

STABILIZING CABLE DOMES

JOSEP LLORENS

Barcelona School of Architecture (ETSAB), Universitat Politècnica de Catalunya
 Diagonal, 649 - 08028 Barcelona, Spain - E-mail: ignasi.llorens@upc.edu

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Summary. Despite their light weight and efficiency, cable domes have been relatively little used due to the difficulties of stabilisation and installation. That is why an adjustment method has been developed, starting from the desired shape, counteracting compressions and limiting displacements by pre-stressing. On the other hand, to overcome the difficulties that arise when it comes to assembly, some adopted solutions are presented.

1 INTRODUCTION

Despite the recognised advantages of the cable dome, its use has not been widespread, except for the initial impulse given by D. Geiger [1]. A few examples are shown in figures 1 to 11 and table 1. This limited use of such an efficient structure can be attributed to the difficulties of stabilisation and installation [2], as well as the impact of the compression ring, which dramatically increases the amount of structure required. The need for stabilisation starts at the design stage because ridges, diagonals and hoops are cables in tension, meaning that they have to be pre-stressed to achieve rigidity, find the form, withstand the loads and limit deformations. That is why an adjustment method has been developed, starting from the desired shape and counteracting compressions by pre-stressing. Structural decomposition [3] and the equivalent simplified planar model [15] methods have been applied.

Cable dome	Year	Location	Span (m)	R	L	F	S
Gymnastics Hall	1986	Seoul	120	16	3	01	[5]
Fencing Hall	1986	Seoul	90	16	2	06	[5]
Redbird Arena	1988	Normal	76,8x91,4	24	1	05	[6]
Tropicana Field (threatened)	1989	St.Petersburg	210	24	4	09	[7]
Amagi Dome	1991	Amagi Yugashima	43	16	1	04	[8]
Georgia Dome (demolished)	1992	Atlanta	233,5x186	26	3	08	[9]
Tao-Yuan Arena	1993	Taoyuan	120	16	3	07	[6]
Crown Coliseum	1994	Fayetteville	99,70	18	3	02	[10]
Bifid Tension Dome	2004	Barcelona	20	12	2	10	[11]
Ciudad de La Plata Stadium	2012	La Plata	239,09x190,39	24	3	03	[12]
SMS Campus	2023	Mönchengladbach	80	20	3	11	[13]

Table 1: Some built cable domes (R: ridges - L: loops - F: figure - S: source)

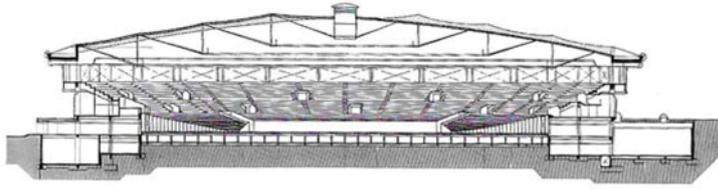


Fig.01: Gymnastics Hall

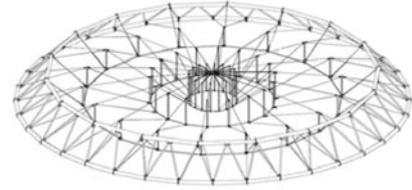


Fig.02: Crown Coliseum

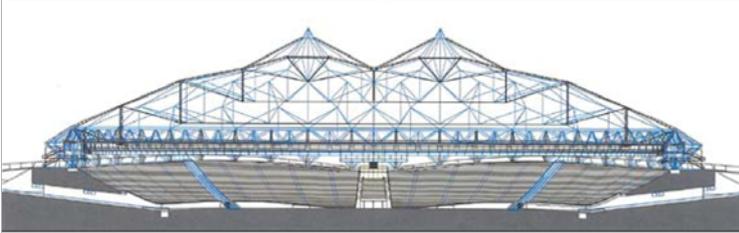


Fig.03: Ciudad de La Plata Stadium

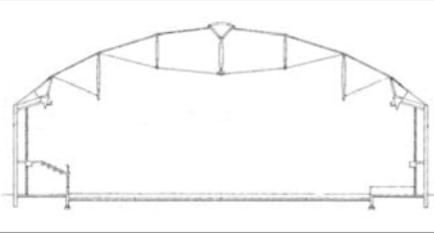


Fig.04: Amagi Dome

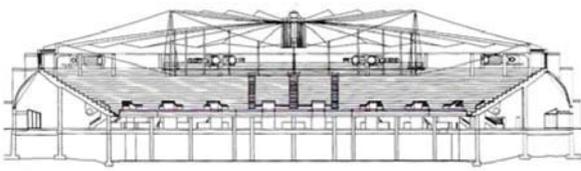


Fig.05: Redbird Arena



Fig.06: Fencing Hall

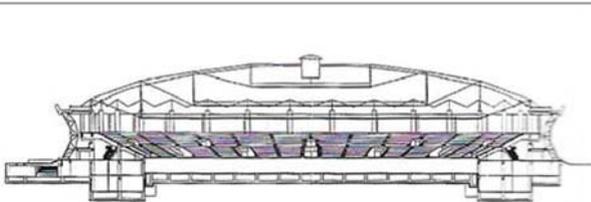


Fig.07: Tao-Yuan Arena



Fig.08: Georgia Dome



Fig.09: Tropicana Field

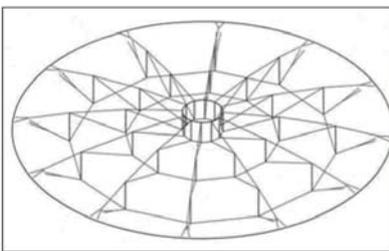


Fig.10: Bifid Tension Dome



Fig.11: SMS Campus

2 PRE-STRESS ADJUSTMENT

A 10 steps procedure has been followed to find the initial state of pre-stress from the initial geometry, including different loading conditions and avoiding instability. The proposed methodology is presented below, together with an example that illustrates its application. Figure 12 shows a radial cable dome in section, plan and perspective views together with its characteristics. Other options are possible taking into account the influence of the critical design parameters such as span, pre-stress, number of sectors, hoops, depth and rise discussed in S.Krishnan, 2015 [4].

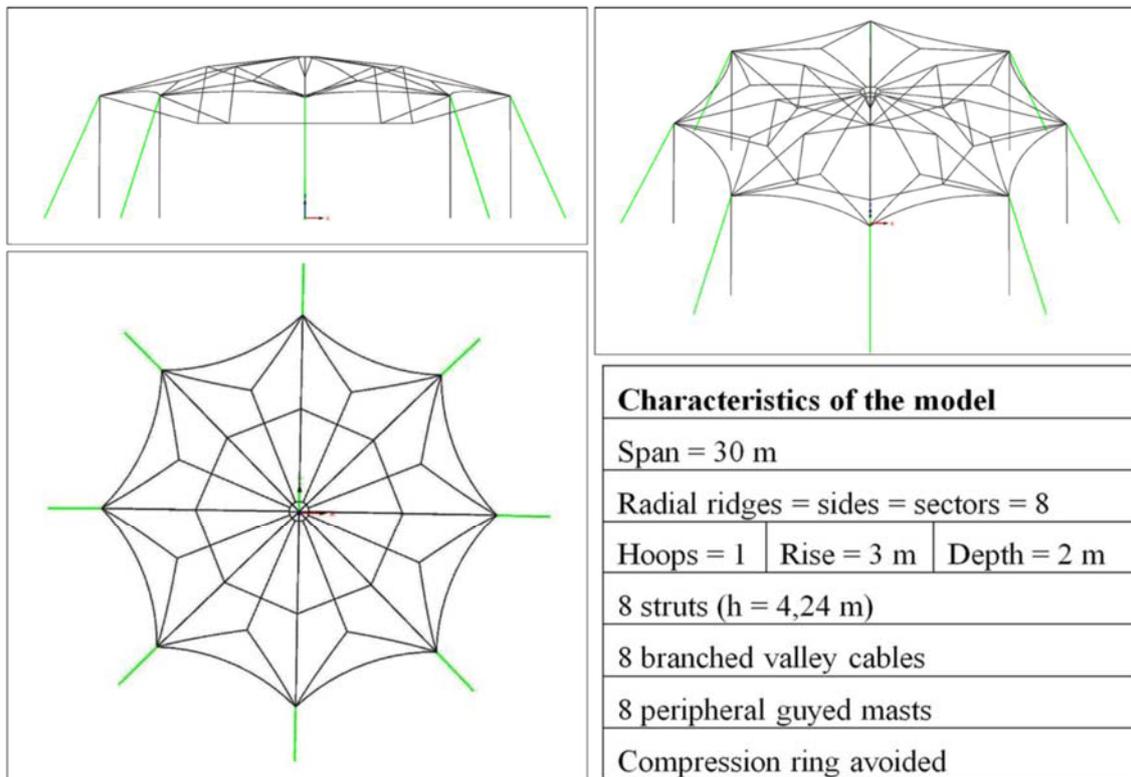
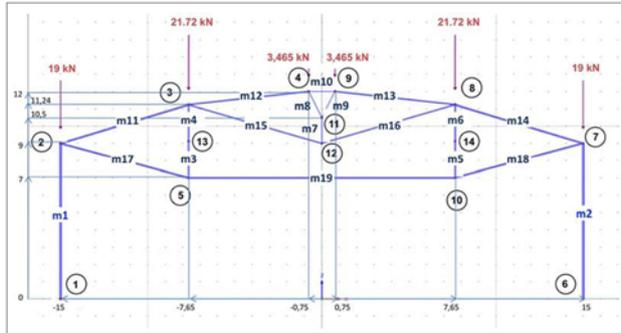


Fig.12: Example of application

2.1 Step 1: equivalent two-dimensional truss model

In order to estimate the initial pre-stress forces and preliminary member sizing, an equivalent two dimensional rigid model [15] is established changing cables by CHS and replacing the hoop by a horizontal bar in the 2D model (figure 13). Self-weight and $0,5 \text{ kN/m}^2$ snow load are included. The distribution of the snow load has been made according to the tributary surface of each of the roof nodes: 2, 3, 4, 9, 8 and 7 (figure 14). After some iterations, the results shown in figure 13 are obtained. Note that members 1,2 (masts), 3 to 6 (struts), 10 (crown), 11 to 14 (ridges) and 15,16 (inner diagonals) are compressed. Stress

ratios (existing stress/limit stress) may appear small due to buckling. They will disappear pre-stressing in ridges and diagonals.



Two-dimensional model				
n°	Member	Section	N (kN)	Ratio
1-2	Masts	Ø150x5	-73,06	0,16
3 to 6	Struts	Ø100x3	-18,18	0,16
7	Central strut	Ø50x3	3,61	0,03
8-9	Branches	Ø50x3	1,94	0,26
10	Crown	Ø50x3	-68,14	0,71
11-14	Ext. ridges	Ø120x3	-75,43	0,34
12-13	Int. ridges	Ø100x3	-68,00	0,43
15-16	Int. diag.	Ø90x3	-5,10	0,12
17-18	Ext. diag.	Ø120x3	74,67	0,31
19	Loop	Ø120x3	72,16	0,36

Fig.13: Step 1. Equivalent 2D truss

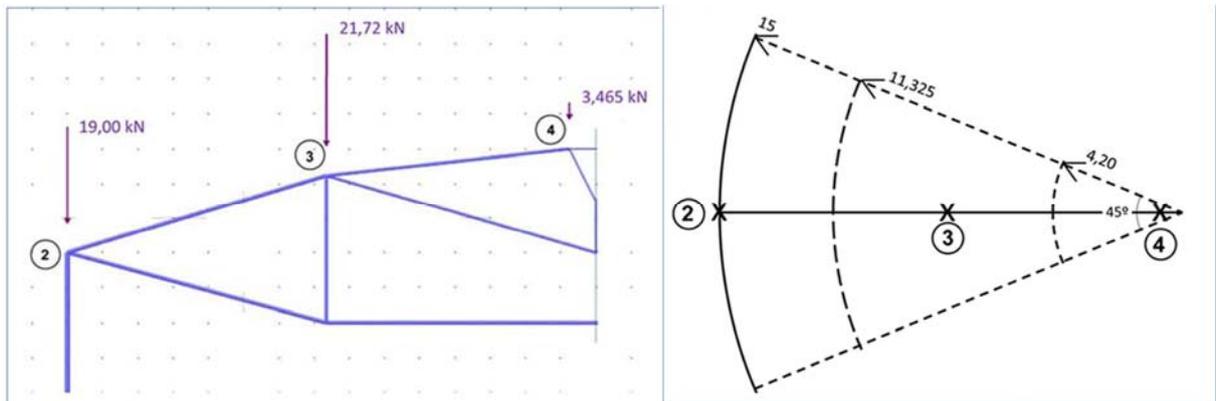


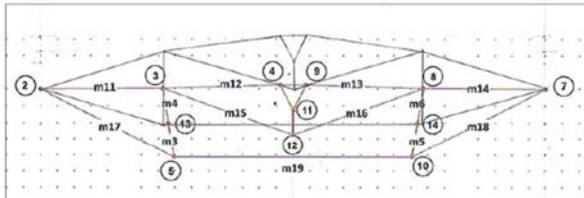
Fig.14: Snow load distribution. Nodes 2-7: $\Pi \cdot (15^2 - 11,325^2) \cdot 45 \cdot 0,5 / 360 = 19,00$ kN. Nodes 3-8: $\Pi \cdot (11,325^2 - 4,2^2) \cdot 45 \cdot 0,5 / 360 = 21,72$ kN. Nodes 4-9: $\Pi \cdot 4,2^2 \cdot 45 / 360 = 3,465$ kN

2.2 Step 2: introducing pre-stress

With the method of structural decomposition, otherwise applied by M.Ding et al. 2018 [3], the entire structure is sub-divided, dispensing with the cable-stayed supporting masts. Ridges, diagonals and hoop are replaced by cables. They are pre-stressed with values opposite to those of the compressions obtained in the previous step. Figure 15 shows the results of the form-finding. The compressions are overcome, cables don't go slack but the displacement values are not acceptable: up to 2.772,7 mm = span/11! More/different pre-stresses are needed.

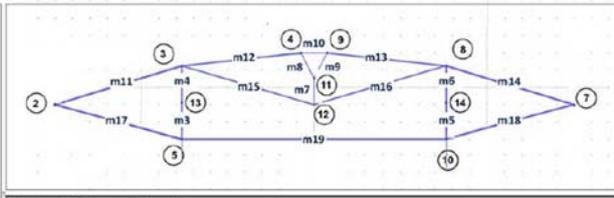
2.3 Step 3: limit the displacements

The displacements are reduced re-arranging the pre-stresses considering that pre-stressed diagonals counteract vertical displacements and pre-stressed ridges and loops control the verticality of the struts. Figure 16 shows the results of the form-finding. A maximum displacement of 65 mm = span/459 is achieved. Note that the diagonals lift the structure.



Introducing pre-stress T				
nº	Member	Section	T (kN)	N (kN)
3 to 6	Struts	Ø100x3		-0,5
7	Central strut	Ø50x3		-3,54
8-9	Branches	Ø50x3		-1,84
10	Crown	CableØ30	68,14	69,75
11-14	Ext. ridges	CableØ30	75,43	73,97
12-13	Int. ridges	CableØ30	68,00	69,21
15-16	Int. diag.	CableØ30	5,10	5,18
17-18	Ext. diag.	CableØ30	1,00	1,08
19	Loop	CableØ30	1,00	0,88

Fig.15: Step 2. Introducing pre-stress T

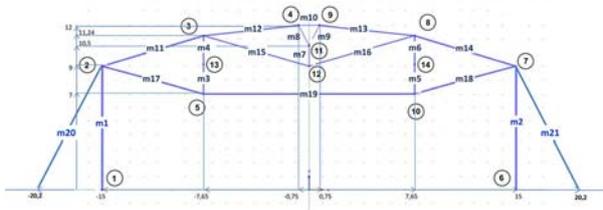


Limit the displacements				
nº	Member	Section	T (kN)	N (kN)
3 to 6	Struts	Ø100x3		-20,82
7	Central strut	Ø50x3		-11,14
8-9	Branches	Ø50x3		-6,25
10	Crown	CableØ30	51,4	51,25
11-14	Ext. ridges	CableØ30	70,60	70,94
12-13	Int. ridges	CableØ30	49,00	48,74
15-16	Int. diag.	CableØ30	20,00	19,91
17-18	Ext. diag.	CableØ30	80,70	80,57
19	Loop	CableØ30	78,00	78,12

Fig.16: Step 3. Limit the displacements 2D

2.4 Step 4: check the complete truss

Cable-stayed masts are re-introduced (figure 17). Note the increase from 73,06 kN to 251,58 kN in their axial force due to the pre-stress. In addition, the ties are subjected to a tensile stress of 291.14 kN.



Check the complete truss				
nº	Member	Section	T (kN)	N (kN)
1-2	Masts	Ø200x5		-251,58
3 to 6	Struts	Ø100x3		-20,87
7	Central strut	Ø50x3		-11,17
8-9	Branches	Ø50x3		-6,27
10	Crown	CableØ30	51,4	51,27
11-14	Ext. ridges	CableØ30	70,60	70,95
12-13	Int. ridges	CableØ30	49,00	48,74
15-16	Int. diag.	CableØ30	20,00	19,91
17-18	Ext. diag.	CableØ30	80,70	80,58
19	Loop	CableØ30	78,00	78,11
20-21	Ties	RDØ36		291,14

Fig.17: Step 4. Complete truss (T: pre-tension)

Add the self weight and the snow load				
nº	Member	Section	T (kN)	N (kN)
1-2	Masts	Ø250x5		-376,06
3 to 6	Struts	Ø100x3		-44,47
7	Central strut	Ø50x3		-10,93
8-9	Branches	Ø50x3		-6,09
10	Crown	CableØ30	51,4	3,21
11-14	Ext. ridges	CableØ30	70,60	20,32
12-13	Int. ridges	CableØ30	49,00	0,89
15-16	Int. diag.	CableØ30	20,00	18,79
17-18	Ext. diag.	CableØ30	80,70	162,96
19	Loop	CableØ30	78,00	157,79
20-21	Ties	RDØ36		352,49

Fig.18: Step 5. With self-weight and snow (2D)

2.5 Step 5: add the self weight and the snow load

Including the self-weight and the aforementioned 0,5 kN/m² snow load (figure 18), the masts and ties are even more loaded, the inner ridge becomes much looser and the outer diagonals and the loop are heavily overloaded.

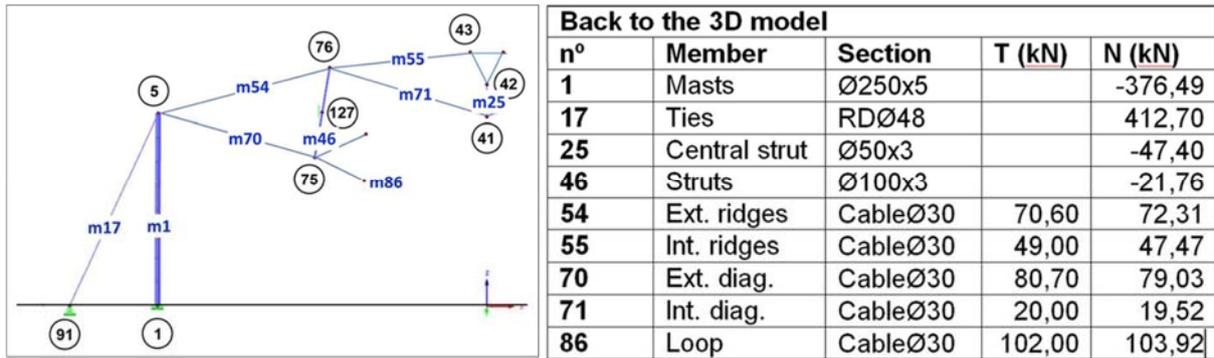


Fig.19: Step 6. Back to the 3D model

2.6 Step 6: Back to the 3D model

According to S.Krishnan 2015 [4], the hoop force in 3D = **T** can be determined from the following relationship:

$$T = \frac{T_R}{2 \times \text{SIN}\left(\frac{\pi}{n}\right)} = \frac{78}{2 \times \text{SIN}\left(\frac{\pi}{8}\right)} = 102 \text{ kN}$$

where **T_R** = hoop cable force in 2D; **n** = number of sides of the 3D hoop. Here n = 8 (figure 12). And back to the 3D model: the radial cable dome established in the step 1 is submitted to the pre-tension forces obtained so far. Results of the form-finding are summarized in figure 19. Maximum displacement (node 76) is 500,9 mm = span/60, so that the pre-stress has to be adjusted again. To achieve this, it is useful to know the effect of pre-stressing each member on the displacement of the nodes, as shown in the figure 20. The process of re-arrangement of the pre-stress consists of all displacements being less than 10 mm (figure 21).

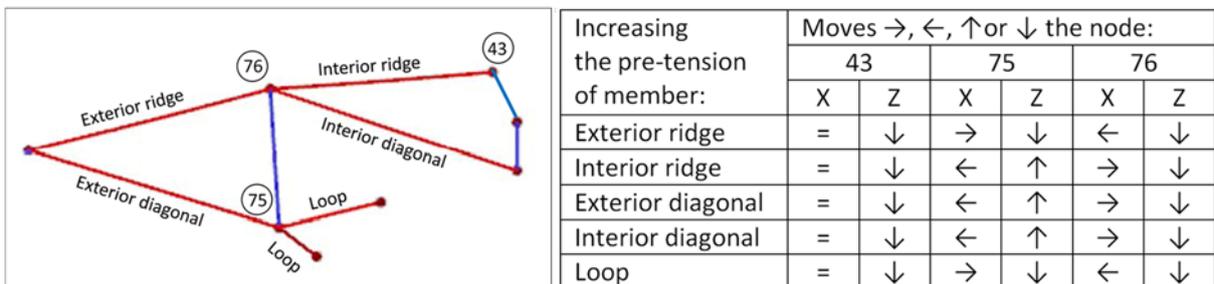


Figure 20: Effect of increasing the pre-tension of each member on the displacements of significant nodes.

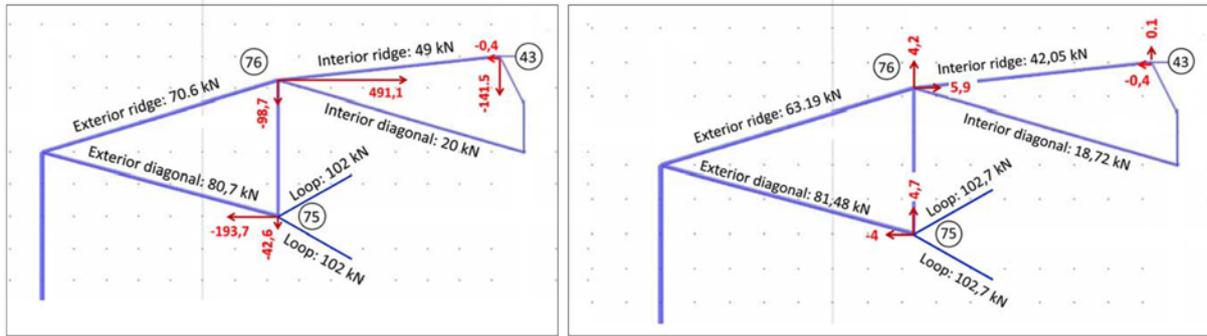


Figure 21: Re-arrangement of the pre-stress from step 6 (left) to step 7 (right). All displacements are reduced to less than 10 mm.

2.7 Step 7: limit the displacements (3D)

A re-arrangement of the pre-stress leads to the reduction of the maximum displacement to 218,9 mm = span/137 (figure 22).

Limit the displacements 3D					Add the self weight and the snow load 3D				
nº	Member	Section	T (kN)	N(kN)	nº	Member	Section	T (kN)	N (kN)
1	Masts	Ø250x5		-360,85	1	Masts	Ø250x5		-472,23
17	Ties	RDØ48		395,56	17	Ties	RDØ48		437,81
25	Central strut	Ø50x3		-42,19	25	Central strut	Ø50x3		-44,69
46	Struts	Ø100x3		-21,41	46	Struts	Ø100x3		-45,29
54	Ext. ridges	Cable Ø30	63,19	63,14	54	Ext. ridges	CableØ30	63,19	20,87
55	Int. ridges	Cable Ø30	42,05	42,10	55	Int. ridges	CableØ30	42,05	0,00
70	Ext. diag.	Cable Ø30	81,48	81,44	70	Ext. diag.	CableØ30	81,48	158,79
71	Int. diag.	Cable Ø30	18,72	18,74	71	Int. diag.	CableØ30	18,72	18,79
86	Loop	Cable Ø30	102,7	102,73	86	Loop	CableØ30	102,70	199,90

Fig.22: Step 7. Limit the displacements (3D)

Fig.23: Step 8. With self-weight and snow (3D)

2.8 Step 8: add the self weight and the snow load (3D)

If the self-weight and the aforementioned 0,5 kN/m² snow load are included, the inner ridges go slack (figure 23), meaning that pre-stress has to be re-arranged.

2.9 Step 9: refine pre-stress and re-size

Pre-stress is increased in ridges, diagonals and loops to prevent from slacking under the snow maintaining the shape with limited deformations. Sections are optimized taking into account instabilities due to buckling and checking that the permissible stresses of the materials used are not exceeded (figure 24).

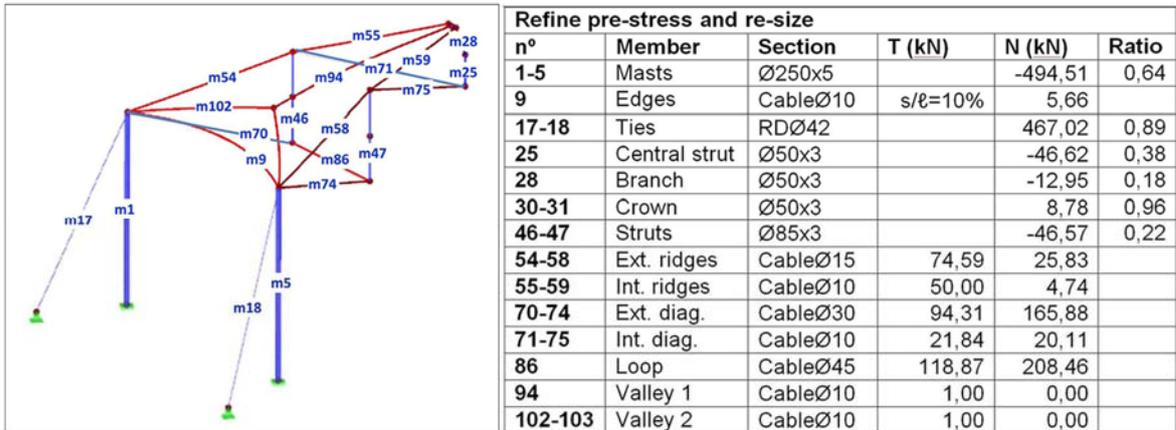


Fig.24: Step 9. Refine pre-stress and re-size

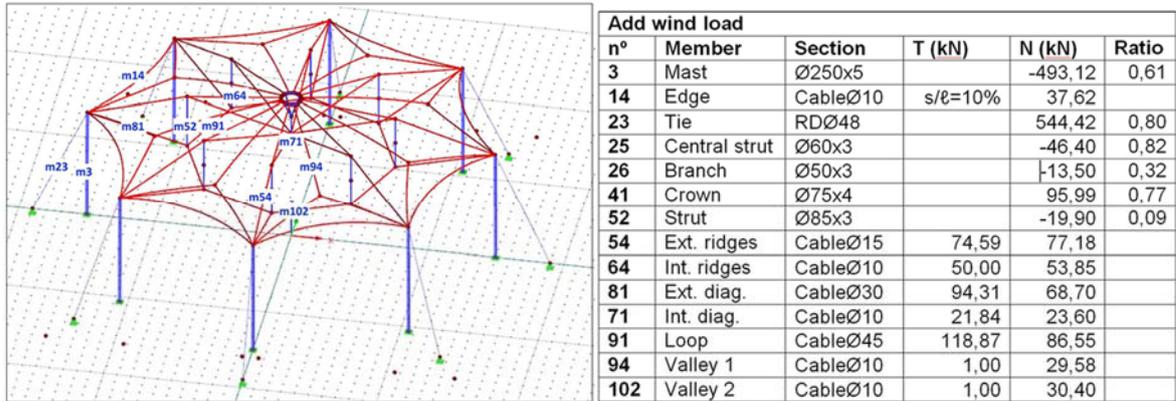


Fig.25: Step 10. With wind load

2.10 Step 10: add wind load

Finally, asymmetric loads could also be checked. In this case, the wind action corresponding to a wind speed of 26 m/s in zone IV has been considered (figure 25). It is unfavourable for some edges, ties, crown, ridges and valleys, especially on the windward side. The struts diagonals and loop are relieved. Maximum displacement is 332,2 mm = span/90 (figure 26) and the total weight (structure + membrane) 4.622,80 kg = 6,54 kg/m².

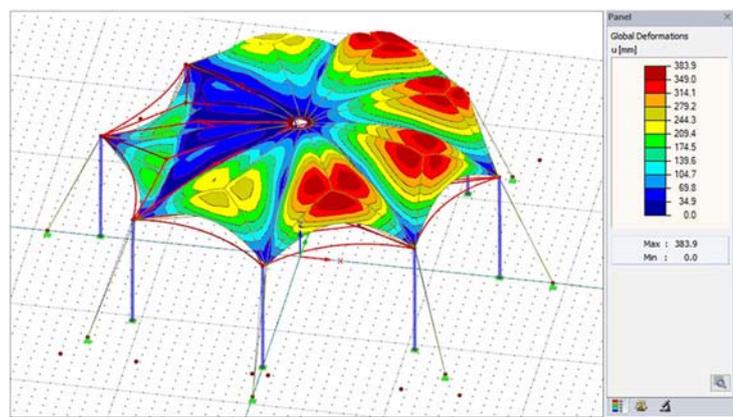


Figure 26: Step 10. Displacements under wind load

2.11 Recap

Summary of the whole process:

- Step 1: equivalent 2D model (figure 13)
- Step 2: introducing pre-stress (figure 15)
- Step 3: limit the displacements 2D (figure 16)
- Step 4: the complete 2D pre-stressed truss with masts and ties (figure 17)
- Step 5: add self-weight and snow 2D (figure 18)
- Step 6: back to the 3D model (figure 19)
- Step 7: limit the displacements 3D (figure 22)
- Step 8: add self-weight and snow 3D (figure 23)
- Step 9: refine pre-stress and re-size (figure 24)
- Step 10: add wind load (figures 25, 26)

Looking at the whole process of the procedure, it is noted that:

- a) Pre-stress of ridges and interior diagonals is practically configured from the third step onwards (figure 27).
- b) Outer diagonals and loop require progressively increasing the pre-stress (figure 27).
- c) Masts and cable-stays are most and progressively affected by the pre-stress (figure 28).

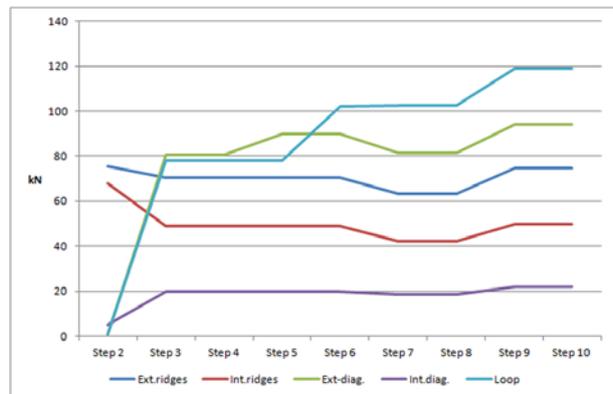


Figure 27: Evolution of the pre-tension of members

- d) Self-weight + snow load demand pre-stress to prevent slackening and limit deformations (figure 29, steps 5 and 8).
- e) Wind action is not unfavourable (figure 29, step 10).

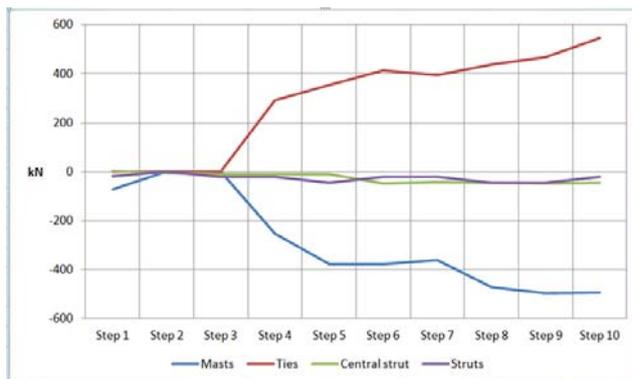


Figure 28: Interior forces of masts, ties and struts loop

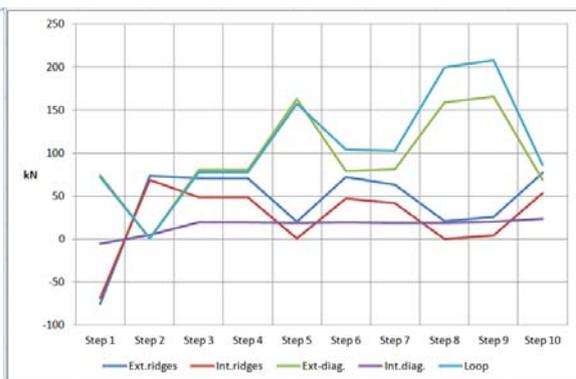


Figure 29: Interior forces of ridges, diagonals and loop

3 INSTALLATION

The construction of a cable dome is rather challenging because pre-stressing has to be introduced avoiding imbalances which would lead to instability. A general description of the installation process can be found in D.Rastorfer 1988 [17] and S.S.El-lishani, 2004 [16]. The basic steps referred to the case presented above can be summarised as follows:

- 3.1 Construction of the outer edge, be it a compression ring, guyed or braced masts.
- 3.2 Laying in place on the ground and assembling the central tension ring and ridge cables.
- 3.3 Tightening the ridge cables at the outer compression ring.
- 3.4 Hanging the struts from the ridge-cables.
- 3.5 Hanging the loop from the bottom of the struts.
- 3.6 Hanging the external diagonals.
- 3.7 Balanced step by step pre-tension in perfect unison of the external diagonals. The struts have to reach their height. The loop raises to its final position
- 3.8 Hanging the internal diagonals.
- 3.8 Balanced step by step pre-tension of the internal diagonals. The central mast and branches have to reach their height.



Figure 30: Gymnastics Hall, Seoul



Figure 31: Ciudad de La Plata Stadium



Figures 32 and 33: Cable dome of the SMS Campus Courtyard, Mönchengladbach.

The works carried out to date illustrate how to deal with the problem of instability during assembly. Temporary auxiliary erection towers were used in Seoul (figure 30, [17]), Ciudad de La Plata (figure 31, [12]) and in the cable dome of the SMS Campus courtyard, Mönchengladbach 2023 (figure 32, [13]) together with the rigid tension ring (figure 33 [14]) . For smaller dimensions, a platform lift may be sufficient. It was the case of the Bifid Tension Dome in Barcelona (figure 34, [11]). In addition, the operation was facilitated by stiffening the central row of masts together with the traction ring, as if they were the structure of a round table (figure 35). It could also be used the "big lift" method of the spoke wheels as established by M.Majowiecki in 1990 [18] (figure 36). The entire cable structure is assembled at ground level on wooden protection boards oriented in plan according to the final geometry. Then the ridges are lifted simultaneously. They start detaching from the ground and end up connecting to the outer compression ring. This method has been used repeatedly since then as in the Adana Stadium described by D.Lombardini et al. 2019 (figure 37), [19]).



Figures 34 and 35 (top): Bifid cable dome, Barcelona. Big lifts: Roma Olympic Stadium (figure 36 bottom left) and Adana Stadium (Figures 37, bottom middle and right).

4 CONCLUSIONS

- a 10 step procedure has been presented for the stabilisation of the cable-dome during the design process.
- key elements for introducing the pre-stress are the outer diagonals and loop
- most affected members are masts and cable-stays

- the critical load combination is self-weight + pre-stress + snow load
- several procedures have been mentioned to maintain stability during the installation process
- the impact of the compression ring deserves further investigation

Acknowledgements

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<https://www.dlupal.com/en/products/stand-alone-structural-analysis-software/rwind-simulation>

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