History and documentation
Construction and structural behavior of Vladimir Suchov’s Nigres tower

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ABSTRACT: From 1927–1929, the renown Russian engineer Vladimir Grigorevic Suchov (1853–1939) designed and built several towers for the Nigres power transmission line. As one of the last and most refined hyperboloid steel lattice towers, they represent the culmination of his lifelong quest in the development and optimization of this structural system.

1 INTRODUCTION

Suchov designed and built the first hyperboloid steel lattice tower for the All-Russian exhibition in Nizhniy Novgorod in the year 1896. The 25.6 m tall structure with a bottom diameter of 11 m and a top diameter of 4.3 m consisted of 80 straight steel angles and intermediate horizontal ring elements. The extremely light and filigree structure served as a water tower combined with a viewing platform and attracted the attention of the engineering community and public alike.

In the following years, his invention gained widespread circulation throughout Russia due to its cost-effectiveness and structural stability in comparison to other tower structures. Suchov standardized the building type and constructed more than hundred towers with varying heights and proportions for different purposes. The tallest structure of this kind is the 150 m tall Sabolovka radio tower in Moscow, built between 1920 and 1922.

At the end of the same decade, from 1927–1929, Suchov built six power transmission line towers at the Oka river, about 100 km southwest of Nizhny Novgorod (fig. 1). The four smaller ones were composed of three, the two taller ones of five hyperboloids, reaching 130.2 m in height. With their structural clarity, extreme lightness, and simple detailing, these towers mark the high point of Suchov’s advancement of hyperbolic lattice structures.

Five of the six towers have been dismantled since they became out of use more than ten years ago. Only one of the two 130.2 m tall structures is still existent today. This remaining structure was recently severely damaged when 16 of the 40 elements in the lower part of the first drum were cut out by looters. The missing elements will be substituted in the fall of 2007.

2 THE NIGRES TOWERS

2.1 Construction and geometry

The structure consists of five hyperboloids sections with decreasing diameters (34.0 m, 25.8 m, 19.4 m, 14.0 m, 10.0 m, 6.0 m) stacked upon each other. The sections are 24.9 m tall, except for the highest one, which measures 24.3 m. The three transmission lines were supported at 128.0 m. With the trussed outrigger at the top, the structure reaches an overall height of 130.2 m.
The first three drums are made of 40 members (steel angles), the fourth and fifth one of 20 members. Thus, the first, second and third hyperboloids are divided in plan in $9^\circ$ segments, whereas the two upper ones are divided in $18^\circ$. In plan view, the angle $\phi$ between the lower and upper end of each vertical expresses the twisting of the hyperboloid. The angle $\phi$ is constant for the three lowest sections ($36^\circ$) and becomes more pronounced towards the top of the structure ($54^\circ$ and $72^\circ$). Due to the rather subtle twisting of the verticals only the two upper sections display the typical “waisted” hyperboloid form. The two sets of inversely running verticals intersect each other 4 times in the lowest three sections and the highest one, and 3 times in the fourth sections. (See fig. 2).

Between the hyperboloid segments, horizontal lattice girders act as stiffening elements (See fig. 3) The two sets of inversely arranged verticals are slightly offset and joined with two rivets at each “intersection”. The 10 mm distance between the faces of the steel angles is hereby bridged with a shim plate.

Each drum is stiffened by ten horizontal ring elements, spaced 2.26 m on centre vertically. The connections between the verticals and the horizontal ring elements do not coincide to allow for more simple connection details. (fig. 4)

The connection between the brackets, which are cantilevering of the verticals, and ring elements is bolted. All other connections of the structure (e.g.
Figure 4. Steel structure of the first hyperboloid section, showing the slightly offset verticals and the horizontal ring elements. The vertical to the right has been cut by looters.

Table 1. Steel sizes of the structure according to the original construction documents.

<table>
<thead>
<tr>
<th>Drum</th>
<th>Verticals</th>
<th>Ring elements</th>
<th>Lattice girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>L120 × 120</td>
<td>L80 × 80 × 10</td>
<td>L100 × 100 × 12</td>
</tr>
<tr>
<td></td>
<td>12 (40)</td>
<td></td>
<td>− 240 mm</td>
</tr>
<tr>
<td>Second</td>
<td>L100 × 100</td>
<td>L75 × 75 × 8</td>
<td>L90 × 90 × 9</td>
</tr>
<tr>
<td></td>
<td>12 (40)</td>
<td></td>
<td>− 200 mm</td>
</tr>
<tr>
<td>Third</td>
<td>L100 × 100</td>
<td>L75 × 75 × 8</td>
<td>L75 × 75 × 8</td>
</tr>
<tr>
<td></td>
<td>10 (40)</td>
<td></td>
<td>− 200 mm</td>
</tr>
<tr>
<td>Fourth</td>
<td>L100 × 100</td>
<td>L60 × 60 × 6</td>
<td>L75 × 75 × 8</td>
</tr>
<tr>
<td></td>
<td>12 (20)</td>
<td></td>
<td>− 200 mm</td>
</tr>
<tr>
<td>Fifth</td>
<td>L100 × 100</td>
<td>L50 × 50 × 6</td>
<td>L75 × 75 × 8</td>
</tr>
<tr>
<td></td>
<td>10 (20)</td>
<td></td>
<td>− 10 mm</td>
</tr>
</tbody>
</table>

2.2 **History of the assembly**

The construction of the two five-tiered towers started in spring 1928. The first drum was constructed with help of an inner wooden scaffolding, which supported the structure at certain locations. By mid March, the erection of the second drum started. Unlike in the case of the neighbouring three-tiered towers, which were built conventionally with an assembly platform on top of each hyperboloid, a new construction method was used for the subsequent upper drums. Here, the telescope method was employed, which had already proven efficient in the erection of the Sabolovka tower a few years earlier. (See fig. 5)

The subsequent sections were assembled inside the shaft. Their feet were pulled towards the center with a wooden corset. Five wooden derricks located at the upper lattice girder of the last drum then lifted the next section to the top. Once the section had reached its intended height, the mountings were slowly released. Then the feet were bent outwards to the ring and were connected at their final position.

The construction of one drum element needed about six to eight weeks. The towers were completed in 1930. The employed telescope method explains why Suchov designed the lower sections without a real “necking”, even though it would have been structurally advantageous.

3 **SUCHOV’S STRUCTURAL ANALYSIS**

Suchov’s original calculations for the tower resurfaced in the town archive of Nishni Novgorod in 2007. The 10 page typewritten structural analysis comprises load
assumptions, steel member design as well as the design of the riveted connections and the foundations.

Referring to load combinations given by the Russian building code, Suchov states that the highest forces in the tower are caused if “the transmission lines aren’t ruptured, no ice build-up exists, and the wind pressure is 250 kg/m²”.

The design wind pressure of 250 kg/m² – constant over the height of the structure – is used to calculate the resulting forces on the different hyperboloid sections and at the base. This design pressure is applied on all the vertical members, using their unprojected length times the width of the steel angles as the reference area.

To account for wind shielding at the sides of the ring elements, Suchov uses the following formula with the diameter D and the width of the steel angle b to calculate the resulting horizontal wind force on one ring.

$$ F = 250 \text{ kg/m}^2 \times 4/3 \times D \times b $$

The resulting overturning moment of the tower is computed as 7384.21 tm; the self weight of the tower including the weight of the power lines is 144.26 t.

The forces on the verticals are calculated for each section with the z following equation:

$$ F = M / (n \times r) $$

with the overturning moment M, 2n the number of verticals, and the radius of the considered ring r.

Hence, the highest forces in the first section due to dead load and wind are:

$$ F = 144.26t / 40 + 7384.21tm / (20 \times 17m) = 25.3t $$

This load is used to design the verticals in the first hyperboloid. As buckling length, the distance between the support and the connection between the verticals and the first horizontal ring is used. Suchov argues that the connections provide enough torsional restraint, so that column buckling of the L-shaped vertical will not occur around the principal but the strong axis. The allowable maximum compressive stress is computed with the following formula:

$$ f_{c, max} = 0.4 \times (3100 - 11.4 \times \lambda) $$

with slenderness ratio $$ \lambda = l_i/i \). In this manner, the verticals of the first section are designed.

(E.g. L120 × 120 × 12 with $$ i_y = 3.65 \text{ cm} $$, $$ A = 27.54 \text{ cm}^2 $$

$$ l = l_i = 244\text{cm} \Rightarrow \lambda = 67 $$

$$ \Rightarrow f_{c, max} = 0.4 \times (3100 - 11.4 \times 67) = 934 \text{ kg/cm}^2 $$

$$ \Rightarrow f_c = 25300\text{kg/} 27.54\text{cm} = 920 \text{ kg/cm}^2 < f_{c, max} $$

This procedure is repeated with the respective overturning moments to design the L-shaped verticals of the higher hyperboloid sections.

Finally, the riveted connections of the first vertical of each section are designed. The combined shear and bearing pressure appears to be at the maximum 800 kg/cm² for 7/8 inch rivets. 7/8, 3/4 and 5/8 inch rivets are used for the connections according to the calculations.

In addition, the calculations entail the design of anchor bolts, the steel members at the top, the stability against overturning of the structure as well as the design of the concrete foundation. There is no structural design of ring elements or lattice girders included in the original calculations. (Suchov 1927)

4 NEW ANALYSIS

4.1 Structural model

The structural analysis model is based on the sizes and dimensions given in Suchov’s original construction drawings. For the calculations, the FEM program RStab was used. The structure was modelled in the following way:

- In order to minimize the number of nodes, the slight offset between the two sets of inversely running verticals is only included in the first hyperboloid section. This is due to the fact that the first section is more susceptible to buckling (Compare fig. 6). In the higher sections, the joints of all vertical hyperboloid shell members are in one plane, thus neglecting the slight offset.
- Verticals are joined at each intersection of the hyperboloid shell with fixed connections.
- All the horizontal ring elements and lattice girders are joined with moment-released connections to the verticals.

Material testing showed that the historic steel has similar mechanical properties to S 235 ($f_y = 235 \text{ N/mm}^2$, $E = 210000 \text{ N/mm}^2$), which was used in the structural model.

4.2 Wind loads

Wind loads were based on the DIN 1055-4. The code calls for the wind velocity measured 10 m above ground with an exceeding probability of 1/50. Only the 5 year wind velocity was available for the locality of the tower. Based on various reference material (Schueller 1981), the wind velocity was extrapolated rather conservatively with the factor 1.39. The resulting reference velocity pressure for 50 years was thus determined as 0.32 KN/m².

$$ F_w = c_f \times q(Z_e) \times A_{ref} $$
Table 2. Resulting wind pressures \( c_f^* q(z_e) \) on the steel members according to DIN 1055-4.

<table>
<thead>
<tr>
<th>Drum</th>
<th>( h_0 ) [m]</th>
<th>( h_e ) [m]</th>
<th>( z_e ) [m]</th>
<th>( q(z_e) ) [KN/m(^2)]</th>
<th>( q(z_e)^* ) [KN/m(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>0</td>
<td>24.9</td>
<td>15</td>
<td>0.632</td>
<td>1.16</td>
</tr>
<tr>
<td>Second</td>
<td>24.9</td>
<td>49.8</td>
<td>40</td>
<td>0.909</td>
<td>1.67</td>
</tr>
<tr>
<td>Third</td>
<td>49.8</td>
<td>74.7</td>
<td>65</td>
<td>1.053</td>
<td>1.94</td>
</tr>
<tr>
<td>Fourth</td>
<td>74.7</td>
<td>99.6</td>
<td>90</td>
<td>1.139</td>
<td>2.10</td>
</tr>
<tr>
<td>Fifth</td>
<td>99.6</td>
<td>123.9</td>
<td>114</td>
<td>1.205</td>
<td>2.22</td>
</tr>
<tr>
<td>Outrigger</td>
<td>124.0</td>
<td>130.2</td>
<td>127</td>
<td>1.237</td>
<td>2.28</td>
</tr>
</tbody>
</table>

Taking reduction ratios for angular sections into account, \( c_f \) is ascertained as 1.84. The velocity pressure \( z \) meters above grade level is in midland areas:

\[
q(z_e) = 1.7 * q_{ref} * (z/10)^{0.37} \quad \text{for } h = 7 - 50\text{m}
\]

\[
q(z_e) = 2.1 * q_{ref} * (z/10)^{0.24} \quad \text{for } h = 50 - 300\text{m}
\]

Wind pressures were applied to all the members, taking their flange width times length as the reference area. Unlike in Suchov’s original calculations, where the wind pressure on the horizontal rings is reduced, no reduction factor was applied to account for shielding of the members. Even though there certainly will occur some shielding on the sides of the ring elements, no code regulation could be found that was applicable to this specific geometry.

4.3 Stability

To assess the ultimate bearing load of the tower, it was critical to study its stability behaviour. Therefore, a geometrical non-linear stability analysis had to be performed. For that reason, it was essential to find suitable imperfection shapes.

Following recommendations by Graf, the imperfection shape of grid shell structures should be determined by scaling the first eigen-value mode shape under ultimate load. The first oscillation period was determined in the modal analysis as 1.02 seconds.

In the case of a tower-like structure, the eigen-value mode shapes are obviously extremely from the buckling shape of the perfect geometry. The structure of the first hyperboloid section proved to be the most critical. The buckling shape shows six evenly around the perimeter line distributed bulges, which are confined to the lower half of the section. This buckling shape of the perfect geometry was scaled and imposed on the structure as the imperfection shape. The maximum deformation was selected rather conservatively as 150 mm, a deformation considered to be visible to the naked eye.

According to DIN 18800-2, the stiffness of structures susceptible to buckling has to be reduced by dividing the E-modulus of elasticity by the coefficient \( \gamma_M = 1.1 \). The load increase factor of the imperfect geometry was determined as 4.39, compared to 4.87 of the perfect geometry. This is a rather modest decrease compared to other grid shell geometries.

Hyperboloids and some other anticlastic formed shells are typically less sensitive to imperfections, due
Table 3. Comparison of forces in the first hyperboloid section under different loadings.

<table>
<thead>
<tr>
<th>Load case/combination</th>
<th>Maximum normal forces in verticals of the lowest drum</th>
<th>Maximum normal forces in lower lattice girder</th>
<th>Maximum bending moment in lowest lattice girder, around strong axis</th>
</tr>
</thead>
<tbody>
<tr>
<td>LC 1</td>
<td>−30 KN</td>
<td>20 KN</td>
<td>−</td>
</tr>
<tr>
<td>LC 2</td>
<td>−42 KN</td>
<td>29 KN</td>
<td>−</td>
</tr>
<tr>
<td>LC 3</td>
<td>−393 KN</td>
<td>−245 KN</td>
<td>11 KNm</td>
</tr>
<tr>
<td></td>
<td>+309 KN</td>
<td>+303 KN</td>
<td></td>
</tr>
<tr>
<td>LC 4</td>
<td>−132 KN</td>
<td>−39 KN</td>
<td>3 KNm</td>
</tr>
<tr>
<td></td>
<td>+48 KN</td>
<td>+99 KN</td>
<td></td>
</tr>
<tr>
<td>LC 5</td>
<td>−281 KN</td>
<td>−169 KN</td>
<td>7 KNm</td>
</tr>
<tr>
<td></td>
<td>+219 KN</td>
<td>+212 KN</td>
<td></td>
</tr>
<tr>
<td>LC 6</td>
<td>−246 KN</td>
<td>−131 KN</td>
<td>8 KNm</td>
</tr>
<tr>
<td></td>
<td>+167 KN</td>
<td>+184 KN</td>
<td></td>
</tr>
</tbody>
</table>

4.4 Load cases

The subsequent load cases/combinations were examined, in which the wind was applied perpendicular to the direction of the transmission lines, in the following called x-direction (fig. 7). For the analyses, the load factors 1.35 for dead load, and 1.5 for live loads were applied.

The self weight of the tower was determined as 1090 KN. The resultant wind forces in x-direction for the tower structure itself are $1.5 \times 1002 \text{ KN} = 1503 \text{ KN}$ for the 50 year wind, compared to 1239 KN as calculated by Suchov. Although the base shears are quite similar, the overturning moments are significantly higher if the modern code is applied, due to the vertical distribution of wind pressures. The forces in the members of the first hyperboloid section under different load cases are summarized in table 3.

LC 1: self weight of the tower
LC 2: self weight of the tower including the transmission line, factored
LC 3: like LC 2 plus 50-year wind load in x-direction, factored
LC 4: like LC 2 plus 24% of the 50-year wind in x-direction, factored
LC 5: Self weight including transmission line plus wind load in x-direction according to the original calculations of Suchov.
LC 6: Current condition without transmission lines. Self weight of the tower plus 5-year wind in x-direction, factored.

4.5 Structural behaviour

4.5.1 Under self weight

Under self weight, all verticals are subjected evenly to compression forces, whereas the lattice girders are...
subjected to tension forces. This is caused by the differently inclined verticals above and below each lattice girder. The outward thrust is thus balanced by the lattice girder.

The ring elements are not subject to any forces. They simply act as bracing elements for the verticals.

4.5.2 Under self weight and horizontal wind load in x-direction

The tower displays a remarkably high bending stiffness. The horizontal deformation of the structure under unfactored wind load is 340 mm, resulting in a deflection ratio of 1/388.

Under self weight and wind load, the structure displays a tube-like load bearing behaviour. All verticals are subjected to normal forces. Predictably, the verticals at the front face are under tension and the ones at the rear face under compression. At the "sides", verticals are alternating under tension or compression, depending on their twisting angle. This imposes a vertical distortion of the lattice girder at the end of each section.

The discontinuity of the verticals between the different hyperboloid sections results in a decrease of the shear stiffness, causing large deformations in these areas. The lattice girders are subject to linearly increasing compression forces on the front face and tension forces on the opposite side, thus balancing the normal forces of the inclined verticals below.

The bending action of the tower causes the lattice girder to ovalize, thereby inducing bending moments around the strong axis of the member. (fig. 8) In addition, the girders are even more affected by the push-pull action of the sidewise verticals, acting around their weak axis. The ring elements adjacent to the lattice girders get affected by the distortion in this area as well, causing overstress due to bending.

Based on the current analysis and load assumptions, the structure would not be adequate to sustain the 50-year wind, due to buckling of the verticals in the first section. The structure including transmission lines would only satisfy code requirements if the factored wind loads are reduced by 76%. Even the current condition without the transmission lines would not be sufficient to sustain the 5-year wind.

Despite of some minor local overstressing, almost all other members of the structure are suitable (load case 3).

5 CONCLUSION

The buckling behaviour and wind load assumptions of the structure will have to be investigated more deeply, as the results are not in line with the 80 year life span of the tower. One critical point is the lack of the 50-year wind velocity at the locality of the tower. Shielding of members in an open circular framework is another issue that requires further research. The disregard of any reduction factor for shielding (due to the lack of suitable specifications) certainly effectuates a too conservative assessment of the structural safety of the tower.

Despite of these unanswered questions, it is intriguing to comprehend the structural behaviour of these multi-storeyed hyperboloid lattice towers. The timeless elegance, ingenuity and courage that the NIGRES tower exhibits still have a humbling effect on the beholder.

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