# A NEW STADIUM FOR THE 1990 WORLD FOOTBALL GAMES

by

M. Majowiecki
Dept. of Civil Engineering
University of Bologna, Italy

F. Ossola
Dept. of Civil Engineering
University of Torino, Italy

SUMMARY

This paper illustrates the actual state of the works of the under construction stadium of the city of Torino which must be ready for the 1990 world football games. The stadium has two stands under the ground level. The third level is designed in steel space structures. The covering system of the grandstand is made by cable trusses radially oriented and anchored on the ground. The horizontal forces are equilibrated by a tension central ring. Corresponding to the longitudinal axis two cable nets are connected to the ring and covers the North-South curves.

1. DESCRIPTION OF THE STRUCTURAL SYSTEM

The tensile-structural system which has been adopted to cover the new stadium of Torino is mainly formed by:

- a radially oriented set of cable trusses,
- an inner tension ring,
- two cable nets with hyperbolic surfaces,
- an external stayed anchorage system,
- a gravity foundation system.

1.1. Radial cable trusses

The cable trusses are formed by two load bearing upper cables, two lower stabilizing cables interconnected, in vertical planes, with hanger cables spaced every 5 m. They are placed in radial direction as to centers C1 and C2 of the polycentric homothetic curves which generate the general geometric scheme of the stadium.
The trusses corresponding to alignments 1 to 6 and symmetrical as to x and y coordinates are generated with a constant angle of 2°.54 at the center. Trusses 7-14 have a generating relative angle of 7°.22.
Depending on the stress and deformation state, the cable structures have been dimensioned differently and collected into two groups. The main geometrical and mechanical characteristics of the system are shown in Fig. 1.

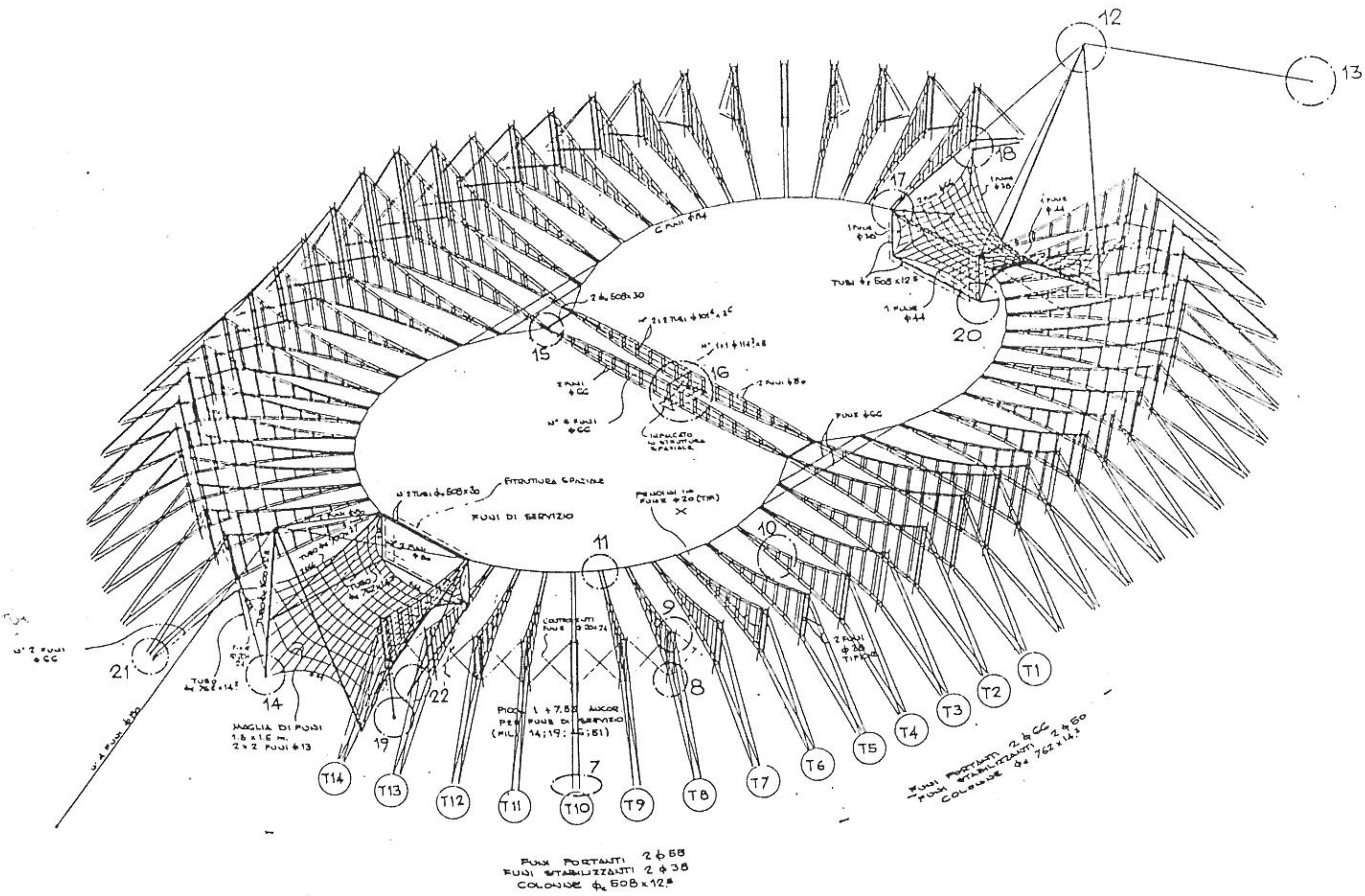


Fig. 1. Perspective view of the structural system.

All the cables are spiral zinc coated steel structural strands (class B) and have a locked and/or open section; they have an elementary resistance of the external "Z"-shaped wires greater than 1570 $N/mm^2$ and of the internal circular ones greater than 1770 $N/mm^2$.
a secondary transversal structural system is suspended at the stabilizing cable level. The typology adopted for this system is that of a simple supported (hanging) spatial truss with triangular section made of tubular steel.elements. The covering system (DECK) which is placed on the extrados of the secondary beams and connected to it, is formed by aluminium corrugated plates, with height of 200 mm and thickness of 12/10 mm, by an insulation layer and by extrados impermeabilization mantle consisting in two aluminium thin plates which are mechanically connected to the intrados plates with anchor bolts.

1.2. Internal tension ring

The internal ring whose main function is to balance the horizontal forces, transmitted by the radial oriented cable system, in a local closed system, is geometrically configured on design by four circular arches with radius 56.468 m.
In order to increase the curvature and consequently to reduce the tension forces, it has been necessary to introduce a transversal tie bar characterizing the bilobate polycentric configuration of the inner ring.
The ring is formed by 6 zinc coated, locked section spiral shaped cables with diameter of 84 mm; they are arranged on horizontal planes at a height of +41.00 m, and are vertically spaced in order to allow a functional connection to radial trusses. The cables of the internal tie bar have a 2x6 diam. 66 mm having the same shape of the previous cables.

1.3. Cable nets

Two cable nets with quadrangular mesh and with negative total curvature surfaces (saddle-shaped) are arranged in the central areas of the North-South curves.
The cable net is anchored to the central ring at +41.21 m height, to the carrying structure of the third level of the grandstands at +31.5148 and at +21.00 m respectively and to the foundation at a level of +8.215 m.
The covering of these areas is linked to the general covering in a sector of the stadium where the third level of grandstands is missing.
The covering adopted for the cable nets is a membrane system made with fiber-glass + PTFE with the 60% of translucidity. The membrane is fixed to the net on lines and/or points through mechanical connections.

1.4. Anchorage systems

The anchorage system of the cable trusses and cable net consists of external stayed frames with tubular columns and stay cables.
As to the rectilinear areas, the anchorage frames are placed on the structures of the third level grandstands at a height which is variable between +35.539 and 35.556 m. The tops of the columns are placed at

heights included between +50.868 and +60.834 m. The anchorages of the stays are set at height +21.05 m.
The steel tubular columns have diameters included between 508 and 762 mm and thicknesses between 10 and 14.2 mm. The quality of the material is Fe 510 C. The upper and lower stays have been realized with the same cables which have been used for the carrying and stabilizing cables.
Also the anchorage system of the cable nets and of the ring in the curve is formed by and anchorage frame consisting of tubular columns having a diameter of 1600 mm and a thickness of 28 mm. They are positioned according to a "V"-shape and have an inclination of 10° tooward the outside as the vertical axis. The top of the "V" is placed at the height of 73.5296 m and the base (with width of 35.8 m) is set at the height of 9.90 m. The stays realized with 4 cables having a diameter of 80 mm are anchored along the longitudinal axis of the construction at a distance of 237.5 m from the center of the green field and at the height of +15.00 m from the playground.

### 1.5. Gravity foundation system

The foundation system adopted in order to equilibrate the tension forces transmitted by the cable stay anchorages are of gravity type. All the adjustable connections provided to introduce the pre-tension forces are placed in the top of the special shaped concrete anchorage blocks.

## 2. ANALYSIS OF THE LOADS

### 2.1. Permanent loads

| | |
|---|---|
| Dead load of cables + clamps | 72 $N/m^2$ |
| Ring-truss connectors | 15 kN/each |
| Covering | 250 $N/m^2$ |
| Purlins | 220 $N/m^2$ |
| Concentrated technological loads (4x3 for each cable truss) | 12 kN |
| Extra add. perm. load | 80 $N/m^2$ |

### 2.2. Accidental loads

#### 2.2.1. Statistical analysis of snow load qs

At this regard, a statistical analysis has been carried out about the snow load to be assumed, during the planning, for a locality of Piemonte placed at an height of 300 m around above sea level (1).
For this research, it comes out that the characteristic value of the snow load at ground level which has been defined as argumental value "q" with a pre-fixed mean return period "To" for a locality placed at a height of 250 m above the sea level results (see Fig. 2)

$q_{50} = 965\ N/m^2$ (minimum intensity of the snow design load)

$q_{175} = 1220\ N/m^2$ (design intensity which has been adopted according to the characteristic requests of durability of the main structural system)

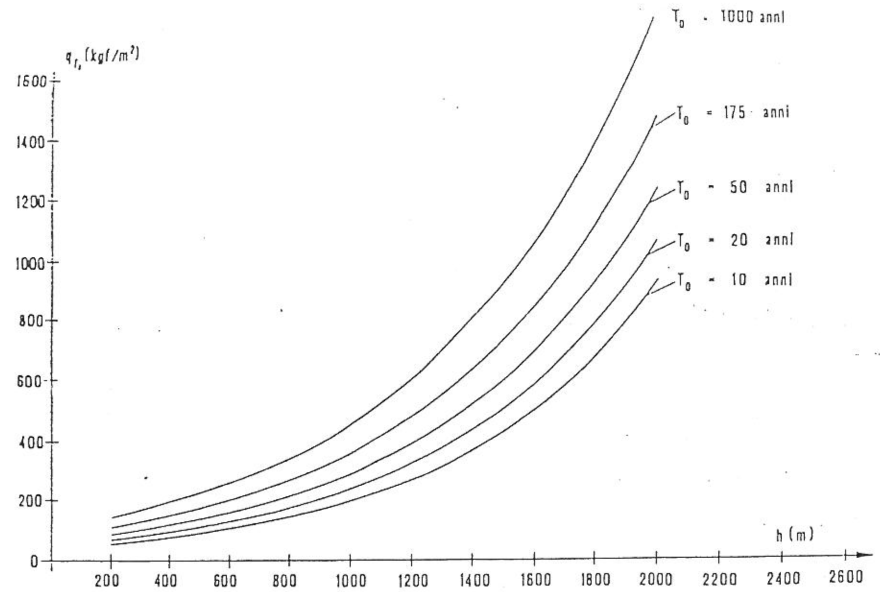


Fig. 2. Characteristic value of the snow load at ground level for a definited return period To.

The exposure coefficient which can be adopted depending on the shape of covering is $\mu = 0.8$, according to CNR-UNI 19912/85 specifications and the French "Règles no. 84". In consideration of the extension of the covering, the exposure and distribution coefficients have been defined more precisely through the experimental analysis shown here below.

### 2.2.2. Experimental analysis of the distribution of the snow load according to the direction of the wind

In consideration of the unusual dimension of the construction, an experimental analysis has been carried out to understand the distribution of the snow according to a pre-fixed intensity and duration of the wind and according to three different directions (azimut 0°; 45°; 90°).

The experimental studies have been carried out with BEAN (Banc d'Essais d'Accumulation de la Neige due au vent) by the CEBTP at Saint Remy les Chevreuse.

The experiment consists in putting a constant layer having a thickness of 5 mm and formed by wooden particles of 0.2 mm on the covering. Afterwards, a turbulent flow is generated having such a speed that the wooden particles are removed.

This experimental speed has been defined as 3.10 m/sec, measured at 115 mm on the bench and it corresponds to a real level of 34.50 m. The exposure duration is of $t_m = 30$ min.

On the basis of the dimensional analysis, the similarity relations have been obtained; they allow the convertion from the model to the real and supply the following data:

- speed of the wind: 9.7 m/sec,
- duration of the wind: 2 days.

As far as regards the static and dynamic analyses, the distribution shown in Figs 3a and 3b are adopted for directions of wind incidence with $\alpha = 0°$ and $\alpha = 90°$.

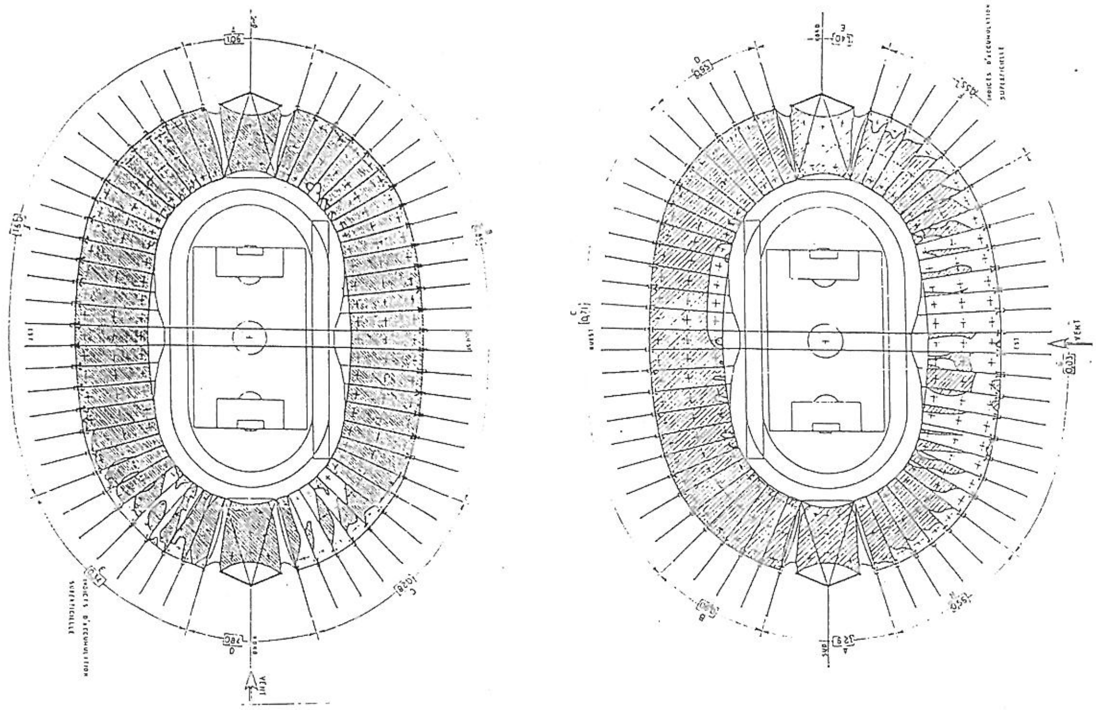

Fig. 3. (a) Distribution of the snow with wind acting from 0°.
(b) Distribution of the snow with direct wind acting from 90°.

### 2.2.3. Statistical analysis of the reference speed of the wind $V_{ref}$

The importance of the work and the characteristics of the covering structural system lead to the need of an accurate examination of the effects caused by the wind.

In order to perform this examination, a reliable definition of a "local model of the wind" is required and it includes:

- the evaluation of the structure of the undisturbed wind,
- the determination of the distribution of the wind pressure on the construction.

The structure of the wind is univocally defined by the profile of the mean speed and by the cross power spectral density of atmospheric turbulence.

The mean speed of the wind is then defined by the speed intensity and by the shape of the profile.

The criteria for the determination of the structure of the wind are made known in a very simple way in the most recent national specifications: the instructions of the CNR 10012/85 "Actions on the constructions". Reference to these specifications has been made as far as regards the numerical values of some parameters which define the model of the wind.

The mean speed of the wind Vz (m/sec) is expressed by the relation (CNR-UNI 100012/85 on the "Actions on the buildings"):

$$V_z = V_{ref} \; \alpha_r \; \alpha_t \; \alpha_z$$

where as to Vref (value of the mean speed of the wind in 10 min interval (measured at 10 m from the ground), the specification assumes a value of 30 m/sec for each direction, which seems too conservative for the locality where the construction will be realized. At this regard, the necesary

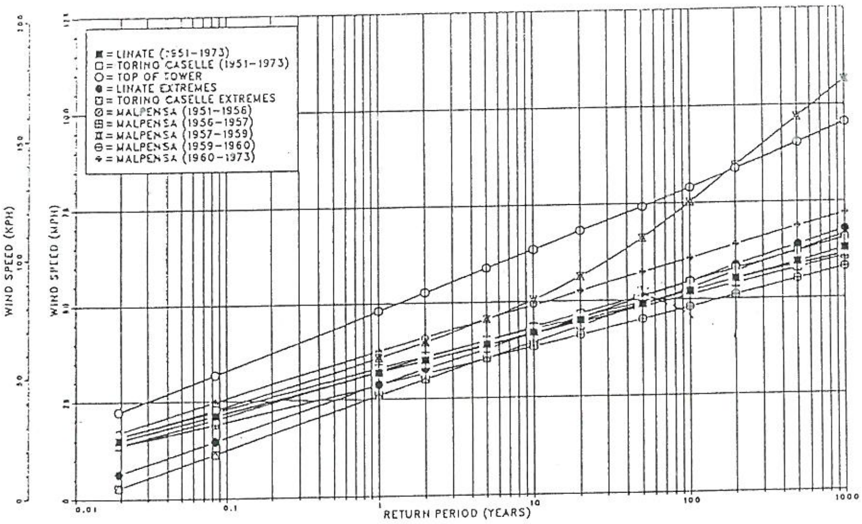


Fig. 4. Statistical analysis of the wind speed for Torino Caselle.

statistical investigations have been carried out relevant to the intensity and the direction of the wind for this site, with the help of the data supplied by the "Centro nazionale di metereologia e climatologia aeronautica" of the Military Air Force in the station of Torino Caselle (Airport).

The data about the wind which have been detected in the period from 1951 to 1978 and relevant to the maximum yearly values Vm (m/sec) of the mean speed of the wind acting from sectors having a width of 30° have been submitted to a statistical analysis and the distributive and cumulative Gumbel law has been ascribed to the sampled variable Vm:

$$F\ (V_m) = \exp - \exp\ - V_m - U)/a \quad ;$$

from this law it is possible to deduce the relation between the yearly maximum generic event Vt and the relevant return period t (years):

$$V_t = U - a \ln\ - \ln\ (1-1/t) \quad .$$

As the quality and quantity of recordings is quite scanty, the statistical analysis has been correlated by the "parent" analysis on monthly and daily measurements obtained at intervals of 6 h. The results which have been obtained supply for To = 50 years the value

$$\bar{V}_{ref} = 23.50 \text{ m/sec}$$

In accordance with the CNR-UNI 10012/5 specifications, it has been adopted:

$\alpha_t$ (topography coefficient) = 1

$\alpha_r$ (return coefficient) = 1,

while as to the profile coefficient $\alpha_z$ , the structure belonging to category 2 (paragraph 5.2.4) is considered for directions of the wind: 0°, 30°, 60°, 90°. The structure belonging to category 4 is considered for directions of the wind: 120°, 150°, 180°.

Considering Zref = 34.50 m (for safety), from the relation:

$$\alpha_z = K \ln (z/z_o)$$

it is obtained:

$$\alpha_z = \begin{cases} 1.242 & (\text{2nd category} \quad K = 0.19 - z_o = 0.05) \\ 0.247 & (\text{4th category} \quad K = 0.27 - z_o = 1.50) \end{cases}$$

Finally, it is possible to get the following design values:

- mean kinetic speed

$$\bar{V}_z = \begin{cases} 29.187 \text{ m/sec (2nd category)} \\ 19.904 \text{ m/sec (4th category)} \end{cases}$$

- mean kinetic pressure

$$\bar{P}z = \begin{cases} 532 \text{ N/m}^2 & \text{(2nd category)} \\ 248 \text{ M/m}^2 & \text{(4th category)} \end{cases}$$

### 2.2.4. Experimental analysis of the distribution of the wind. Determination of pressure coefficient Cp

In order to get quite realistic indications about the effective range of the pressure (which cannot be got from the existing specifications), the relevant experimental researches have been performed in the wind tunnel with a model of the stadium including the surrounding environment.
The determination of the local mean static pressures on the extrados and intrados of the covering has been carried out on a rigid model in scale 1/200.
Considering the symmetries, the instrumentation has been placed on the extrados for the quarter N-E and on the intrados for the quarter S-W.
The measuring points 72x2 (extrados and intrados) allow to get the measurements of the local static mean pressure for 7 incidences of the wind every 30° on 180°.

Incidences I = 0°, 30°, 60°, 90° are associated to the roughness of the ground characterized by $\alpha = 0.16$ (flat area with various isolated obstacles), while incidences I = 120°, 150°, 180° are associated to a roughness characterized by $\alpha = 0.28$ (urban areas where the 50% at least of the buildings have a height greater than 15 m).
The vertical gradients typical of the mean speed are defined and simulated according the following law:

$$\bar{V}_z/V_g = (\bar{Z}/Z_g)^{\alpha}$$

where:

$\bar{V}_z$ = mean speed at level z ,
$V_g$ = speed at level $z_g$ ,
$z_g$ = characteristic height
$\alpha$ = exponent whose value depends on the type of roughness.

The tests have been carried out in the turbulent state tunnel of the Western Ontario University. The results have been presented in the form of pressure coefficients relevant to a hour mean speed at a reference height of 30 m.

The relation between the reference speed Vr and the design speed which has been adopted is:

$$\bar{V}r = \bar{V}d \,.\, Ka \,.\, Kz \,.\, Kt \,.\, Kr$$

$K_r$ = roughness coefficient of the ground (= 0.64),
$K_t$ = turbolance coefficient (= 1.07),
$K_a$ = mean speed coefficient (= 0.95,)
$K_z$ = height coefficient (= 1.335).

The reference pressure is therefore:

$$P_r = 260 \text{ N/m}^2 .$$

The results supply the peak local coefficients and the average area coefficients. The positive sign indicates that the induced load is downward. Figures 5a and 5b show the pressure peak local coefficients Cp and the spatial averaged coefficient in an area of 200 square meters around.

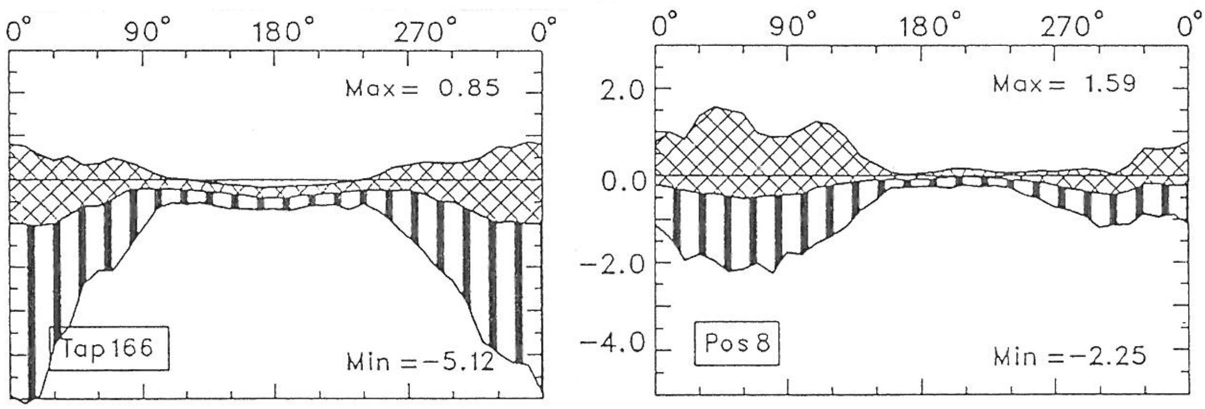


Fig. 5. (a) Pressure peak local coefficient.
(b) Spatial averaged pressure coefficient.

The peak local coefficients are used for the dimensioning of the local connections of the covering material, while the spatial averages on areas included between 100 and 200 $m^2$ are adopted for the dimensioning of the secondary structures.

## 2.4. Indirect actions

In accordance with the CNR-UNI 10012/85 specifications, it is assumed a temperature deviation with respect to the medium local one of Dt = ± 25°C for metallic structures which are directly exposed to atmospheric actions).
The temperature variation is considered uniform as any sensible temperature difference cannot be foreseen among the single elements for difference in the exposure.
The coefficient of thermal expansion is:

$$\alpha = 0.000012°C-1 .$$

## 2.5. Load static conditions

The load static conditions which have been considered are the following.

2.5.1. Pre-stress state.

2.5.2. Pre-stress state + permanent loads (state "O").

2.5.3. Action of the snow
- qs (To = 50) = 965 $N/m^2$
- qs (To = 175) = 1220 $N/m^2$.

2.5.4. Pseudo-static action of the wind
- qv = 532 $N/m^2$; Cp = -0.8 (0°, 30°, 60°, 90°)
- qv = 248 $N/m^2$; Cp = -0.8 (120°, 150°, 180°).

2.5.5. Temperature variations
DT = ± 25°C.

## 2.6. Load dynamic conditions

2.6.1. Dynamic action of the wind in direction $\alpha$ = 0°.
2.6.2. Dynamic action of the wind in direction $\alpha$ = 90°.

## 2.7. Laad combinations

2.7.1. Non-symmetrical static combinations:

a) Symmetry x

A. STATE "O"
B. STATE "O" + snow (according to Fig. 3a - N)
C. STATE "O" + snow (according to Fig. 3a - N) + DT = -25°C
B. STATE "O" + snow (according to Fig. 3a - S)
C. STATE "O" + snow (according to Fig. 3a - S) + DT = -25°C
E. STATE "O" + wind (0°) + DT = +25°C
F. Pre-stress only.

b) Symmetry y

A. STATE "O"
B. STATE "O" + snow (according to Fig. 3b - E)
C. STATE "O" + snow (according to Fig. 3b - E) + DT = -25°C
B. STATE "O" + snow (according to Fig. 3b - O)
C. STATE "O" + snow (according to Fig. 3b - O)
E. STATE "O" + wind (0°) + DT = +25°C
F. Pre-stress only.

2.7.2. Symmetrical static combinations

A. STATE "O"
B. STATE "O" + uniform symmetrical snow ($\mu$ = 1)
C. STATE "O" + uniform symmetrical snow ($\mu$ = 1) + DT = -25°C
D. STATE "O" + uniform symmetrical wind + DT = +25°C
E. STATE "O" + uniform symmetrical wind + DT = +25°C
F. STATE "O" + uniform symmetrical wind + DT = +25°C
G. STATE "O" + uniform symmetrical wind + DT = +25°C
F. Pre-stress state.

2.7.3. Dynamic combinations

A. Action of the wind ($\alpha$ = 0°) + snow (according to Fig. 3a)
B. Action of the wind ($\alpha$ = 90°) + snow (according to Fig. 3b).

3. GEOMETRIC-MECHANICAL MODEL

The geometric model has been obtained by using the symmetry axes of the structure and for a quarter of the structure by means of the classical method of research of state "O", typical of the membrane structures.
Once a pre-stressing level has been fixed on a set of interconnected nodes and rods, the research of the initial geometric-tensional configuration is carried out by using the equilibrium conditions for each internal node, according to the following mathematical model:

$$\sum_i \bar{S}_{ki} + \bar{P}_k^o = 0$$

where:

$\bar{S}^o_{ki}$ = force in the generic rod k1 in STATE "O"

$\bar{P}^o_k$ = load applied to node k in STATE "O"

The summative is extended to all the rods converting in k .
The final geometric configuration has been visualized in Fig. 6.

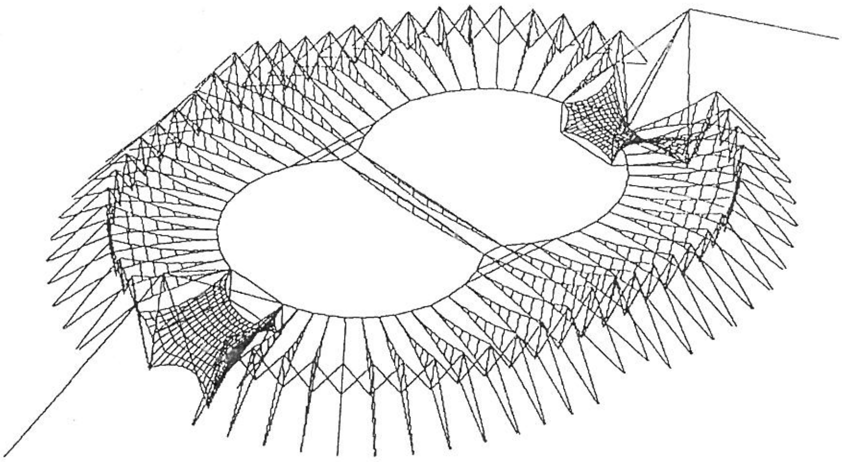

Fig. 6. Mathematical model of the cable structural system.

# 4. VARIATIONS OF THE STATE: THE MATHEMATICAL MODEL

## 4.1. The static analysis

The mathematical model implemented in TENSO program (2) considers the analysis in the range of geometric non-linearity of the scheme of net-interconnected rods through displacement method, according to what is expressed by:

$$[K]\ \{\delta_k\}^r = \{P_k\} - \{R_k\}^{r-1}$$

where:

$[K]$ = matrix of the global stiffness ($[K] = [Kg] + [Ke]$)

$\{\delta k\}^r$ = vectors of displacements obtained at the r-th iteration

$\{Pk\}$ = load applied to node k

$\{Rk\}^{r-1}$ = residual load of the non-linear terms.

### 4.1.1. Deformation state

For each load combination which has been considered, the deformation state has been calculated.

More significant synthetic results are shown in the following Table 1.

Table 1 - Maximum displacements

Load combinations

A. pre-tension + permanent loads
B. pre-tension + permanent loads + snow
C. pre-tension + permanent loads + snow - dt
D. pre-tension + wind (5)
E. pre-tension + wind (6)
F. pre-tension + wind (7)
G. pre-tension + wind (8)
H. pre-tension

| | ux (cm) | | | |
|---|---|---|---|---|
| Combination | max | node | min | node |
| B | 19.16 | 21 | -10.70 | 30 |
| C | 17.60 | 21 | -10.70 | 30 |
| D | 3.82 | 30 | - 9.56 | 21 |
| E | 3.55 | 30 | - 8.25 | 21 |
| F | 3.63 | 30 | - 9.54 | 21 |
| G | 3.56 | 30 | - 8.36 | 21 |

| | uy (cm) | | | |
|---|---|---|---|---|
| Combination | max | node | min | node |
| B | 8.79 | 245 | - 8.16 | 255 |
| C | 8.36 | 261 | - 7.84 | 255 |
| D | 2.80 | 255 | - 4.66 | 261 |
| E | 3.11 | 255 | - 4.48 | 261 |
| F | 3.14 | 255 | - 4.01 | 261 |
| G | 3.67 | 262 | - 3.70 | 245 |

| | uz (cm) | | | |
|---|---|---|---|---|
| Combination | max | node | min | node |
| B | 10.77 | 1 | -73.11 | 21 |
| C | 19.02 | 1 | -69.14 | 21 |
| D | 32.53 | 21 | -19.75 | 1 |
| E | 31.09 | 21 | - 1.43 | 30 |
| F | 32.51 | 21 | -19.67 | 1 |
| G | 31.36 | 21 | - 1.44 | 30 |

4.1.2. Stress state

In the same way as for the deformation state, the most important results of the static analyses are illustrated here below and they are relevant to the various load combinations and to the topologic schems associated to it.

Figures 7, 8 and 9 show the values of the stresses for the various load combinations and relevant to the positions of the tensostructure (1 to 28) for the columns, the central ring and the upper anchorages.

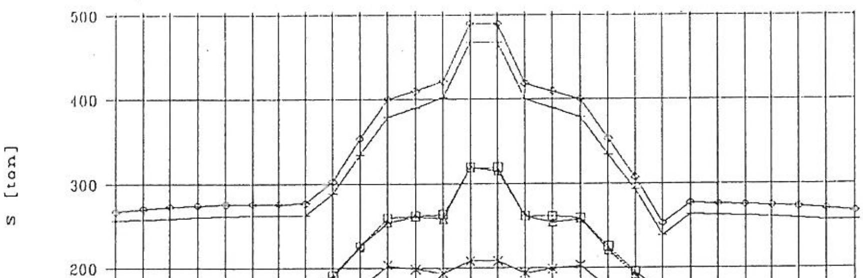


Fig. 7. Stresses in the columns.

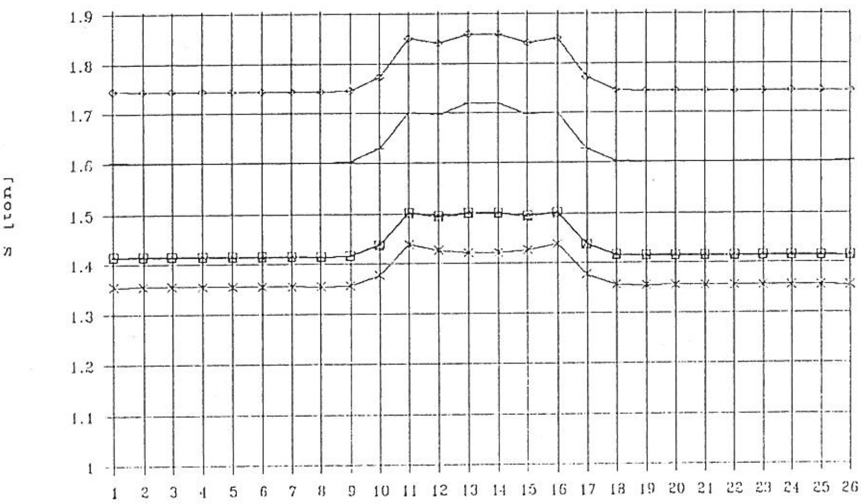


Fig. 8. Stresses in the main ringg.

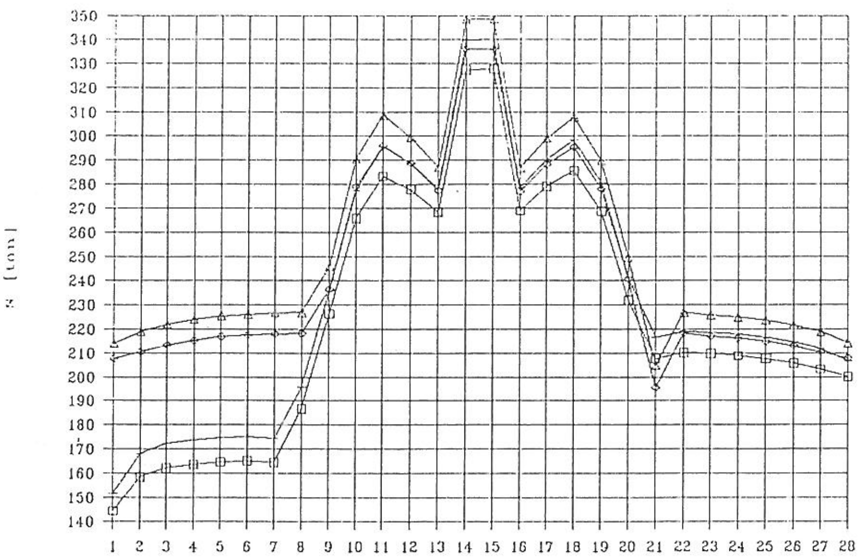


Fig. 9. Stresses in the anchorages of the upper cables.

## 4.2. Dynamic analysis

### 4.2.1. Research of the dynamic characteristics of the structure - Frequency analysis

In order to get the dynamic characteristics of the structure, a preliminary analysis of the frequency equation has been carried out using the SUBSPACE ITERATION technique.
The frequency equation without damping which has been used is:

$$||(Ke + Kg) - \omega^2 M|| = 0$$

where:

Ke = matrix of the elastic stiffness
Kg = matrix of geometric stiffness
M = matrix of the masses

The first 8 eigenvalues and the corresponding vibrations have been considered.
The analysis of the frequency has been performed for three different cases corresponding to the presence of 0%, 30%, 50% of the mass relevant to the snow load.

4.2.2. Analysis of the structural response - Aleatory dynamic analysis in the domain of the frequencies

In order to perform a complete control of the dynamic characteristics of the structure, a modal aleatory dynamic analysis has been worked out in the domain of the frequencies in the range of geometric linearity.
This method of dynamic analysis needs the determination of the power spectral density of the fluctuating part of the wind, at the height of the covering.
In accordance with ESDU spectrum we have:

$$Sv(n) = 30.85.\bar{V}/[1 + 337500.(n/\bar{V})^2]^{5/6}$$

where: n = frequency.
The expression of the correlated spectral density is (function of spatial correlation):

$$Svj\ (p1, p2, n) = Sv(n)\ Coh\ (p1, p2, n)$$

The spatial correlation function is always:

$$\sqrt{Coh(n,D)} = \sqrt{Coh(n,P1, P2)} = \exp\{- n/V \sqrt{(Cx^2..Dx^2 + Cy^2.Dy^2)}\}$$

where Cx = 2; Cy = 16 and x is the wind propagation direction.
In terms of nodal forces, the PSD is expressed by:

$$Sfj\ (p1, p2, n) = Sp\ (p1, p2, n)\ Aj$$

Matrix Sfj is a NxN matrix for each frequency which has been considered, with Aj = influence area of node j .

- Power spectral density of modal excitations.
By using the modal analysis and considering the first m vibration modes, we have:

$$[S_\phi(n)] = [\Phi]^T [Sf] [\Phi]$$

where columns of $[\Phi]$ contain the n vibration modes which have been considered: for each node, the vertical components of the N nodes of the covering.
The dimensions of the $[S_\phi(n)]$ are mxm .

- Power spectral density (PSD) of displacements

The passage from the PSD of the modal excitation to the PSD of the nodal displacements is supplied by:

$$[S_\delta(n)]\, mxm = [H^*(n)]\, S_\phi(n)]\, [H(n)]$$

where H(n) is a diagonal matrix containing the response complex functions in frequency of the different modes which have been considered, and H*(n) is its complex conjugate.

The expression of the responses in frequency is:

$$H(i\Omega) = 1/\{ Km\, [1+2\, i\lambda m(\bar{n}/nm) - (\bar{n}/nm)^2]\}$$
$$H^*(-i\Omega) = 1/\{ Km\, [1+2\, i\lambda\, m(\bar{n}/nm) - (\bar{n}/nm)^2]\}$$

- Generalized response - displacements - stresses

The generalized response of the displacements is obtained by passing from rank mxm to the one of global freedom degrees 3nx3N. Therefore, as to real displacements we have:

$$[Sg\ (n)] = [\Phi]\ [Sg(n)]\ [\Phi]^T$$

where $[\Phi]$ contains all the modes.

The generalized response about the stresses in a generic number of elements is obtained by following the matrix operation:

$$[Sm(n)] = [Ke][Sg(n)]^*\ [Ke]^T$$

where $[Sg(n)]^*$ is an elementary submatrix of $[Sg(n)]$ .

- Maxima of the dynamic response

The knowledge of the power spectral density of each response quantity which has been chosen (displacements or stresses) allows to get the maxima probability function of a given response value in a pre-fixed interval of time. In the hypothesis of Gaussian distribution of the process, the probability function is:

$$Pr\ \{Ymax\} = 1 - \exp\ (-\nu(Y)DT)$$

where the mean frequency of threshold crossing is given by the expression:

$$\nu(Y) = \{\lambda/\lambda o\}^{1/2}\ \exp\ (-Y^2/2\nu o)$$

The mean value of Ymax can be obtained by:

$$Ymax = \{(2\ \ln \nu_o\, DT)1/2 + 0.577/[2\ \nu_o \ln\, DT]^{1/2}\ \}$$

The mean value of the dynamic maxima for DT = 10 min summed to the static response due to the mean pressure forms the verification value for the service state.

The results of the dynamic analysis are shown in Fig. 10.

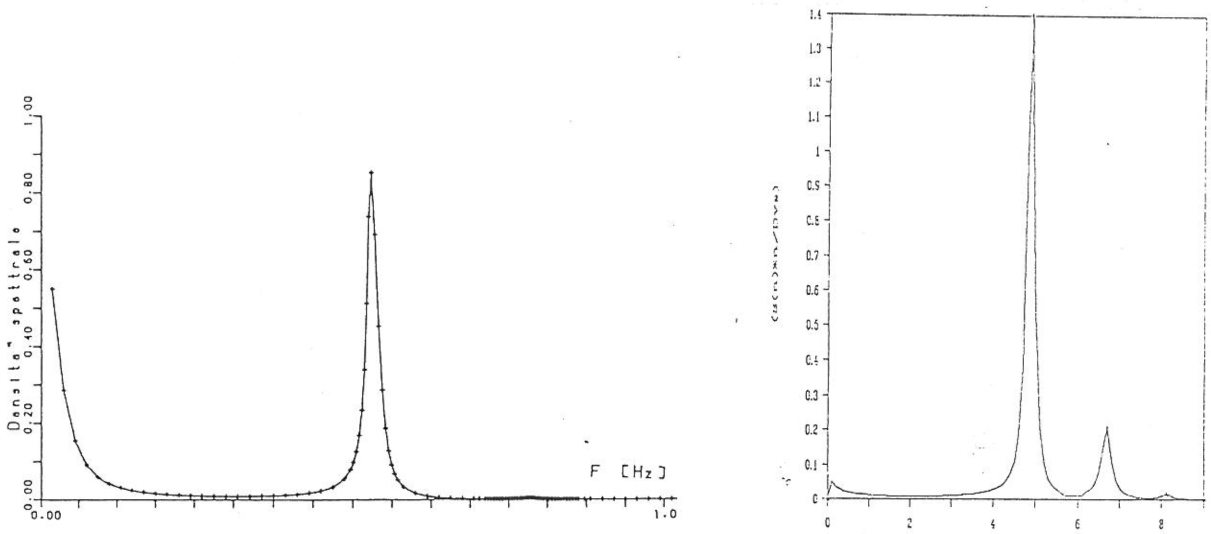


Fig. 10. (a) Displacement spectra of joint n. 37.
(b) Force spectra of element no. 121.

5. BIBLIOGRAPHY

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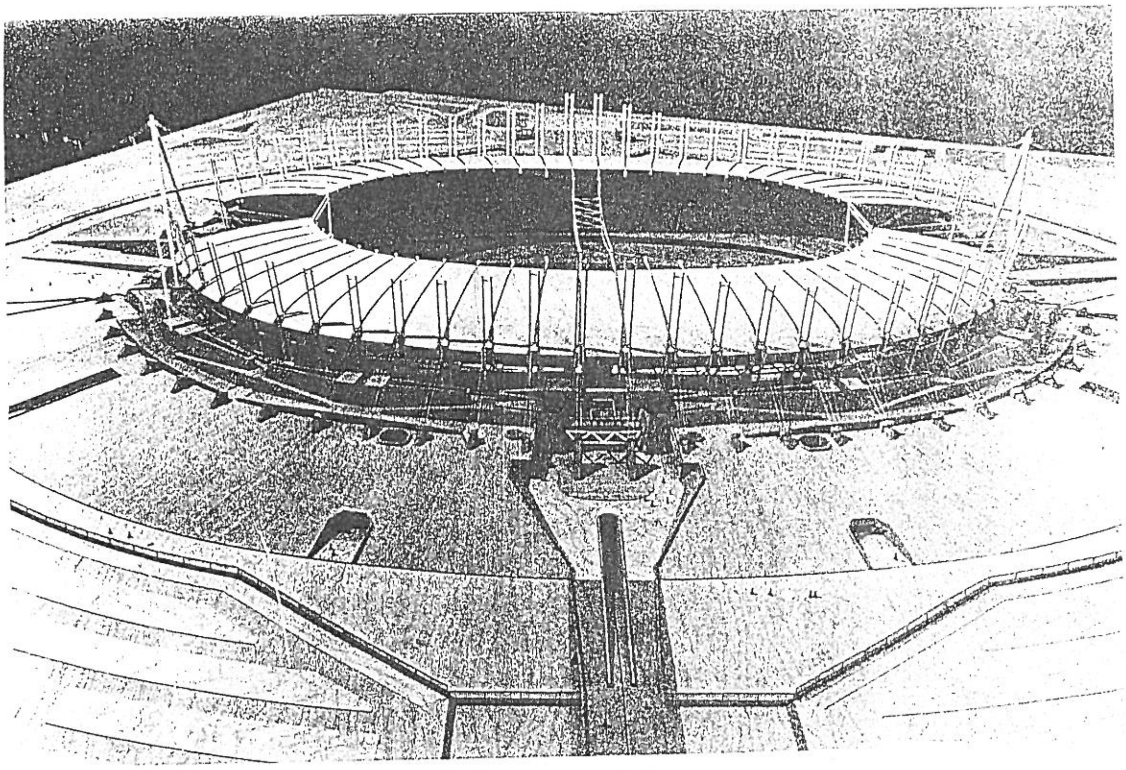
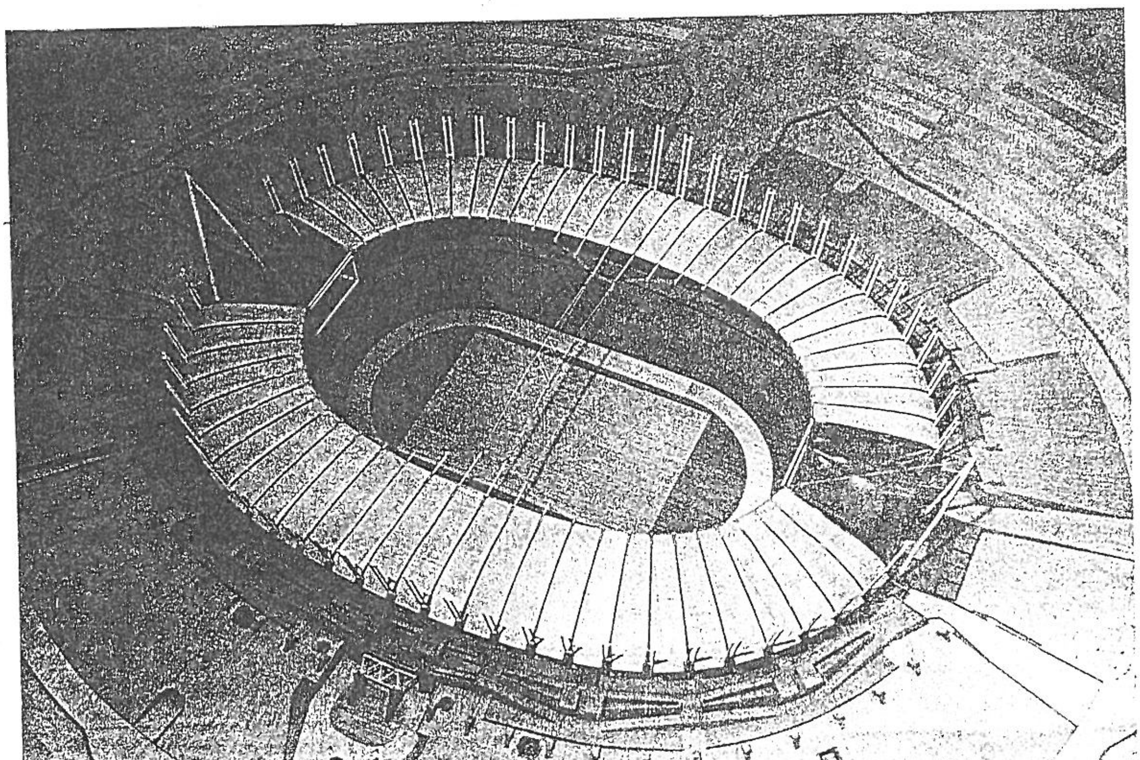
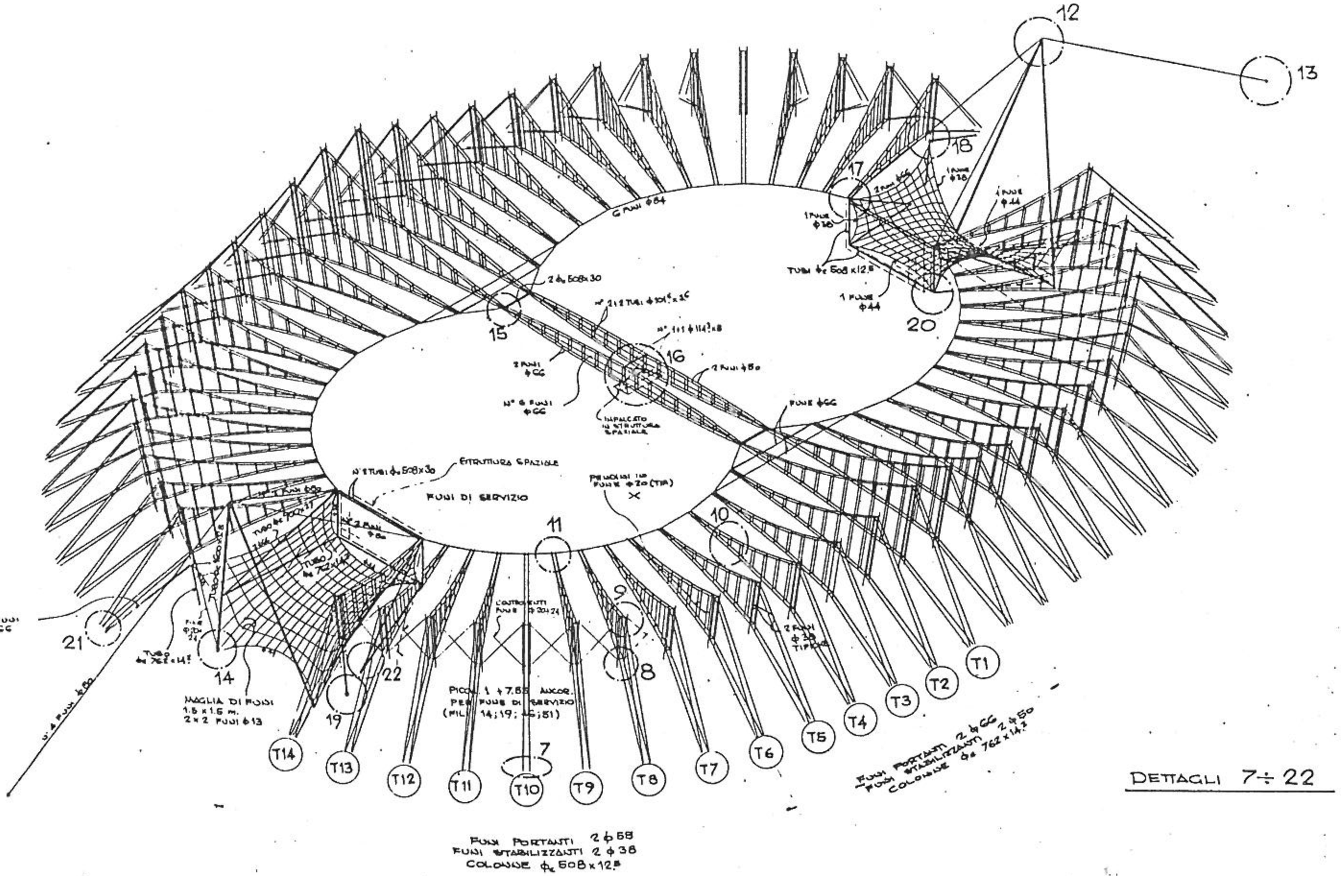
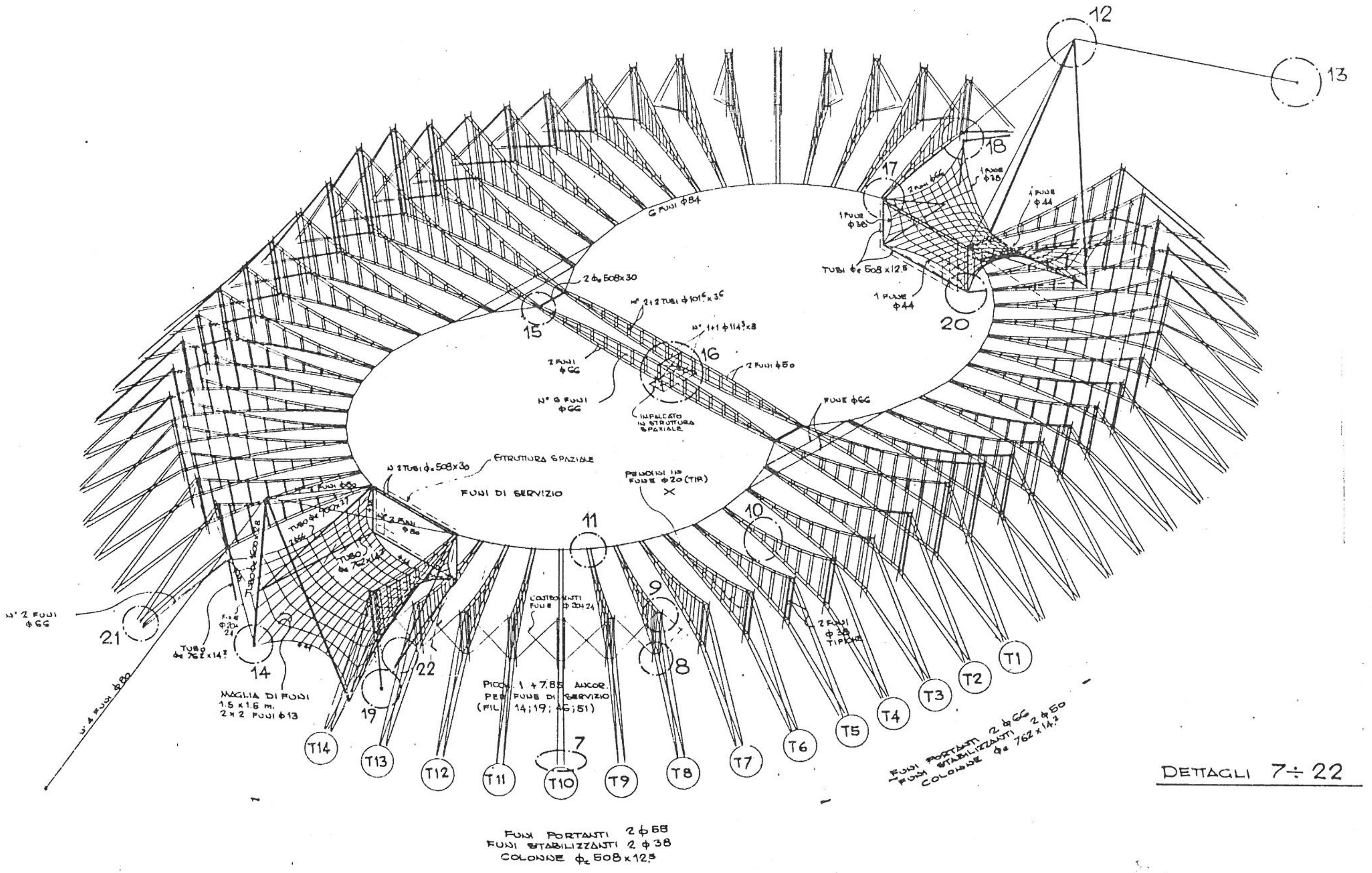







DETTAGLI 7 ÷ 22


DETTAGLI 7÷22
FUNI PORTANTI 2 φ 58
FUNI STABILIZZANTI 2 φ 38
COLONNE φe 508 x 12.5
FUNI PORTANTI 2 φ 66
FUNI STABILIZZANTI 2 φ 50
COLONNE φe 762 x 14.2
FUNI DI SERVIZIO
STRUTTURA SPAZIALE
6 FUNI φ84
MAGLIA DI FUNI
1.5 x 1.5 m.
2 x 2 FUNI φ13
N° 2 FUNI
φ66
FUNE φ66
2 FUNI φ50
2 FUNI
φ38
TIPICHE
TUBI φe 508 x 12.5
2 φe 508x30
PENDINI IN FUNE φ20 (TIR)