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Structural design for seismic and wind action of a telecommunication tower-case studi

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Abstract

This paper contains the results from the dynamic analysis carried out for the design of a flexible structure of a complex form for which it was difficult to estimate, a priori, whether earthquakes or winds have a more unfavourable effect upon its dynamic behaviour. The matter at stake is analysis of a telecommunication tower with a total height of 155 meters that is being built on Vodno mountain, in the vicinity of Skopje, at an altitude of 1034 meters. The central part of the tower is in the form of an irregular quadrangle designed as a reinforced concrete core from the terrain up to the level of 112 m. Above the RC core, there rises an antenna truss with a square cross-section running to the level of 140 m. Upon it, a multi-purpose antenna with a height of 15 m will be mounted. The reinforced concrete core is designed to have four circular working platforms starting at 56 meters along height with the last one being at the level of 82 m with a diameter of 18 m. The first three platforms are placed on three different sides of the tower while their centres lie at distance of approximately 2.5 m beyond the face of the tower. The fourth platform is placed centrically around the tower, at the level of 82 m. All of them cover an area of approximately 200 m^2 each. The structure of the platforms is designed as a 3D steel truss in the form of a cylinder that is rigidly anchored to the reinforced concrete core. Designed on the structure are also four decorative elements in the form of rafters that extend from the terrain to below the platforms on all four sides of the core. For the considered structure, a study for definition of the seismic effect of near- and far field seismic zones as well as a study for definition of the wind effect were carried out by testing of the tower model in an aerodynamic tunnel for the purpose of defining the aerodynamic pressures (both positive and negative) in eight directions of wind effect. The dynamic analysis of the structure showed presence of many modes along the three axes. During structural design, many analyses were carried out to obtain an objective response for definition of the most unfavourable dynamic effect and appropriate proportioning of the structural elements.

Key words: structural design, mega structures, wind tunnel testing, numerical modelling, shaking table testing

1 Introduction

To design a specific structure as is the Telecommunication Tower in Skopje, with a specific and non-standard, combined structural system, detailed investigations and input data are necessary for the purpose of appropriate, high quality analysis of the structure from the aspects of usability, stability and durability. If one takes into account the fact that the Telecommunication Tower is located in an active seismic zone of IX degree MCS, on the top of Vodno hill in Skopje, directly exposed to possible wind effects, the design of the structure required a special treatment. It should be based on detailed investigations of the site seismicity as well as expected wind effects upon both the integral structure and its different structural parts. Analytical investigations of expected intensities and additional effects were verified through experimental tests on the model of the structure in a wind tunnel. These were the main input bases for design of the architectonic structure of a specific form and purpose that also required corresponding criteria of design of a structural system that had to satisfy the parameters of seismic safety, behaviour and safety against wind effect as well as appropriate design of the structure and its elements of different materials and their connection into an integral structure and analytical modelling as the basis of final design and construction of the structure in the specific conditions presented in this paper. The authors of this paper have made an attempt to present the procedure of design of the telecommunication tower that is being built on Vodno hill, in the immediate vicinity of Skopje city. Considering the architectural shape of the tower [1], the terrain topography, the exposure to strong wind or earthquake effects, the specific requirements of users of the space, the design engineers were required to propose a non-standard design procedure. Within this limited space, the authors of this paper have tried to explain the applied procedure of design of the considered, relatively complex structure.

2 Description of structural system and components

The Telecommunication Tower structure in Skopje financed by AEK (Agency for Electronic Communications of the Republic of North Macedonia) has been designed as an attractive architectonic structure of a unique form and shape by the design firm "Stone design" from Skopje (Fig. 1). The structure consists of three parts: a reinforced-concrete shaft, an antenna truss and four platforms as well as decorative "horn" elements. At plan, the reinforced concrete shaft represents an irregular quadrangle with a changed shape in the upper storeys. The dimensions of the quadrangle change along height since, architectonically, three out of four sides are inclined upwards by 1.4 cm per meter height.

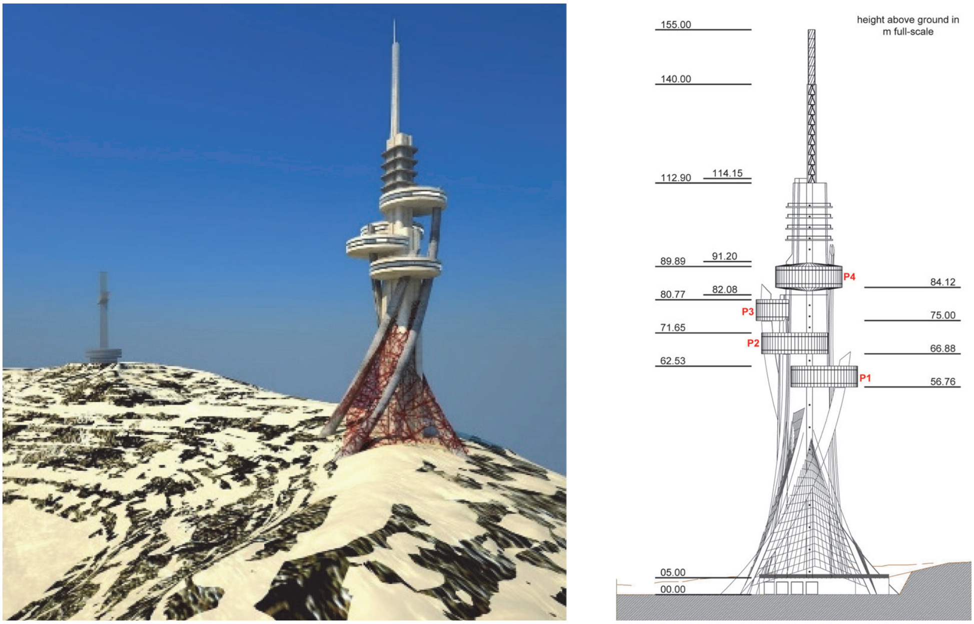

Figure 1. The Telecommunication Tower in Skopje

Only the north side of this structure is vertical. In the inner part, there are secondary vertical, reinforced concrete walls that shape the space forming the platforms, the vertical channels for the installations, the staircase area, and the elevators. The thickness of the external walls is variable along height so that it amounts to 55 cm along the first ten heights (30 – 40 m), 50 cm along the second ten heights, 45 cm along the third ten heights and 45 cm along the last eight heights. The total height of the reinforced concrete shaft is 112.48 m. The interior walls have a constant thickness of 20 cm, while the platforms and the slabs at the staircases have a thickness of 15 cm. The antenna truss is anchored to the reinforced concrete shaft at level 112,48 m and has ten storey heights of 275 cm each, i.e., the upper part of the truss is at level 140,0 m. It is designed as a steel 3D truss with a square plan proportioned 250 x 250 x 12,5 cm (external dimensions). At the four corners, square hollow profiles 300 x 300 x 12,5 mm are placed along the first two heights and then the profiles are narrowed at 250 x 250 x 8.0. The four platforms have different purposes, namely, the first three are used for installation of the telecommunication equipment of the users of the telecommunication, as well as working area of the companies, while the fourth platform houses a café restaurant upon a rotational area. The shape of the three platforms is similar at plan. It represents part of a circular segment that has a circular form on the external peripheral side, while the shape of the interior part is governed by the intersection with the shaft. Structurally, the three platforms are identically solved, namely the floor and the roof platform represent a structural system formed by a peripheral complex truss 2 x 1,0 + 3,70 m and two, vertically interconnected inner rings with a height of 1,0 m. The inner rings and the external circular truss are anchored into the reinforced concrete shaft. The developed length of the external truss is 44,0 m, while the biggest cantilever dimension of the platform is 12,0 m. The diameter of the inner rings is 3,0 m, from which, radially toward the external truss, 11 trusses with a height of 1.0 m are designed. These radial trusses are horizontally and vertically mutually connected by bracings. Specific elements of the structure and the structural system are the so called “horns” that are designed on all sides of the base and extend from the foundations to the lower level of the corresponding platforms. The horns are designed as spatial truss-like structures with variable cross-section resting on contacts with the central concrete core through specially designed ties. The foundation structure of the central core is designed as a foundation slab, while the horns are founded on special foundation plates. The structural system of the tower is shown in Figure 2.

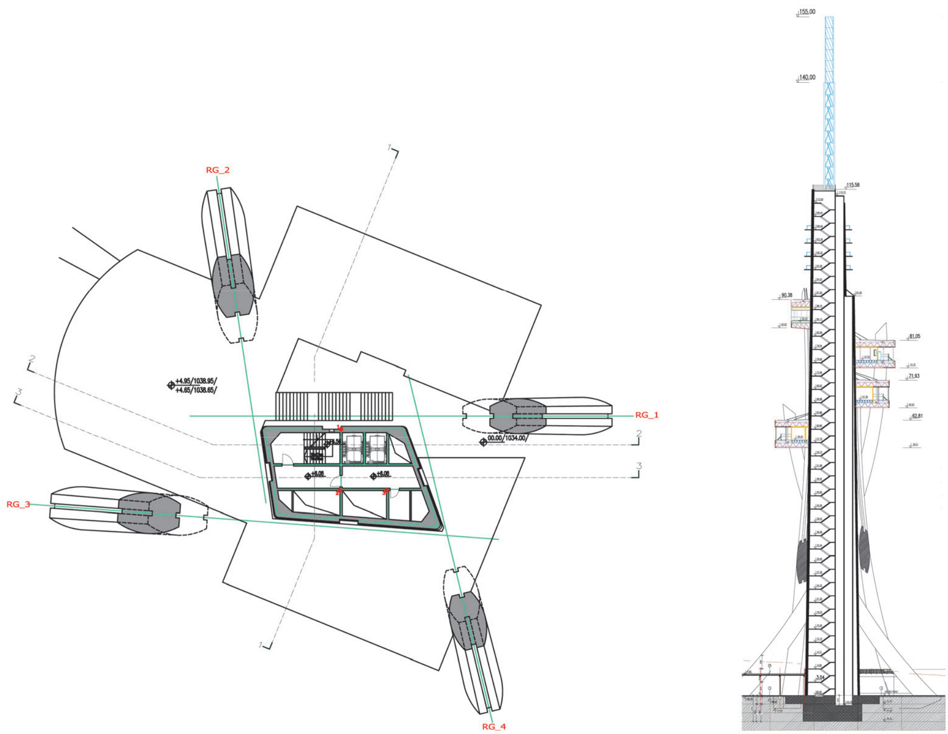

Figure 2. Plane and cross section of the Tower.

3 Analytical modelling

Complex geometry and different material used for shaft, platforms, and antenna requires adoption of analytical model that will provide correct results for internal forces, stress state and level of deformations. High demands for the analytical model, as well as objectiveness of the results, should be guaranteed by verified and widely accepted computer program for structural analysis and design.

This tower has been modelled and analysed as a 3D structure using SAP2000 computer program (Fig. 3). The shaft (exterior and interior shear walls, and slabs at each level) has been modelled by 31846 shell elements, each having area between 0.4 -0.6 m^2. The steel structures, four platforms and the antenna truss have been modelled with 6944 frame elements. So, the analytical model has, 29831 joints and 126813 edges.

The structure has been analysed and designed for the following loads: dead weight, serviceability technological load, live load, snow and ice, wind, and earthquake effect.

For gravity loads and wind loads, static analysis have been conducted, while for earthquake loads dynamic time history analysis have been performed.

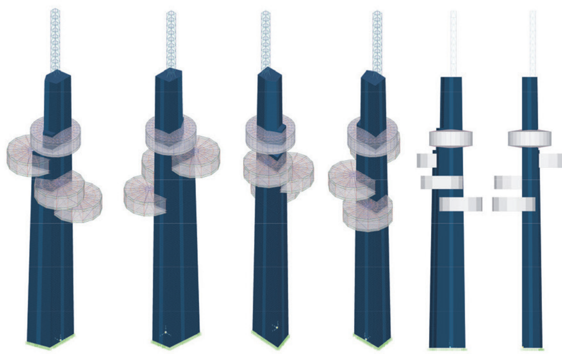

Figure 3. Analytical model

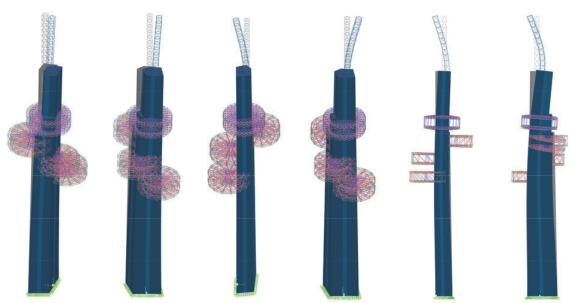

Figure 4. Mode shapes: Mode 1, 2, 3 4, 5, 7 (from left to right)

At first modal analysis has been conducted, and total 24 natural frequencies and corresponding mode shapes have been identified. In Table 1 first 10 natural periods and frequencies are presented, while on Figure 4 6 modes are graphically presented.

Table 1. Modal Periods and Frequencies

Mode	Period [s]	Frequency [Hz]	DESCRIPTION
1	2.08	0.48	core first-y
2	1.32	0.75	core first-x
3	0.57	1.77	antenna mast first - y
4	0.54	1.85	antenna mast first - x
5	0.41	2.44	core second - y
6	0.32	3.17	core torsion
7	0.28	3.53	core second - x
8	0.27	3.65	first platform vertical
9	0.26	3.87	third platform vertical
10	0.25	4.06	second platform vertical

4 Design of structure for earthquake motion

The seismic design parameters of the site of the antenna tower have been defined based on detailed investigations of the site carried out by IZIIS Skopje [3], while the defined seismic effects correspond to return time periods of seismic hazard of 95 and 475 years. In the uniform elastic acceleration spectrum, 5 % of the critical damping has been adopted for two directions, namely horizontal and vertical. The values of spectral acceleration for a period close to zero amount to PGA = 0.140g and PGA = 0.228 g for horizontal direction and correspond to a return period of 95 and 475 years. However, in vertical direction, they amount to PGA = 0.136 and PGA 0.222 g for the return periods of 95 and 475 years, respectively. For the defined response spectra for both directions, time histories of acceleration have been obtained in a digital form.

Total 12 time histories of artificial earthquakes have been generated [3], set of three for return period of 95 and 475 years, and for horizontal and vertical direction accordingly.

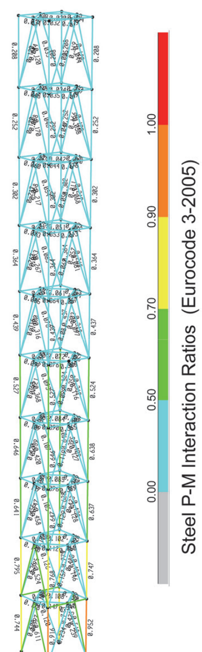

Figure 5. P-M interaction ratios for antenna elements

For dynamic time history analysis of the tower one set of two horizontal and one vertical component have been selected, based on the unfavourable influence on the structural response. The combination of the horizontal and vertical components of the seismic action has been done according to the chapter 4.3.3.5. of the EN 1998-1:2004 [10]. Total 11 combinations have been made and used as input for time history analysis.

5 Design of structures for wind load

The design of standard structures for wind effect is regulated by the national standards of each country. Wind load upon structures depends on several factors as are the physical properties of air, the characteristics of winds, namely the direction and the basic wind velocity, the relief, and the altitude at which the structure is built, particularly the topography and the roughness of the terrain, as well as the characteristics of the structure in the sense of material of which it is built, stiffness of the structure, dynamic characteristics, and its form.

The wind velocity at each point of the territory of a country is defined in the national maps of wind velocity expressed in m/s. This basic wind velocity is usually given by mean value per hour, but the regulations for design of structures frequently require mean values per 10 min or less, meaning that the basic wind velocity should be increased. The mean wind velocity $V_m(z)$ at height z above the terrain depends on the terrain roughness, the orography, and the basic wind velocity V_b. It is determined by the following expression (1):

$$V_m(z) = K_z \cdot C_0(z) \cdot V_b \tag{1}$$

where K_z is the roughness factor, while $C_0(z)$ is the orography factor, which is a terrain level equal to 1.

The roughness factor K_z accounts for the variability of the mean wind velocity at the site of a structure due to the height above the ground level and the roughness of the terrain upwind of the structure, in the considered wind direction. The roughness factor is determined by the following expression:

$$K_z = \sqrt{b} \cdot \left(\frac{z}{10} \right)^{\alpha} \tag{2}$$

where b and α are the factor of roughness determined by the standards for the considered terrain category.

For the site of the tower and altitude higher than 1000 m, according to the code, the basic wind velocity is 35.0 m/s. The factor for time averaging interval of 10 min – mean value is 1.1, thus $V_{m,5010}$ = 38.0 m/s and this wind velocity is constant from level zero of the structure up to the height of 10 m.

The mean velocity pressure of the flow can be calculated from:

$$q_{m,T,z} = q_{m,T,10} \cdot S_z^2 \cdot K_z^2 \tag{3}$$

where S_z is the topography factor. In accordance with the national code for structures at an altitude of over 1000 m, it should be 1, while,

$$q_{m,T,10} = \frac{1}{2} \cdot \rho \left(V_{m,50,10} \cdot k_t \cdot K_T \right)^2 \cdot 10^{-3} \text{ [kN/m}^2\text{]} \tag{4}$$

where ρ is the air density.
The peak velocity pressure can be calculated from the following equation:

$$q_{g,T,z} = q_{m,T,10} \cdot G_z \tag{5}$$

where G_z is the dynamic coefficient, which depends on the wind effect and the structural characteristics like the first mode of vibration and the damping capacity. According to our code, the dynamic coefficient G_z is calculated from:

$$G_z = 1 + 2 \cdot g \cdot I_z \cdot B \cdot \sqrt{1 + \left(\frac{R}{B} \right)^2} \tag{6}$$

where g is the gust factor, I_z is the intensity of the turbulence, B is a factor for the spatial correlation of the wind velocity and R is the resonance factor. In the case of this structure, G_z has been calculated to the value of 1,99.
The wind load on the structure can be calculated if eq. (5) is multiplied by the attached area and aerodynamic coefficient. The aerodynamic coefficients in this case are obtained by wind tunnel test [4] and additionally, for the new architectural design of the platforms, by using the CFD method (Computational Fluid Dynamics) [5]. The aerodynamic coefficients are evaluated for all structural components taken separately like the shaft, the platforms, and the horns from z = 0 to z = 140 m. Using the mean pressure coefficients evaluated in [4] and [5] and the peak velocity pressure (eq. 5.5), the wind load can be calculated for positive pressure, suction, and other wind effects.

$$w(z) = C_{oe} \cdot q_{g,T,z} \tag{7}$$

where Coe are the aerodynamic coefficients, while $q_{g,T,z}$ is the peak velocity pressure.

For structural analysis, the wind load (Eq. 5.7) multiplied by the corresponding area has been applied for all mesh elements. Usually, the analysis of the wind load represents a linear, properly validated numerical procedure assuming the mode shape with one sign.

For complex and large structures, all wind loads have to be properly simulated. Usually, this requires involvement of wind experts in evaluation of such effects. In this case, Prof. Dr. Hans-Jurgen Niemann, retired professor from the Bochum University, has been involved [4, 5].

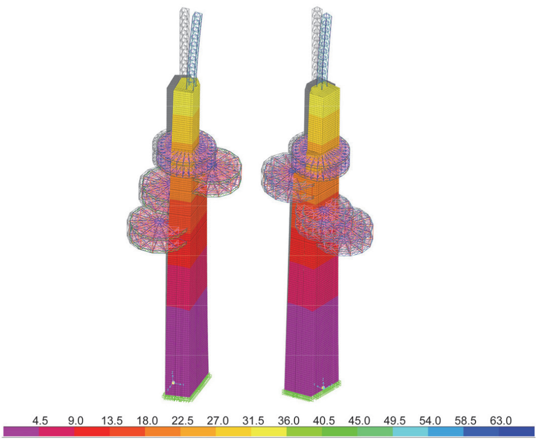

Figure 6. Displacement due to wind combination

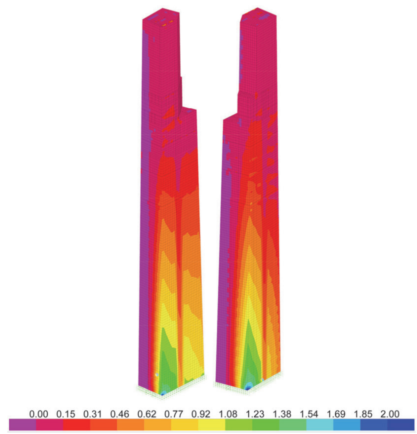

Figure 7. Required reinforcement in the shaft

The gust factor is defined as the ratio between the static wind load, which gives the maximum response of the structure during a referent time interval, usually taken as 10 minutes, and the mean wind load, which gives the mean response of the structure [4]. In our case, the gust effect was evident in the calculations of the wind load to the same façade walls of the platforms [4].

Another considered effect was the vortex shedding. Vortex shedding may occur in slender structures like the horns in this case. The vortex shedding forcing mechanisms have proved to be so complex that there is no general analytical method available to calculate the vortex structural response. The best procedure is to do model tests in a wind tunnel. Wind tunnel tests have demonstrated that the horns are exposed to the risk of vortex-induced vibrations. The critical velocity for vortex resonance is calculated by imposing $f_v = f_{1,\,horn}$ in the equation for the Strouhal number and it can be written that

$$v_{cr} = (f_{1,horn}\ b_1)/St \tag{8}$$

where v_{cr} is the critical velocity for vortex resonance; $f_{1,horn}$ is the natural frequency of vibration of the horn; b_1 is the reference length of the horn (larger side of the cross-section of the horn). To avoid vortex, Eurocode 1, part 1-4 requires

$$v_{cr} \geq 1{,}25\ v_m(z_v) \tag{9}$$

where z_v is height at which vortex shedding occurs and v_m is the mean wind velocity at height z_v. From equation 5.8 and 5.9, $f_{1,horn}$ frequency can be expressed as follows:

$$f_{1,\text{horn}} \geq 1.25\ St/b_1(z_v) \times v_m(z_v) \tag{10}$$

To avoid vortex shedding in structural analysis of the horns, namely, three horns (H2, H3 and H4), additional support had to be designed in such a way that eq. (5.10) is fully satisfied. By wind tunnel tests, it has been demonstrated that all structural components are resistant to the galloping effect. The wind load has been used for structural analysis of the shaft, the platforms, and the horns.

6 Conclusion

It was difficult, a priori, to assess during design state, whether the structure of the telecommunication tower, which is under construction on the mountain Vodno, is more sensitive to wind or earthquake actions. This real case showed that parts of the structure are more sensitive to earthquake actions (antenna), while the horns are more sensitive to wind actions. Analysis of vertical stability of the platforms showed that stress state as well as vertical deformations of the platforms are almost identical for both, wind, and earthquake actions. However, for design of the connections of the platforms with the shaft dominant action is the wind. In general, the authors of this paper recommend that in design process of such, complex, structures, both actions, wind, and earthquake, should be considered equally. For design of each individual part of the structure the most unfavourable state should be taken as relevant.

Acknowledgement

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