tensiometer; (d) Total station.



**Figure 12.** Layout of the measuring points of the specimen. (**a**) Strain points for the hinged beams; (**b**) Axial force points for the OCs; (**c**) Axial force points for the ring cables; (**d**) Displacement points for the struts.

#### 4.4. Numerical Model

A numerical calculation model is constructed in ANSYS to simulate the cable force, nodal displacement, and component stress of the specimen.

The 3D two-node spar element (LINK8), displayed in Figure 13a, is used to simulate the STs, which bear both a uniaxial compressive force and a uniaxial tensile force. The 3D two-node cable element (LINK10), illustrated in Figure 13b, is used to simulate the Ocs and RCs, which bear only a uniaxial tensile force. The 3D two-node beam element (BEAM188), shown in Figure 13c, is used to simulate the HBs and IRR; this element includes the stress stiffness, shear-deformation effects, and large deflection and can simulate the mechanical properties of the HBs and IRR under loading. A single-node, concentrated mass element (MASS21), present in Figure 13d, is used to simulate the connecting joints between the cables and struts, which has three movement freedoms and three rotational freedoms.



**Figure 13.** Element types used in the numerical simulation. (**a**) 3D two-node tension-compression element, LINK8; (**b**) 3D two-node cable element, LINK10; (**c**) 3D two-node beam element, BEAM188; (**d**) Single-node, concentrated mass element, MASS21.

# 4.5. Calculation of the Initial Prestress

Because the self-weight of the beam-cable-strut members and joints is large and cannot be ignored, the actual prestress with the considered self-weight was calculated to control the forming state of the specimen. According to the mechanical properties of the specimen under an external load, the internal forces of the members are illustrated in Table 2.

The initial pretension in the numerical model is built by applying equivalent temperature differences to the cables, as listed in Equation (3):

$$F = E\alpha\Delta TA \tag{3}$$

where *F* is the tensile force of a cable, *E* is the elastic modulus of a cable, *A* is the area of a cable,  $\Delta T$  is the equivalent temperature difference of a cable, and  $\alpha$  is the temperature expansion coefficient of a cable. In this study,  $\alpha = 1.2 \times 10^{-5}$ .

Name	<b>Element Section</b>	Initial Pretension Force/N
HB-1	Beam188	19,878
HB-2	Beam188	7657
HB-3	Beam188	8456
ST-1	Link8	-9454
ST-2	Link8	-2758
RC-1	Link10	18,900
RC-2	Link10	6402
OC-1	Link10	14,558
OC-2	Link10	3750
OC-3	Link10	2341

Table 2. Component specifications and initial prestress of the specimen.

# 5. Experimental Study and Numerical Simulation

5.1. Accumulative Traction-Hoisting Construction Technology for the Levy Hinged-Beam Cable Dome

Accumulative traction-hoisting construction technology is an efficient method that possesses the characteristics of a non-bracket and small lifting force. The specific steps of accumulative traction-hoisting construction technology are as follows: (1) assembling the outer compression beam, lifting cables, HBs, and IRR, alternately, to the designed position of the construction site, then connecting the lifting cables to the outer compression beam, and connecting the IRR and the lifting cables to the HBs; (2) shortening the lengths of the lifting cables and traction cables to lifting the IRR for a short distance, and installing the OC-3s, ST-2s, RC-2, and OC-2s under the HB grid; (3) lifting the IRR for a short distance and installing the OC-1s, ST-1s and RC-1; (4) alternately lifting the IRR by lifting cables and traction cables until the outmost ends of HB-1s are connected to the outer compression beam; (5) removing the lifting cables and traction steps are shown in Figure 14.



**Figure 14.** The detailed process of accumulative traction-hoisting construction technology. (**a**) Install outer compression beam, lifting cables, HBs, and IRR, connect the lifting cables to the outer compression beam, and connect the IRR and lifting cables to the HBs; (**b**) Shorten the lengths of the lifting cables and traction cables to lift the IRR for a short distance, and installing the OC-3s, ST-2s, RC-2, and OC-2s under the HB grid; (**c**) Lift the IRR for a short distance, install the OC-1s, ST-and RC-1; (**d**) Lift the IRR by lifting cables and traction cables until the outmost ends of HB-1s are connected to the outer compression beam; (**e**) Remove lifting cables and traction cables; (**f**) Tension the OC-1s.

In the Levy hinged-beam cable dome specimen, four hoisters are symmetrically arranged in the middle of the eight ring beams around the periphery. Each hoister is welded to the ring beam at one end, and the other end is connected to the IRR through a wire rope to simulate a traction cable, as shown in Figure 15a. Each outermost end of HB-1 is connected with a gusset plate through a bolt, and each OC-1 is connected with a gusset plate through a bolt, and each OC-1 is connected with a gusset plate through the center of the gusset plate and the support sleeve, is provided at each support node to simulate lifting cables, as displayed in Figure 15b. The details of the adjustable stainless steel 304 basket screw are presented in Figure 15c.



Figure 15. Toolings of the traction-lifting and tensioning technology. (a) Hoister; (b) Gusset plate; (c) Adjustable stainless steel 304 basket screw.

For the specimen adopted in this study, the accumulative traction-hoisting construction technology is implemented as follows:

1. Traction and lifting

By tightening the nut of the adjustable screw, the length of the screw is adjusted, and HB-1s and the OC-1s are hauled. Meanwhile, the IRR is towed by the four hoisters. During this process, lifting plays the major role, while traction plays the supporting role to ensure the stability and synchronization of the entire structure. The adjustable screws are shortened in six steps, and the adjustment lengths are -40 mm, -40 mm, -40 mm, -40 mm, -40 mm, -40 mm, -50 mm, -50 mm, -50 mm, and -25 mm, respectively. After each step, the strain value of each measured member and the ground clearance of each measured node are recorded.

# 2. Tensioning and molding

The basket screws are adjusted six times to build the prestress of the overall structure, and the adjustment lengths are -40 mm, -40 mm, -40 mm, -40 mm, -15 mm, and -5 mm. After each step, the strain value of each measured member and the ground clearance of each measured node are recorded as well.

The actual construction process is listed in Figure 16a–f, and the analysis steps are listed in Table 3, where the symbol '+' means the added value of OC-1 during construction when compared with formed state.



**Figure 16.** Paragraphs of the actual construction process. (**a**) Assemble the specimen components at the test site; (**b**) Adjust the lengths of the adjustable screw; (**c**) Finish the traction and lifting process; (**d**) Adjust the lengths of basket screws; (**e**) Mold the entire structure; (**f**) Fine-tune the specimen.

Step		Original Length (mm)				
		<b>Traction Cables</b>	Lifting Cables	OC-1		
Initial Stage	0	220	220	+200		
	1	3950	180	+200		
	2	3900	140	+200		
Traction lifting	3	3850	100	+200		
macuon-mung	4	3800	60	+200		
	5	3750	20	+200		
	6	3725	0	+200		
Demolish traction cables and lifting cables	7	-	-	+200		
	8	-	-	+140		
	9	-	-	+100		
Tension	10	-	-	+60		
OC-1s	11	-	-	+20		
	12	-	-	+5		
	13	-	-	0		

Table 3. Analysis steps of the construction process and lengths of different cables after shortened.

#### 5.2. Results for Accumulative Traction-Hoisting Construction

Figure 17a–c depicts the vertical difference of ST-1, ST-2, and IRR, respectively. Figure 18 shows the local deformation of the mid-span temporary hinged joints. Figure 19a,b give the internal forces of the lifting cables and traction cables, respectively. Figures 20 and 21 give the internal forces of the Ocs and RCs, respectively. Figure 22a,b give the tensile stresses and bending stresses of the HBs during the accumulative traction-hoisting construction, respectively.

The results indicate that accumulative traction-hoisting construction technology could be used to realize the entire construction process of the Levy hinged-beam cable dome, and the numerical model ensures the accuracy of the simulation. In the early steps of tractionlifting, the structure establishes the overall prestress, accumulates stiffness, and finds its own internal force balance, while the entire structure is in a shape of " $\omega$ ". In these steps, the internal forces of the structural components are small, which magnifies the production deviation of the specimen and the measurement deviation of the testing instruments and thus leads to large deviations between the experimental values and simulation values. The maximum deviation of vertical displacement is 37.2 mm, which occurs at the top node of the IRR. As the internal force of the component increases in the following construction steps, the effect of these two deviations decreases, the configuration of the entire structure changes from the " $\omega$ " shape with both ends and midpoint sunken to the "m" shape with both ends and midpoint convex. In the last stage of tensioning, small variations occur in the vertical displacements of the struts, while the forces of the cables and beams increase rapidly, the local deformations of the hinged beams gradually decrease, the overall prestress of the structure is established, and the whole structure changes from the initial relaxed state to the designed state. After the tensioning is completed, the largest difference between the experimental values and simulation values is within 18%.



Figure 17. Average vertical difference of the top nodes of ST-1, ST-2, and IRR. (a) ST-1; (b) ST-2; (c) IRR.



Figure 18. Average local deformation of the mid-span temporary hinged joints.



Figure 19. Internal forces of tooling cables during construction. (a) Lifting cable; (b) Traction cable.



Figure 20. Internal forces of OC-s during construction. (a) OC-1; (b) OC-2; (c) OC-3.



Figure 21. Internal forces of the RCs during construction. (a) RC-1; (b) RC-2.



Figure 22. Stresses of HBs. (a) Tensile stress; (b) Bending stress.

# 5.3. Summary of the Static Loading Test

The static loading test contains two parts, namely, a full-span uniform loading test and a half-span uniform loading test. Specifically, a vertical load was added to the HBs by hanging sandbags. The loading patterns are listed in Figure 23a,b, and photographs of hanging sandbags are shown in Figure 24a,b. Note that the uniform load was added to the HBs in four steps and unloaded in four steps as well. The loading values are listed in Tables 4 and 5.



Figure 23. Elevation graphs of the static loading test. (a) Full-span; (b) Half-span.



(a)

(b)

Figure 24. Photographs of the static loading test. (a) Full-span; (b) Half-span.

	Table 4.	Loading	values f	for the	full-span	loading	test (unit:	: N).
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Loading Step	OC-1	OC-2	OC-3
1	370	233	200
2	740	466	400
3	1110	699	600
4	1480	932	800

Table 5. Loading values for the half-span loading test (unit: N).

Loading Step	Region	OC-1	OC-2	OC-3
1	Loading part	370	233	200
1	Unloading part	0	0	0
2	Loading part	740	466	400
2	Unloading part	0	0	0
2	Loading part	1110	699	600
3	Unloading part	0	0	0
4	Loading part	1480	932	800
4	Unloading part	0	0	0

# 5.4. Results of the Full-Span Static Loading Test

The variations in the cable forces with different loading steps of the full-span static loading test are listed in Table 6. The variations in the vertical displacement of the upper nodes on the struts, variations in the local deformation of the mid-span joints on the HBs, axial stresses of the HBs, and bending stresses of the HBs are illustrated in Figure 25a–d, respectively.

The results show that during the full-span loading process, the experimental forces of cables are close to the numerical forces, and the maximum error is within 12.5%. The experimental values of the beam stresses are basically consistent with the numerical values. Among them, the tensile stresses of the HBs decrease continuously with increasing upper load, while the bending stresses increase. The total stresses of the HBs do not exceed their yield strength; that is, the HBs are in the elastic stage during the loading steps.

		-					
Mea	suring Point No.	Before Load	Step 1	Step 2	Step 3	Step 4	Unloading
	Simulation value	18.9	19.10	19.34	19.74	20.60	18.90
RC-1	Experiment value	17.28	17.94	18.67	19.48	20.96	17.28
	Deviation (%)	-8.6%	-6.1%	-3.5%	-1.4%	1.7%	-8.6%
	Simulation value	6.40	6.44	6.49	6.60	6.88	6.40
RC-2	Experiment value	6.79	7.06	7.07	7.20	7.57	5.95
	Deviation (%)	6.0%	9.6%	8.9%	8.9%	10.0%	-7.1%
	Simulation value	14.56	14.80	15.12	15.47	15.76	14.56
OC-1	Experiment value	13.53	15.66	14.89	14.46	14.64	13.53
	Deviation (%)	-7.1%	5.8%	-1.5%	-6.5%	-7.1%	-7.1%
	Simulation value	3.75	3.813	3.93	3.99	4.133	3.75
OC-2	Experiment value	4.06	4.09	3.90	4.00	4.23	4.06
	Deviation (%)	8.3%	7.3%	-0.9%	0.4%	2.3%	8.3%
	Simulation value	2.34	2.26	2.23	2.12	2.130	2.34
OC-3	Experiment value	2.20	2.07	2.21	2.03	2.18	2.20
	Deviation (%)	-6.1%	-8.3%	-0.6%	-4.3%	2.2%	-6.1%

Table 6. Variations in the average cable forces of the full-span static loading test (unit: kN).



**Figure 25.** Results of the full-span static loading test. (a) Variations of vertical displacement of upper nodes on struts; (b) Variations of local deformation of mid-span joints on HBs; (c) Axial stresses of hinged beams; (d) Bending stresses of HBs.

Furthermore, the load-displacement curve has a good linear law. The experimental values of the structural displacements are generally larger than the numerical values, while

these two kinds of values are consistent, which can also be seen in Figures 17 and 18. Due to the small rise-span ratio of the specimen (i.e., 0.1), bit differences in axial forces of cables or stresses of hinged beams leads to the changing of component lengths, which enlarges the variations of overall configuration, and thus result in the relatively larger errors for displacement than for beam stresses. The main reason for the deviation is that there is a certain difference between the actual test model and the numerical model, and some errors exist in the instrument reading as well. During the whole process, large local deformations occur at the hinges of HB-3s. However, the bending stresses of these HBs are relatively small, which means that releasing the mid-span rotation constraints of the tensile beams could, to a certain extent, reduce the bending moment.

# 5.5. Results of the Half-Span Static Loading Test

The variations in the cable forces with different loading steps of the half-span static loading test are listed in Table 7. The variations in the vertical displacement of the upper nodes on the struts, variations in the local deformation of the mid-span joints on the HBs, axial stresses of the HBs, and bending stresses of the HBs are displayed in Figure 26, Figure 27, Figure 28, Figure 29, respectively.

The results show that the Levy hinged-beam cable dome is sensitive to asymmetric loads. Under half-span loads, the displacement of the brace node is greater than the fullspan effect, and the increase is more obvious. The maximum vertical node displacement occurs in the ST-2s of the loading part, while the STs in the unloading part exhibit an upward deflection, and the entire structure deforms asymmetrically centered on the IRR.

Large errors are observed when comparing the numerical values and experimental values of the internal forces of the cables and bending stresses of the HBs, with a maximum value of more than 20%. However, the errors of the internal forces of the other components are relatively smaller. For the components in the loading part, the internal forces of OCs and RCs increase, and the bending stresses of the HBs increase as well. For the components in the unloading part, the internal forces of HBs.



**Figure 26.** Vertical displacement of the upper nodes on the struts for the half-span static loading test. (**a**) Loading part; (**b**) Unloading part.



Figure 27. Mid-span joints on the HBs for the half-span static loading test. (a) Loading part; (b) Unloading part.

Measu	uring Point No.	Before Load	Step 1	Step 2	Step 3	Step 4	Unloading
Loading	Simulation value	18.90	20.23	21.56	22.59	24.07	18.90
Loading	Experiment value	17.29	18.42	19.97	20.38	22.16	17.43
part KC-1	Deviation (%)	-8.5	-8.9	-7.4	-9.7	-7.9	-7.8
Unloading	Simulation value	18.9	17.57	16.68	16.08	15.29	18.90
Dinoaunig	Experiment value	17.33	16.40	15.95	15.17	13.97	17.12
part KC-1	Deviation (%)	-8.3	-6.6	-4.4	-5.7	-8.7	-9.4
Loading	Simulation value	6.40	6.78	7.28	7.59	8.20	6.40
Dort RC 2	Experiment value	6.01	6.22	6.47	6.74	6.99	6.60
part KC-2	Deviation (%)	-6.2	-8.2	-11.2	-11.1	-14.8	3.0
Unloading	Simulation value	6.40	6.14	5.84	5.62	5.26	6.40
part RC 2	Experiment value	5.89	5.77	5.63	5.75	5.79	5.84
part KC-2	Deviation (%)	-8.0	-5.9	-3.6	2.3	9.9	-8.8
Loading	Simulation value	14.56	14.90	15.31	15.66	16.10	14.56
part OC-1	Experiment value	13.53	14.69	15.40	15.42	17.26	13.66
partoci	Deviation (%)	-7.1	-1.4	0.56	-1.56	7.2	6.9
Unloading	Simulation value	14.56	14.47	14.38	14.35	14.22	14.56
part OC-1	Experiment value	13.54	13.08	15.03	15.34	15.24	13.33
part OC-1	Deviation (%)	-7	-9.6	4.5	6.9	7.2	-8.4
Loading	Simulation value	3.75	4.07	4.49	4.73	5.25	3.75
nort OC 2	Experiment value	4.11	4.51	4.73	5.06	4.90	4.10
part OC-2	Deviation (%)	9.6	10.8	5.4	6.9	-6.8	9.3
Unloading	Simulation value	3.75	3.50	3.20	2.99	2.62	3.75
	Experiment value	3.91	3.74	3.43	2.93	2.49	3.86
part OC-2	Deviation (%)	4.3	6.8	7.4	-2.3	-5.3	2.8
Loading	Simulation value	2.34	2.36	2.44	2.43	2540	1.79
Loading	Experiment value	2.21	2.06	2.25	2.44	2682	1.94
part OC-5	Deviation (%)	-5.7	-12.8	-7.5	0.6	5.6	8.6
Unloading	Simulation value	2.34	2.24	2.14	2.04	1955	2.34
part OC-3	Experiment value	2.19	2.44	2.24	1.89	1728	2.09
part OC-5	Deviation (%)	-6.5	8.9	4.5	-7.6	-11.6	-10.9

Table 7. Variations in the average cable	forces of the half-span static	loading test (unit: kN).
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Figure 28. Axial stresses of the HBs for the half-span static loading test. (a) Loading part; (b) Unloading part.



Figure 29. Bending stresses of HBs for the half-span static loading test. (a) Loading part; (b) Unloading part.

For the mid-span hinged joints of the HBs, the local deformation in the loading part of the half-span loading test is larger than that of the full-span loading test, while the local deformation in the unloading part is relatively smaller. The maximum local deformation occurs at the mid-span of the HB-3s, with relatively larger values for experimental results than for the numerical results.

After unloading the half-span loads, the maximum displacement of the structure is larger than that before loading, which means that residual deformation exists. This residual deformation is mainly caused by the movement of bolts in holes of connecting joints, after checking the experimental model after the test.

# 5.6. Error Analysis

The measured value of the test is in good agreement with the finite element analysis value, but some errors still exist. The errors are mainly caused by the following aspects:

- 1. The friction between adjacent components, including the relative movement of the RCs in the bottom hole of the STs, the pivoting friction between the ear plates of HBs and the ear plates of STs, etc.
- 2. The manufacturing deviation of the test model, including the welding positioning of the ear plates, the blanking length of the member, the processing of the HBs, the installation of the HBs and cable connecting joints, etc.
- Measurement deviation caused by instruments. During the test, a certain drift inevitably exists in the reading of the static strain gauges. In addition, nodal displacements are collected by a total station, which will generate errors from the instrument alignment and target eccentricity.
- 4. Discrepancy between the numerical model and the test model. Although the dead weights of all the members and connecting nodes are considered in the numerical model, the additional weight of the test model (i.e., ear plates, screw bolts, cable head) are difficult to accurately simulate. Moreover, the forces of the cables at the symmetrical position of the numerical model should be consistent. However, due to the initial defects (i.e., manufacturing error, installation error, device error), the uniformity of the prestress distribution is disrupted, which enlarges the deviation between the simulated values and experimental values and affects the accuracy of the test.

# 6. Comparative Analysis of a Levy Hinged-Beam Cable Dome and a Geiger-Type Ridge-Beam Cable Dome

Due to that the Levy hinged-beam cable dome and the Geiger-Type Ridge-Beam cable dome [25] both replace the flexible ridge cables with hinged tensile beams, it is needed to perform a comparative analysis to investigate the mechanical behaviors of these two structures. The Geiger-Type Ridge-Beam cable dome model is shown in Figure 30, which shares the same size and parameters as that of the Levy hinged-beam cable dome model in Figure 4 and Table 1, respectively. The initial prestress of the Geiger-Type Ridge-Beam cable dome model is displayed in Table 8, in which the internal force of RC-1 is almost same as that of the Levy hinged-beam cable dome in Table 2 to obtain a similar prestress level.



Figure 30. Geiger-Type Ridge-Beam cable dome model.

Name	<b>Element Section</b>	Initial Pretension Force/N
HB-1	Beam188	21,332
HB-2	Beam188	13,935
HB-3	Beam188	10,875
ST-1	Link8	-7362
ST-2	Link8	-2764
RC-1	Link10	18,812
RC-2	Link10	8397
OC-1	Link10	16,216
OC-2	Link10	7004
OC-3	Link10	3031

Table 8. Initial prestress of the Geiger-Type Ridge-Beam cable dome model.

The Levy hinged-beam cable dome model and the Geiger-Type Ridge-Beam cable dome model were both tested under the half-span static loading condition, listed in Table 7, to compare their static responses. Figure 31, Figure 32, Figure 33, Figure 34, Figure 35 display the vertical displacement of the upper nodes on the struts, variations in the local deformation of the mid-span joints on the HBs, axial stresses of the HBs, bending stresses of the HBs, and cable forces, respectively. According to the results shown in Figure 31, Figure 32, Figure 33, Figure 34, Figure 35, the following facts can be identified:

- 1. With an increasing number of the asymmetric vertical loads, the deformations and the cable forces of the Geiger-Type Ridge-Beam cable dome model both vary similarly with that of a Levy hinged-beam cable dome model.
- 2. Under the same asymmetric vertical load, the variations of both the vertical displacements and the local deformations of the Geiger-Type Ridge-Beam cable dome model are relatively larger, while the variations of the cable forces are relatively smaller.



**Figure 31.** Comparative results of the vertical displacement on the upper nodes of the struts. (**a**) Loading part; (**b**) Unloading part.



**Figure 32.** Comparative results of the local deformation on the mid-span joints of the HBs. (**a**) Loading part; (**b**) Unloading part.



Figure 33. Comparative results of the axial stresses of the HBs. (a) Loading part; (b) Unloading part.



Figure 34. Comparative results of the bending stresses of the HBs. (a) Loading part; (b) Unloading part.



Figure 35. Comparative results of the cable forces. (a) Loading part; (b) Unloading part.

Therefore, compared with the Geiger-Type Ridge-Beam cable dome, the Levy hingedbeam cable dome significantly increases the cable forces and reduces the deformations under the same loading conditions, which makes it able to resist larger vertical loads with a relatively higher integral rigidity and a relatively lower out-of-plane displacement.

# 7. Conclusions

In this study, a new type of spatial structure called a Levy hinged-beam cable dome was proposed. This structure replaces the upper components of the conventional cable dome for HBs, and thus, rigid plates can be installed overhead. To fulfill the requirements of integral tow-lifting construction, the setting criteria for the temporary hinged joints on ridge beams were presented. In addition, an 8-m diameter specimen was manufactured and monitored to investigate the structural configurations in the different steps of the accumulative traction-hoisting construction process and to obtain the structural responses under full-span and half-span uniform loading conditions. A numerical simulation model was built to verify the experimental values. Finally, a comparative analysis was conducted to investigate the mechanical behaviors of Levy hinged-beam cable dome and Geiger-type rigid-beam cable dome. The following conclusions are drawn:

- (1) In the early stages of traction-lifting, the structure established overall prestress, accumulates stiffness, and found its own internal force balance. As the internal force of the component increased in the following construction steps, the local deformations of HBs gradually decreased, the configuration of the entire structure changed from the "w" shape with both ends and midpoint sunken to the "m" shape with both ends and midpoint convex. In the last stage of tensioning, the forces of the cables and beams increased rapidly, the local deformations of the hinged beams gradually decreased, the overall prestress of the structure was established, and the whole structure changed from the initial relaxed state to the designed state.
- (2) The tensile stresses of the HBs decrease continuously with the increasing of full-span symmetrical load, while the bending stresses increase. The total stresses of the HBs do not exceed their yield strength, which ensures the stability of these components. Large local deformations occurred at the hinges of the HB-3s, but the bending stresses of these hinged beams were relatively small, which means that releasing the mid-span rotation constraints of tensile beams could, to a certain extent, reduce the bending moment.
- (3) The Levy hinged-beam cable dome is sensitive to asymmetric loads. Under half-span loads, the displacement of the brace node is greater than the full-span effect. The loading part exhibited a downward deflection, while the unloading part exhibited an upward deflection, which made the entire structure deform asymmetrically centered on the IRR. After unloading the half-span loads, the maximum displacement of the structure is larger than that before loading, which means that residual deformation exists.
- (4) Experimental values were in accordance with the simulated ones, while errors still existed. These errors mainly came from the friction and between adjacent components, the manufacturing deviation of the test model, the measurement deviation caused by instruments, and the discrepancy between the numerical model and the test model.
- (5) Compared with the Geiger-Type Ridge-Beam cable dome, the Levy hinged-beam cable dome is able to resist larger vertical loads with a relatively higher global stiffness and relatively lower deformations.

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