Numerical analysis of the steel roofing structure of the Umberto I Gallery in Naples

R. Landolfo, M. Manganiello & F. Portioli
Dipartimento di Costruzioni e Metodi Matematici in Architettura, Università degli Studi di Napoli "Federico II", NAPOLI, ITALY

ABSTRACT: In this paper, the stress level in the steel roofing structure of the Umberto I Gallery in Naples is analysed by using a three dimensional numerical model implemented in a non-linear finite element code. The geometrical and mechanical properties of the structure have been obtained through a wide investigation on available historical documentation. The considered design actions, namely self-load, wind and snow, have been evaluated and combined on the basis of the Eurocode 1 provisions. The numerical results have allowed the actual safety factor respect to the ultimate limit state to be evaluated. In addition, the role played by each member in the overall structural behaviour of the roofing system has been clarified. Finally, some considerations on the structural vulnerability to corrosion damage have been developed.

1 INTRODUCTION

The Italian historical construction heritage is essentially constituted by masonry structures, iron structures or by their combination. Among the most spread applications, the large metallic roofing systems, which between the nineteenth and the twentieth century were frequently used to cover urban galleries, represent an interesting example of the Belle-Epoque architectural style in Italy.

The large-span roofing was undoubtedly one of the first important applications of steel structures. In fact, the possibility to realise simple structural joints was rapidly exploited in complex systems where the use of other materials was very expensive.

After more than one hundred years from the first significant applications in the field of civil engineering, an evaluation of the actual safety factor of such kind of constructions is a topical matter regarding the Italian building heritage. In this perspective, it is necessary to evaluate, for example, the consequences of corrosion phenomena (Tampone, 1989; Bertolini, 1989) or the effects of the modified load conditions as respect to the ones originally considered.

Of course, this task is made easier by employing modern computational methods, which provide simple numerical procedures able to predict local and global structural performance of ancient constructions.

However, in these cases one of the main difficulties is related to the evaluation of admissible stress level. The current code provisions, in fact, prescribe the mechanical properties for new materials, but they do not define the safety factors to be employed for ancient ones. Moreover, no indications are provided for estimating the reliability of the old constructive techniques, often very different from the ones used nowadays.

Nevertheless, the designers should not strictly use modern codes for evaluating the actual safety factor of the historical buildings, because they could penalize these structures whose strength has been tested through several decades. Therefore, the stress rate for each element has to be evaluated by adopting safety criteria properly modified as respect to the current ones.

Moreover, it is noteworthy to mention that the ancient metallic structures were generally designed on the basis of the rules related to the wood ones. As a consequence, some important structural aspects, such as the ones related to the global behaviour of the whole structural analysis of historical constructions.
system, were not appropriately taken into account. Structural failings (Berardi, 1998) can be found in almost all the iron structures; the most important one being surely related to the absence of adequate bracing systems. The horizontal actions, in fact, were often supported by masonry filling walls or by the flexural strength of cast-iron frame structures whose joints were sometimes inadequate.

On the basis of the previous considerations, it can be deduced that the analysis of large-span roofing iron structures shows specific problems, which are different from the ones concerning modern steel constructions.

2 FRAMING OF THE STUDY

The current study is framed within a specific research program devoted to provide suitable analysis methodologies for the assessment of the structural response of nineteenth-century metallic constructions. The attention is focused on the roofing systems that, as above-mentioned, represent a very interesting structural application.

The whole program is subdivided in the following four phases: a) historical research, basically devoted to characterise the physical and mechanical properties of the materials; b) choice of a case study representative of the ancient structures of the considered historical period; c) analytical investigation carried out by means of numerical model, implemented in a finite element code; d) identification of the main structural problems for the selected case and discussion about possible retrofitting approaches.

According to point b), the iron roofing structure of the Gallery “Umberto I” in Naples has been chosen as case study, since it represents one of the most important nineteenth iron structures present in Southern Italy.

Then, a suitable finite element model of the corresponding structural system has been set up in order to carry out the numerical investigation necessary to provide a first assessment of the internal stress distribution. The study is in progress.

The main results concerning the first three phases are summarised in this paper. In addition, some preliminary considerations related to the vulnerability of the structure to the corrosion phenomena are provided.

3 THE UMBERTO I GALLERY IN NAPLES

The “Umberto I” Gallery was built in 1887 in the centre of Naples (Jodice, 1985, Carughi, 1996), amongst other famous historical buildings and churches. It was designed in pure Belle-Époque style analogously to the one built in Milan 20 years earlier.

The Umberto I Gallery is the second and the busiest gallery in Naples. It was built during the urban renewal following a cholera epidemic and according to a special law of 1885. The design was by engineer Emanuele Rocco, and then modified by Ernesto Di Mauro and Antonio Curri. Paolo Boubée designed the metallic structures of the roofing system.

The architecture joins new Renaissance facade with a beautiful glass and iron roofing. Designed on the basis of known structural schemes, as the one used for the Gallery “Vittorio Emanuele II” in Milan (Gianni, 1989), this typology presents a cruciform plant with four orthogonal arms intersecting in an octagonal zone.

Different parts compose the main iron structures: simple lattice arches constitute the framework of both the four arms and the spherical dome (Fig. 2).

The central dome covering the central zone is 56.00 m high and it is one of the tallest nineteenth-century roofing structures made of steel and glass in Europe.

The retrofitting and/or upgrading of this construction are a current topic. In fact, recent inspections have highlighted spread corrosion phenomena near joints, which could be significantly reduce the actual safety factor of the structure.

4 THE NUMERICAL MODEL

4.1 General remarks

A numerical model of the roofing structure of the “Umberto I” Gallery has been set up on the basis of available data.

The metallic framework is composed of different parts (Fig. 3) that can be analysed separately. In particular, as before mentioned, it is possible to distinguish four cylindrical vaults converging on a spherical dome.

In this study, supposing that the interaction between such parts is negligible, the stress rate of the central dome is investigated.
The glass covering is applied on the external side of the dome. It is supported by a secondary framework that will not take into account in the following because its structural role is negligible.

4.2 The material

Since at the moment the evaluation of material mechanical properties can not be directly done by means of specific tensile tests on specimen taken out from the structural system, a wide historical investigation has been carried out in order to define the stress–strain material parameters (de la Grennereis & Landolfo, 2003).

In particular, elastic \((E, f_y)\) and ultimate \((\varepsilon_u, f_u)\) parameters have been identified.

Although the obtained values are very scattered and extremely dependent on the examined source (Boubée 1880; Breymann, 1925), the use of the average values provided by codes coeval with the construction period seems to be the most reasonable approach.

These values are shown in Table I and they are referred to the mechanical properties of metallic materials used at the end of nineteenth century.

It is interesting to note as the steel had mechanical properties significantly better than the rolled iron ones. Nonetheless, its widespread happened at the beginning of the twentieth century, therefore some years later the construction of the Umberto I Gallery. For this reason, the mechanical properties of the rolled iron \((E = 180000 \text{ Nmm}^{-2} \text{ and } f_y = 145 \text{ Nmm}^{-2})\) have been assumed in the numerical investigation.

4.3 The geometry

The central dome on which the four arms converge (Fig. 3) represents the most important structural part of the Gallery.

As before mentioned, the structural interaction between the dome and the arms can be considered as negligible, since it is only due to secondary structural elements supporting the upper glass covering.

As it can be seen in (Figs 4 and 5), the dome is constituted by sixteen lattice arches (2) with external and internal diameters measuring 18.90 m and 18.20 m, respectively.

The outer and inner chords of these arches are made of rectangular solid sections with equal dimensions 0.18×0.016 m. Also the braces are composed of the same element type, whose sections have dimensions equal to 0.07×0.004 m. These arches are connected by means of secondary truss beams (3) whose shape and global dimensions change along the vertical axis; for all of them, chord and braces have the same dimensions, which are 0.120×0.006 and 0.050×0.003, respectively.

Figure 3. Plan view (Guerra, C.A. & Mazzolani, F.M., 1989).

Figure 4. Details of lattice dome (Fondazione Guerra, Naples).

<table>
<thead>
<tr>
<th>Table 1. Mechanical properties of the material [Nmm(^{-2})].</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>---------------------</td>
</tr>
<tr>
<td>Compression Tension</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>(f_y)</td>
</tr>
<tr>
<td>(f_u)</td>
</tr>
<tr>
<td>(\varepsilon_u)</td>
</tr>
<tr>
<td>(\varepsilon_u)</td>
</tr>
<tr>
<td>(\varepsilon_u)</td>
</tr>
</tbody>
</table>
The upper (1) and lower (4) rings, which constitute the structural elements supporting the lattice arches, have rectangular hollow sections. The corresponding radiiues measure 5.15 m and 18.55 m, while the cross-sectional dimensions are (width-height-thickness) 0.50-0.70-0.01 (1) and 0.70-0.80-0.01 m (4), respectively.

The lower ring (4) is supported both by eight circular arches (6) and by eight curved columns (5), the latter located between two consecutive arches. These arches, whose cross-section is hollow rectangular with dimensions equal to 0.70·0.50·0.01 m, have middle radius equal to 7.75 m.

The columns have a T-section: the flange has a section equal to 0.180·0.016 m, while the web has the height varying along the longitudinal axis (between 0.30 and 1.50 m) and thickness equal to 0.01 m.

The springing lines of the arches are alternatively linked by means of beams (7) having hollow squared section with dimensions equal to 1.0·1.0·0.01 m. The last ones are restrained to the masonry structures below.

4.4 The loads

Self-weight, snow and wind are the single load conditions considered in numerical analysis. The Eurocode 1 provisions have been considered for the evaluation of each action.

4.4.1 The wind

As regard as the wind effects, the external pressure has been evaluated by means of the following simplified formulation:

\[ w_e(x, y, z) = q_{ref} \cdot c_e(x, y, z) \cdot c_{pe} \]  

(1)

where \( q_{ref} \), \( c_e(x, y, z) \) and \( c_{pe} \) are the reference pressure, the exposition coefficient and the pressure coefficient, respectively. The other coefficients, not explicitly included in the Eq. 1 and taking into account the topographic and dynamic effects, have been assumed to be equal to 1.

The values used for \( c_{pe} \) are evaluated according to (Fig. 6), while for \( c_e(x, y, z) \) a constant value equal to 2.2 is adopted.

According to the Eurocode 1 indications, the reference pressure \( q_{ref} \) has been assumed equal to 0.456 kNm\(^{-2}\). No internal pressure has been considered because the wind effects damp down from the lateral openings to the central zone where the dome is placed.

According to the code indications, the wind pressure has been considered constant on each plane orthogonal to the wind direction and its distribution has been defined according to the wind profile shown in Figure 6.

The wind effect has been modelled as a system of concentrated forces (acting along the normal (\( \hat{n}_i \)), see Fig. 7) loading the external nodes of the lattice arches constituting the dome.

According to the outward normal (\( \hat{n}_i \)) the coefficient \( c_{pe} \) related to each area \( (c_{pei}) \) has been properly assessed. Finally, in order to define the wind load, the portion of surface related to each node of the lattice arches constituting the dome has been estimated \( (S_i) \).

Then, the resulting force on each node has been evaluated according to the Equation 2:

\[ F_i = q_{ref} \cdot c_{pei} (x, y, z) \cdot c_e \cdot S_i \cdot \hat{n}_i (\alpha_x, \alpha_y, \alpha_z) \]  

(2)
where $x, y, z$ are the coordinates of the reference node and $\alpha_x, \alpha_y, \alpha_z$ the directional cosines of the considered surface afferent to it.

4.4.2 The snow

With regard to snow load, the vertical pressure equal to $0.75 \text{kNm}^{-2}$, evaluated according to the Eurocode 1, has been properly applied (according to the Fig. 8) in order to take into account the structural shape. It acts on the horizontal projection of the spherical surface and it is applied on the nodal points previously mentioned.

As it can be seen in Figure 8, a constant intensity has been stated on the two half-circumferences. The assumed values are $\mu_3/2$ and $\mu_2/2$ for the left and the right side (respect to the middle line), respectively.

4.4.3 The load combination

The single load conditions have been added by means of the formulation (Eq. 3) provided by EC 1 and related to the ultimate limit state corresponding to persistent or transient design situations.

$$F_d = \gamma_g \cdot G_k + \gamma_q \cdot Q_k + \sum_{i=1}^n \gamma_{Q_i} \cdot \psi_{Q_i} \cdot Q_k$$

Alternatively, wind and snow loads have been fixed as leading variable actions and combined among them, in order to obtain all the possible combinations, by using the following partial safety coefficients:

$\gamma_g = 1.0 \div 1.35$, $\gamma_q = 1.5$, $\psi_{Q_i} = 0.6$.

4.5 The finite element model

The numerical model has been implemented in the non-linear finite element code SAP2000 version 8.1 (Computers and Structures, 2003). The geometry and the mesh density used for the different structural elements are shown in (Fig. 9).

Two types of finite elements have been employed: beam elements for principal (2) and secondary (3) truss systems and shell elements for structural parts having hollow sections (1, 4, 5, 6, 7). As far as the latter ones, they have been modelled with one shell for each section side; along the longitudinal direction a uniform mesh size equal to $0.5 \text{m}$ has been adopted.

With regard to the internal restraints, the hinge schematisation has been used for the main lattice structures (2), while no internal releases have been imposed for the secondary ones (3). The last hypothesis has been adopted because the release of internal moments...
for secondary truss beams (Vierendeel beams) would cause rigid body mechanism (see Fig. 9).

Two different assumptions have been done regarding the external restraint configurations: the fully hinged one and the simple supported one.

As it will be shown in the following, these two different boundary conditions have allowed to define a behavioural range within the actual performance of the examined structure is included.

5 RESULTS

In this section, the most representative results obtained for the worst load combination and restraint condition are presented. In particular, the considered wind direction is depicted in (Fig. 10) while, for the snow intensity and distribution, the previous schematisation represented in Figure 8 is adopted.

With regard to the external restraints, the results concerning the simple supported scheme, which represent (as successively shown) the most severe condition, are shown.

The two different load (wind and snow) conditions are added in order to combine the effects of positive and negative pressure in the most unfavourable way. Then, higher snow intensity is applied on upwind surfaces, while the lower is placed on the downwind one.

The following results are referred to the load combinations considering the wind action as the main accidental load and the snow as the secondary one. This choice reflects the environmental and climatic conditions related to the urban contest in which the Umberto I Gallery is located.

Since all the load conditions are symmetric respect to the $z-x$ plane, only the structures related to half-dome is represented in the following figures.

For the sake of graphic representation, the considered half-dome has been further divided in two parts ($A$ and $B$) each containing five lattice arches (Fig. 10).

In (Fig. 11), the axial forces distributions due to the self-weight within internal (Fig. 11a) and external (Fig. 11b) chords of lattice arches constituting the dome are reported.

Although this load condition is not the most severe one, it well highlights the influence of the supporting conditions on the distribution of internal forces.

In particular, since the restraint base conditions of the lattice arches supported by the columns $O'O''$ (Fig. 9) have rotational and translational stiffness higher than the ones corresponding to the point $O'$, the internal forces distributions are completely different for a couple of adjacent arches. As it can be seen from the Figure 11, the stress distributions assume alternatively the same trend (same base restraints) due to the radial symmetry of the dome subjected to the self-weight.

In (Fig. 12) the distribution of internal forces due to the most severe load combination is depicted for the wind acting along positive $x$-axis and snow loads applied according to the Figure 8.
The deformed shapes of the most loaded arches ($A_1$ and $A_2$) having different base restraint condition (see Fig. 9) are depicted in (Fig. 13).

As for the self-weight load condition, also in this case it is evident that the rotational stiffness and vertical displacement of the base restraints related to the first arches $A_1$ is lower than the ones corresponding to $A_2$. Therefore, even if the $A_1$ arches are the most loaded ones because they are parallel to the considered direction of the wind load, they are less stressed than $A_2$ (see Fig. 12).

Since the numerical model has been set up on the basis of the actual structural configuration (global lattice system), an appropriate prediction of the displacements (Fig. 14) for any load condition has been obtained. With this regard, it is worthy to mention that such structural systems have been usually schematised in the past by means of equivalent beams with compact sections. According to this schematisation, the obtained internal forces are successively distributed within the actual element section by using equilibrium conditions. Even if this approach reduces the computational efforts and it allows to the internal stress distribution to be defined, it is evident that no accurate evaluations of the displacement field can be obtained. On the contrary, the implemented finite element model shows that lattice structures, such as the examined ones, must be simulated by considering all the actual parts, in order to correctly evaluate the stiffness of each element and the actual displacements of the whole system.

In (Figs 15 and 16), the axial forces for the internal and external chords of the lattice arches constituting a quarter of the dome are depicted. More in detail, the second quarter ($B_1$-$B_5$) is proposed for the internal chords and the first quarter ($A_1$-$A_5$) for the external ones. According to the internal force distribution reported in Figure 12, it is evident that the considered elements are the most stressed ones. In particular, $B_4$ and $A_2$ are the most loaded arches.

As it has been previously mentioned, the upper and lower chords of lattice arches have been schematised as rectangular solid sections. According to the values of internal forces plotted in Figures 15 and 16, the maximum stress in these elements is about $60 \text{Nmm}^{-2}$.

As far as the stresses in the braces are concerned, the obtained values are similar to the ones previously
indicated for the chords (Fig. 17). However, the absolute values of internal forces are small since the sliding effects among the chords are negligible. This is due to the reduced relative slope of the pressure curve respect to the arch axis. The internal stress distribution in the shell elements used to schematise the top and bottom rings is represented in (Fig. 18). These members take the most part of the thrusts acting along the parallels of the dome since they are much more stiff than the secondary horizontal structural systems. Obviously, the upper ring is compressed and the lower one is tensioned.

Higher stress levels (about 70 Nmm$^{-2}$) have been obtained on the rectangular hollow arches (Fig. 18), mainly where the hollow squared beams (clamped into the masonry structures) are missing below.

Since their flexural stiffness is low, the vertical reactions are concentrated near the ends where the rectangular hollow arches converge. However, the stresses on masonry result lower than 0.20 Nmm$^{2}$ if a base square area whose side dimension is equal to the beam height (1.0 m) is assumed.

With regard to the secondary horizontal structural systems, maximum stress values of about 20 Nmm$^{-2}$ have been found (see Fig. 19 for axial loads).

Therefore, it can be concluded that such systems were adopted both to support the upper roofing structure and, probably, to reduce the buckling length of the lattice arches.

6 INTERPRETATION OF THE RESULTS

With regard to the stress distributions on the different structural elements, two important remarks may be pointed out.
von Mises stresses on lower arches and hollow-supports for different base conditions: (a) simple supported; (b) full restraint.

The first one concerns the maximum absolute value experienced by each structural element, which always ranges from 60 to 70 Nmm\(^2\). Being the assumed yielding strength about 140 ± 50 Nmm\(^2\) (see Table 1), it can be deduced for the safety factor a value equal to 2.

The implemented finite element model, whose suitability has been just discussed, can be successively used in order to verify the effects of the corrosion (reduction of thickness) and of different load conditions (i.e. the seismic action).

The second important consideration is related to the stress distributions and to the considered restraint conditions. In particular, very different stress values have been found on the circular arches supporting the bottom ring of the dome for the two considered restraint conditions (Fig. 20). As it has been previously outlined, they define numerically a behavioural range within the actual boundary conditions of the structure are placed.

In order to evaluate the safety factor of the dome, the simple support condition can be used in the analyses.

In fact, although maximum stress values are approximately the same for fully restrained and simple supported base conditions, larger critical zones may be observed for the last case on the arches whose skewbacks are not directly linked by the foundation beams.

In (Fig. 21), the stress ratio for each structural element is proposed. Even if the T-section columns supporting the lower ring are slightly more stressed, the stress level in all the main structural parts is almost the same with the exception of the secondary truss beams.

Figure 21 allows the vulnerability of each element to the corrosion phenomena, which could be not uniform on the different zones, to be defined. Since the lattice arches are very stressed and present the external chord (Fig. 2) exposed to the atmosphere humidity, they represent the most vulnerable parts of the system to degradation effects. Similar considerations can be done for the lower and upper ring.

7 CONCLUSIONS

A three-dimensional finite element model of the dome of the Umberto I Gallery in Naples has been implemented in a non-linear FE code in order to evaluate the actual safety factor of the structure.

The performed analyses have allowed both the stress level and the role played by each structural member to be clearly evaluated.

Among the meridian lattice arches constituting the structure, the most stressed for the considered load conditions are the ones supported by both lower ring and T-section columns. This difference in the arch stress distributions is due to the actual restraint conditions, which have been correctly accounted for by the adopted model. In addition, the detailed schematisation of the arches has well emphasised the relative displacements among them.

As far as the estimated safety factor is concerned, it appears adequate (max stress ratios equal to about 0.6), even if it is referred to no damaged structure. Certainly, the effects of corrosion must be taken into account in order to evaluate the true residual lifetime of the dome and eventually the need for some structural upgrading.

Such effects should be simulated through an equivalent reduced thickness for damaged elements, which mainly include the upper chord of lattice arches and the supporting hollow arches. These elements, in fact, are directly exposed to atmospheric agents and, therefore, present some corrosion damages whose entity have to be exactly estimated through specific investigations.

Being this study a first step basically devoted to set up a numerical model and to evaluate the internal stress distribution for a given load combination (self-weight, snow and wind), further analyses will be necessary in order to evaluate the effect of the other load conditions.
REFERENCES


Bouée, F. P. 1880. Trattato elementare teorico-pratico di Costruzioni Metalliche. Pellerano, Napoli (Italy).


