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Design procedure for complex structures under dynamic loads

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Abstract

In this paper, the procedure applied in the design of a 155 meter high telecommunication tower that is currently being built on Vodno mountain, in the immediate vicinity of Skopje, is discussed. The complex architectonic shape of the tower, the large number of users of the space with their own specific requirements and the expected flexibility of the structure made the design engineers define an objective and non-standard procedure of design and analysis of this complex structure. The design procedure defined by the design engineers was submitted to the Investor of the structure and it consisted from: a) Research works on the location involving geological and geotechnical investigations, geophysical surveys, definition of seismic design parameters, research for definition of wind effects and meteorological investigations; b) Definition of the technological requirements of users of the space with data on operational criteria, sensitivity to vibrations and technological loads; c) Definition of a modelling procedure, analysis and criteria of stability and serviceability. During analytical modelling, the complex structure was divided into substructures that were individually analysed and then integrated into a model consisting of a central reinforced concrete core, four independent working platforms covering an area of 200 m^2 each, an antenna unit, a truss placed on the top of the concrete core with a height of 28 m, four independent decorative elements (rafters) wrapped in a porous mesh and a foundation structure; d) An experimental programme consisting of tower model testing in an aerodynamic tunnel for definition of all possible wind effects, seismic shaking table testing of the tower model, quasi-static tests of vital joints and a fragment of the main circular truss of the platforms and quasi-static testing of the façade of the decorative elements. The application of the above stated in the design of the structure is briefly presented in this paper.

Key words: structural design, earthquake engineering, wind engineering, wind tunnel testing, shaking table testing, mega structures

1 Introduction

In structural engineering practice, there are structures for which certain assumptions that are necessarily used in their analysis and design require additional expert verification. This is particularly characteristic for complex or mega structures that may be exposed to wind and earthquakes or other dynamic effects in the course of their serviceability period. In such cases, in addition to application of valid standards, design engineers should propose realization of certain experimental investigations and application of the obtained results in the phase of improvement of the design procedure and selection of a corresponding technology of construction and control of construction quality. 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 Basic architectural data

The architectural shape of the tower, as conceived by the architects, is shown in Fig. 1. The tower is located at an altitude of 1034 m, on the top of Vodno hill, on the south side of Skopje city. So far, only the central reinforced concrete shaft has been built to the height of 112 m.

Figure 1. View of the tower - Architectural composition

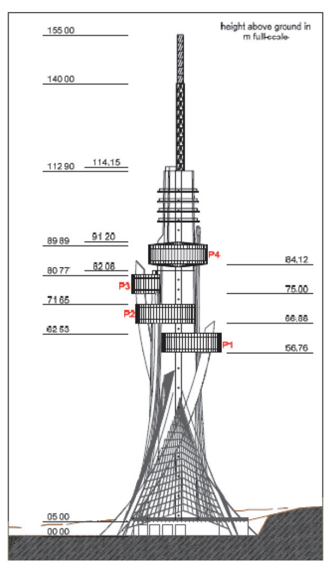

Figure 2. North view of the tower with elevation of platforms

The geometrical data on the height of the tower are shown in Fig. 2, view from the North. The cross-section of the shaft is variable along height and it represents an irregular quadrangle. The view from the fifth façade can be seen in Fig. 5. The north façade of the tower is vertical, whereas the remaining three sides converge upwards under a constant angle of 1.3 cm at each meter along height. Fig. 4 shows the appearance of the shaft cross-section, namely the change of its shape from level 0.0 to the top. The tower, which is of a complex architectonic shape, consists of the following physical components: a central shaft accommodating the vertical communications (staircases, elevators), sanitary premises and channels through which the installations run. On the tower, there are four working platforms covering 200 m^2 each, whereat the first three platforms are eccentrically drawn from the shaft (Fig. 3), while the fourth platform is centrically placed around the shaft. All platforms are of a circular shape or represent part of a circular segment with a diameter of 18.0 m. Fig. 5 shows the cross-section of the tower through platform 2 with a view downwards. Below platform 2, one can see the roof of platform 1 with the position of the tendons.
From level 112 where the reinforced concrete part of the tower ends, there starts the designed steel antenna truss ending at level 140.0 m with a special adapter for connection to an antenna, which is planned to be 15.0 m high. The antenna in this project is treated as an equipment. In addition to these functional components, the tower is also designed to have four decorative horn structures running from the terrain level and each ending below one of the platforms, incising the tower shaft. Around these decorative elements, a highly porous cable mesh is to be placed. The characteristic cross-section of the horn structures is displayed in Fig. 6. In the phase of construction of the shaft, the operators – users of platforms 1, 2 and 3 requested removal of the façade from the platform galleries to eliminate refraction of rays directed to the antennas installed in the galleries with a diameter of 2.5 m. To respond to this requirement, the architects made a new solution with an open gallery that substantially changed the wind effect. In the project, this phase is indicated as "new architectural design".

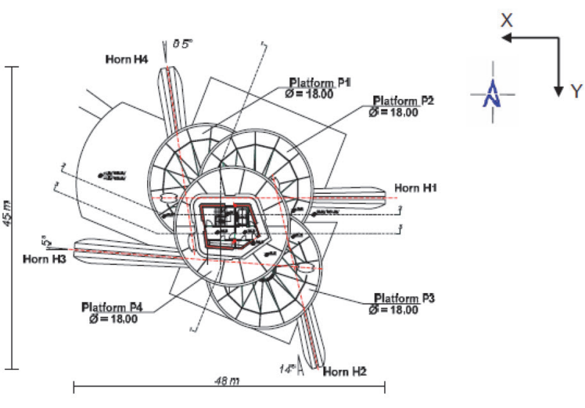

Figure 3. Top view from cross-section at height of 103.36 m

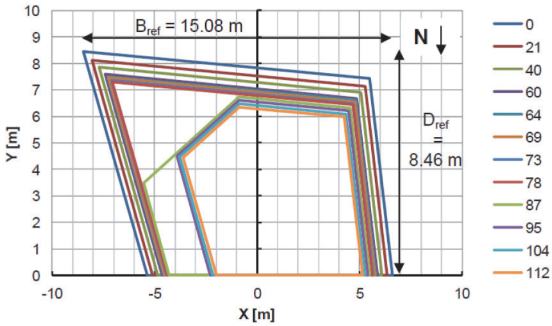

Figure 4. Cross-section of the shaft at different levels

3 Basic structural data

The analysis and design of the tower structure were performed by dividing the structure into substructures that were independently modeled and then integrated into a complex structural system. Considered as substructures were the following units:

Foundation structure. The geometry of the foundation structure at plan is shown in Fig. 7, whereas Fig. 8 displays its cross-section. The figures show that the foundation consists of two parts, namely a central stump that has the form of the tower plan at level 1034, descending to level 1028 and ending at level 1031.5 m above see level. It is designed and constructed by use of concrete class CS20 and, on its upper side, it has a sufficient number of anchors for connection to a reinforced concrete plate. Designed over this stump is a foundation RC slab with thickness of 2.5 m over the stump and thickness of 2.0 m at the ends. For the most unfavorable loading conditions, maximum stress of 801 kN/m^2 < 1000 kN/m^2 (allowed compressive stresses in the rock mass) was obtained. The overturning coefficient of the tower was defined to amount to 3.20.

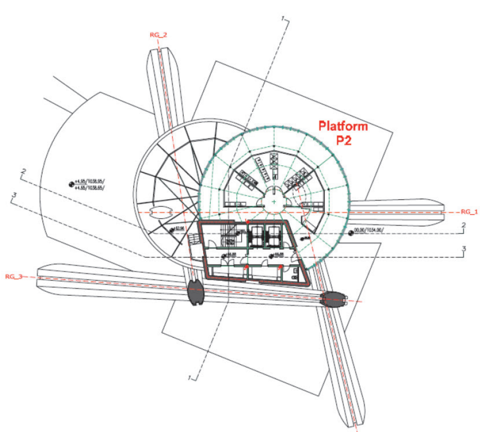

Figure 5. Cross-section at platform P2 (z = 66.88 m), platform below with position of the tendons

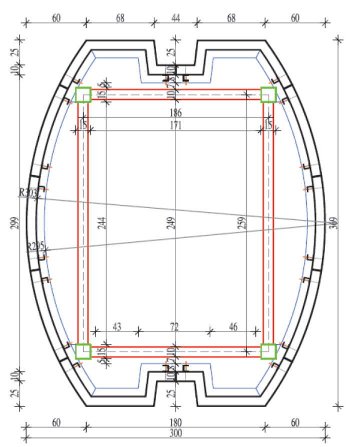
Figure 6. Cross-section of the horn

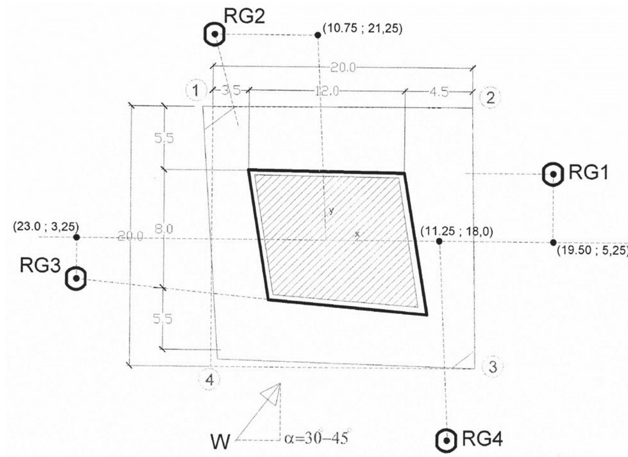

Figure 7. Top view of foundation structure

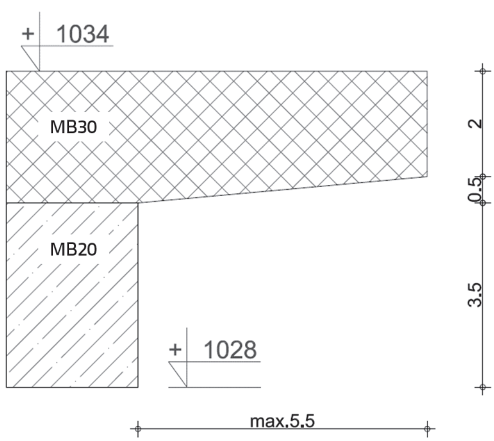

Figure 8. Cross-section of the foundation structure

Reinforced concrete shaft. The cross-section of the shaft at level 3.04 m is presented in Fig. 9. The figure shows that the external walls of this irregular quadrangle have a thickness of 55.0 cm, whereas the thickness of all internal walls is 20 cm.
All walls are designed to be constructed of cast-in-place concrete class CS40 MPa. During construction, the value of 45-50 MPa was achieved. The thickness of the external walls is variable, ranging from 55 cm at the beginning, decreasing to 40 cm at the last 8 levels. The thickness of the interior walls is not varied along height as is not varied the thickness of the floor slabs that constantly amounts to 15 cm. The total weight of the shaft is 86812 kN, i.e., approximately 3472 m^3 concrete mass. The reinforcement of the shaft is produced by SAH Stahlwerk Annahutte company in Austria. It is class 500/550 and its jointing for $\phi > 18$ mm is done by a coupling system produced by the same company, while the jointing of the $\phi < 18$ mm reinforcement is done by overlapping.

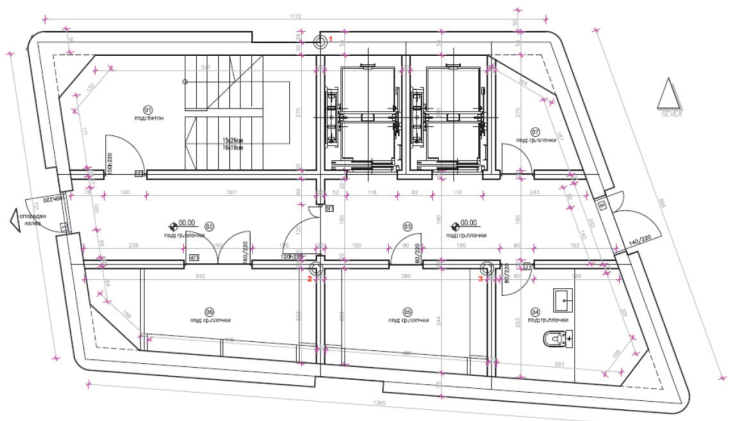

Figure 9. Shape of the shaft at the base

Working platforms. The first three working platforms are designed as cantilevers. Each consists of two rigid discs, one with a height of 1.0 m measured from the floor downwards, while the second is from the ceiling of the working platform 1.0 m upwards. The effective storey height is 3.70 m, i.e., the height of the platform from the lower façade to the roof is 5.70 m. The two discs are mutually connected and have identical structural solution that includes a small ring with a diameter of 3.0 m, a central circular truss and a façade frame. From the center of the small ring, radially in the lower and upper disc, 12 trusses with a height of 1.0 m are placed. Fig. 10 shows the top view of the platform structures. They are designed of steel class 355. Tendons are also designed for the platforms.

Figure 10. View of the platform structure from the top

Antenna truss. The antenna truss represents a steel 3D structure with a square cross-section and axial distance of the sides of 2.20 m. The storey height of the segments is 2.8 m and there are 12 segments over level 112.0 m, meaning that the effective height is 28.0 m. Additionally, two storey heights are anchored to the reinforced-concrete shaft, meaning that the total length of this truss is 33.6 m. It is designed of steel class 355.

Decorative elements – horns and mesh. The cross-section of the horns is variable along height and its shape is presented in Fig. 6. The bearing structure represents a quadrilateral 3D truss with added steel 3D structure to which the façade is mounted. Along length, the curvilinear horn structures are divided into rectilinear segments with a length of 1.75 m. Three segments form a fragment that is completely prepared on field and is mounted at the corresponding place by use of joints designed for the needs of this structure. All four horns are supported at four points on a reinforced concrete horn with a length of 7.0 m measured from the terrain and at four points on the shaft for each horn structure with bearings. For horn structures 2, 3 and 4, at half of the free height over the second support on the shaft, an additional horizontal truss is designed to connect the horn structure with the shaft. This additional connection prevents the occurrence of vortex shedding [6]. Finally, each horn is connected by a horizontal truss to the platform – core connection at the level of the lower façade. The façade of the horn structures is designed of hard styropore strengthened by polyurea with thickness of 3.0 mm from all sides. Fragments of the façade were quasi-statically tested in the IZIIS' laboratory, while the results are presented in the report cited under [8]. The decorative mesh is highly porous and is designed to be constructed of cables ϕ 100 mm. The mesh is suspended from a certain number of points on the horn structures.

4 Preliminary investigation

For the needs of the project, at the request of the design engineers involved in the design of the structure, surveys of the tower site were done for the purpose of definition of the geological and geomechanical characteristics of the soil [1], the geophysical characteristics [2] and study of the seismic hazard and seismic design parameters [3]. The values of the spectral acceleration for a period close to zero amount to PGA = 0.140 g and PGA = 0.228 g in horizontal direction, corresponding to return periods of 95 and 475 years, while for vertical direction, these amount to PGA = 0.136 g and PGA = 0.222 g for return periods of 95 and 475 years, respectively.
The tower height and the terrain relief on which the structure is being built are particularly sensitive to wind effects. For a return period of 50 years and an altitude exceeding 1000 meters, for a 10 min- mean value in accordance with the regulations [11], the main wind velocity of $V_{m,50,10}$ = 38.0 m/s was defined. This velocity acts upon the first 10 meters of the tower height and then it is increased depending on the dynamic factor and the turbulence. So, at level 112 meters, it reaches 53.33 m/s, whereas at level

140, it is 55.98 m/s. The complex geometry of the structure and its exposure to wind effects were sufficient arguments for the design engineers to request additional control of the aerodynamic coefficients defined according to the national standards [11]. The investor accepted the arguments of the design engineers and asked Prof. Hans-Jurgen Niemann from the University in Bochum, Germany, to perform a study for definition of the wind effect based on experimental testing of a model exposed to wind effect in an aerodynamic tunnel. The view of the model in the tunnel is shown in Fig. 11. The tests on the model enabled definition of the positive and negative wind pressure upon the shaft, the platforms and the horn structures. For all combinations of wind effect, these are presented in the report of 2014 [4]. In the course of the construction of the shaft, at the request of the operators – future users of platforms 1, 2 and 3, a change of the façade of these platforms was made leading to essential change of the aerodynamic coefficients for analysis of the platforms. Therefore, Prof. H. J. Niemann was asked to perform a new study for definition of the aerodynamic coefficients of the platforms that was finished in October 2020. The results are presented in report [5]. To check the safety of the connection of the main circular truss with the shaft, quasi-static tests on a fragment of two fields to the scale of 1:2 [7] was performed. Fig. 12 shows the characteristic force – deformation relationship for the tested model no. 03. The results of these experimental tests were applied in preparation of the workshop documentation on the steel structure. The dynamic behavior of the tower structure exposed to earthquake effect was confirmed by testing of a physical model on a seismic shaking table on which the earthquakes defined in study [3] were simulated with a great accuracy. The results from the shaking table testing are presented in [9]. To check the safety of the façade of the horn structures against wind effects, quasi-static tests on a façade panel to the scale of 1:1 were carried out. The results from the tests are shown in [8].

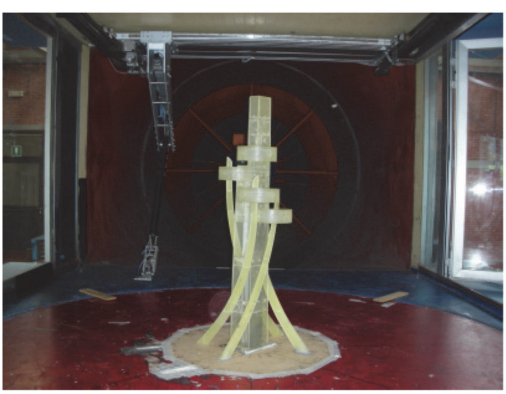

Figure 11. Wind tunnel model – overview

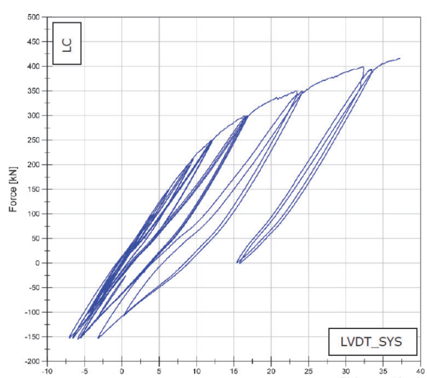

Figure 12. Force deformation relationship for tested model No 03(7)

5 Technological requirements of users of platforms

In the beginning, the design engineers involved in the design of the structure requested information from the potential users of the platforms about the technological requirements for proper functioning of their equipment. Each operator delivered information about the outline of the equipment, weight and operating conditions. None operator stated an acceleration criterion as an unfavorable criterion when their equipment is exposed to wind or earthquake effect. However, all operators requested that the vertical deformation of the platform is as little as possible, i.e., close to zero. For gravity loads, technological loads, snow and wind effect, when rotation of the core cross-section is excluded, this requirement is fulfilled, but when rotation of the core cross-section at the points of the platform – core connection is included, the maximum vertical deformation will be approximately 10 cm, which will cause disturbance of the functioning of part of the equipment. These deformations take place in the case of wind with a return period of 50 years.

6 Procedure of structural design of complex structures

Under the notion "complex structure", the authors of this paper mean a structure:
- whose architectonic geometry has a number of non-standard components that are integrated into a whole, which in this case, are of a quite complex architectural form [Fig. 1];
- whose structural system as a whole consists of more than one structural material;
- for which dominant effects on the integral structure and individual structural components cannot be a priori estimated as to which dynamic effect, earthquake or wind, it is more sensitive to;
- for which there are different criteria for definition of the serviceability level and criteria for acceptability of solutions.

Starting with the above stated arguments and the importance of the structure by which a higher level of quality of telecommunication services in the territory of the state is to be achieved, the design engineers of the tower structure proposed to the Investor of the structure a procedure of design consisting of traditional design based on the requirements contained in the national standards [11] and application of the corresponding Eurocodes [10] as an initial phase of elaboration of a working design of a higher quality. Based on the results from this phase, experimental investigations for correct definition of essential parameters were proposed for the purpose of objective prediction of the dynamic response of the structure. Following the performance of the investigations [4, 5, 7-9] and the obtaining of relevant data, the analysis of the structure was repeated by application of the obtained experimental data. The procedure of design of this complex structure can be presented in a simplified manner as design by application of valid

standards + experimental verification of vital parameters + redesign and re-analysis to obtain objective and optimal structural solution. It should be noted that this procedure is more time consuming and costlier compared to the traditional design, but the benefits in respect to safety of the structure and optimal solutions achieved with this procedure are greater.
Finally, what is of a particular interest for the professional public, is the proposal of the design engineers about installation of a health monitoring system upon completion of this structure for the purpose of monitoring its dynamic response to earthquake and wind effects at several vital points of the structure that was already accepted and entered into the budget by the investor.

7 Conclusions

Based on the results and the knowledge gained in the phase of elaboration of the project on the telecommunication tower structure in the vicinity of Skopje, the authors of this paper consider that elaboration of design documentation on complex structures requires the use of a non-standard procedure of design in which it is necessary to experimentally verify certain procedures. This non-standard procedure is more time consuming and costly, but the benefits regarding the safety of the structure and optimization of the costs of construction completely justify this procedure.

Acknowledgement

The design engineers involved in design of the structure extend their gratitude to the Investor – AEK (Agency for Electronic Communications) of the Republic of N. Macedonia for the entrusted task, cooperation and understanding during the project elaboration. They are also indebted to Granit AD Skopje – the main contractor for the construction of the tower and its subcontractor – FAKOM AD Skopje for the assistance in the realization of the experimental tests.

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