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Full-scale testing of steel lattice towers: requirements, preparation, execution, challenges, and the results

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ABSTRACT: This article encompasses a presentation of preparation works and full-scale experiments of three different steel lattice telecommunication towers. The emphasis was placed here on the way how these experiments were performed and particularly on the experimental site selection, foundation site preparation, equipment application as well as an impact of difficulties, which may have influenced the execution of the tests. The second part of the work includes some of the results obtained during the test such as displacements of particular points of the structure as a function of the external load, the strain measurements, the failure modes as well as the large global deformations. The gathered data remarkably enrich the knowledge on full-scale testing of large-scale skeletal steel structures.

Keywords: full-scale testing, steel lattice towers, elastic buckling, failure mechanism

1. INTRODUCTION

Telecommunication structures have been becoming landscape features in both urban and rural environments. Recent developments in data transmission technology, ever increasing customers' requirements, and technological level required in the cutting-edge telecommunication equipment (in the form of antennas, radio modules and etc.) cause constant increase of requirements for this type of technical infrastructure. The impact of technology advancement in the field of telecommunication devices apparently leads to a need of the modern, efficient and economical design process for the skeletal transmission structures as towers and masts.

The research & development works regarding telecommunication object are focused on the few areas: optimization of production, manufacturing and prefabrication of structures, modification of analytical descriptions concerning structural behavior, elaboration of the complex numerical analyses as well as experimental studies realized on models in laboratory conditions, or - what is particularly important - in the full-scale mode. A variety of the problems in design and analyses of the telecommunication structures were discussed by Travanca et al. [8]. An essential problem concerning a modern engineering practice is a comparison of different standards definitions, which have an undisputable influence on bearing capacity estimation. One of the broadest descriptions of difficulties connected with steel towers and masts can be found in [5, 6] – some structural solutions for different types of loads have been presented and some examples of damage and failures of structures were given. The elaboration on failures resulting from the wind load for slender, lightweight high steel structures was presented by Repetto and Solari [4]. The very valuable reference in the context of an amount and intensity of the wind load would be this written by Carril Jr. et al. [2]. They discussed an influence of the wind action on the steel lattice towers having square cross-sections. The data obtained during the tests in wind tunnel were compared with these obtained analytically. On the other hand, the ice load aspects were discussed by Makkonen et al. [3].

It is known that modification or extension of the telecommunication equipment attached to the structure needs its strengthening in some cases; some examples are given in [1, 9].

The experiments made with the full-scale structures are the great source of the knowledge regarding especially structural behavior close to and under failure load. It is especially important for the large-scale engineering structures, where numerous geometrical imperfections and materials nonlinearities appear. Experimental data generated by tests, as opposed to computer simulations and calculations, can find use in the process of correctness verification or calibration for numerical models.

This article is entirely devoted to a description of the experimental tests performed, particularly to the testing preparation works, execution difficulties, equipment used, and the way the experiments were carried out. In the second part some experimental results were presented. The extreme displacements, overall deformations of the towers legs and failure modes are discussed in details.

2. EXPERIMENTAL SITE

An initial assumed concept of the experiment was the failure test of the few tower structures, which were to be later disassembled. The test was to be carried out at their original locations, but some disadvantages including enormously large costs did not allow for such an activity.
One of the problems was the presence of transmission lines as well as building developments in the closest neighborhood (Figs 1-2). The insufficient free space around the objects posed direct threat to the citizens. The probability of unexpected damages rendered the research experiments impossible. The unfavorable location of the objects did not allow for external load to be utilized to the full extent either: usage of the heavy equipment generating the failure load required hard terrain and an empty space that would provide sufficiently large distance from the tested tower.
The basic criteria affecting a selection process of the device used for the external load simulation were the following: significant value of the effective concentrated load (range of about 20 tones), the possibility to utilize a single line, which would be pulled during the test, and also a large distance in-between the towing machine and this structure (more than 100 meters). It was essential because, as the vehicle was slowly moving away from the tower, the angle between the applied force and the horizontal plane decreased, therefore this distance would allow for a more accurate reflection of the horizontal type of the loading (wind).

Fig. 1 Unfavorable location of the tower structure: close neighborhood of the buildings

Fig. 2 Unfavorable location of the tower structure: close neighborhood of the medium voltage transmission lines

The force simulating the effective external load was attached to the tower through the pulled steel line and the additional steel diaphragm welded directly to the structure, which required additional welding works carried out at the level of its final attachment.
Taking the aforementioned problems into consideration, the final decision was made to perform the research efforts at the specially accommodated place (documented by Fig. 3), which fulfilled all the requirements concerning geometric parameters and area, where the highest security standards were met as well. Such a solution had to include the costs resulting from the plot lease and disassembly of the structure, its transportation and further installation. The additional expenses resulted from the foundation erection, which had to provide, apart from proper anchoring for the towers, a possibility to place the few different structures with different geometries (one by one, including their final failures), namely a triangular cross-section as well as a square one. The discussed additional activities included renting the heavy equipment, the cranes or the excavators, and also hiring the crew of professional assemblers of steel structures.

Fig. 3. Aerial view of the experimental site

The final scheme of the experiment is presented in Fig. 4 below. We can see that a distance in-between the tower and the towing truck reached more than 120 meters.

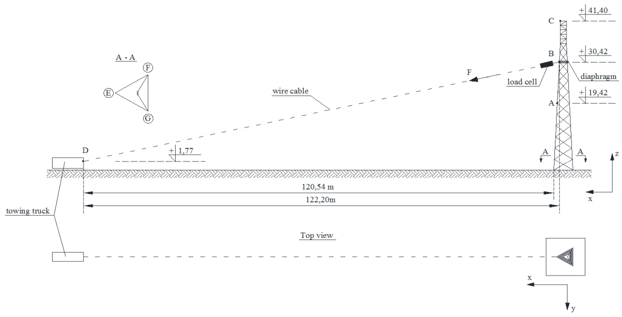

Fig. 4. Scheme of the experiment

3. ADDITIONAL TESTING EQUIPMENT

The additional equipment was needed in order to simulate external load in the experiment. It had to provide, apart from exerting optimal load value on the structure, means of the measurements. An additional requisite was a sufficient control of the tension of the steel cable so that the applied force has been increased incrementally to allow for the geodetic measurements of displacements of this structure's joints.

Fig. 5 Towing truck used in the experiments

A towing truck appeared to be an appropriate experimental device for this purpose (Fig. 5), which fulfilled the aforementioned criteria, had enough mass and could be anchored into the terrain, which guaranteed its stability against uncontrolled variations of normal stresses in the line. A special steel diaphragm was designed in order to allow for an appropriate transfer of the load to the entire cross-section rather than to a single joint or member (Figs. 6-7). The attachment location was selected to avoid the damage of the cantilever part of the tower (top section with the parallel legs) and to generate extreme forces in lower sections legs at the same time. The tower has been installed just before the experiments; therefore the welding of the diaphragm could be done before the structure got vertically fixed in the foundation, which simplified further the works.

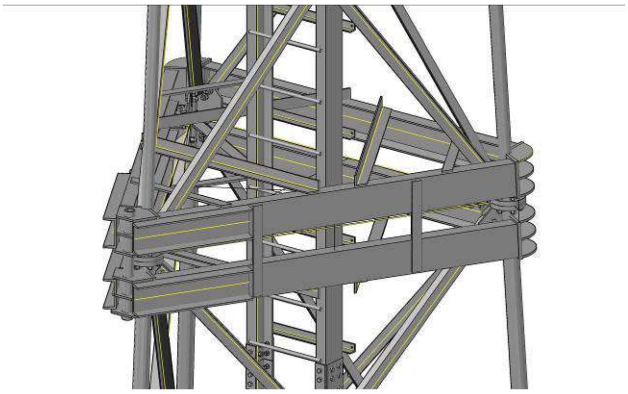

Fig. 6 General idea of the steel diaphragm for the tower with triangular cross section

Fig. 7 Steel diaphragm during the experiment

An essential preparation phase worth discussing was an assurance of the load transfer to avoid accidental twisting of the entire structure resulting in its deplanation. Therefore, the line should be set along the direction "y" (Fig. 4). It was realized by geodetic determination of the hoisting winch location and an appropriate and the very precise steel diaphragm structure.

4. FOUNDATIONS

Stability of the entire structure had to be guaranteed during these experiments: it could neither move nor rotate under the simulated external force and it should have been also properly anchored in the terrain. The foundation had been designed in a specific manner, so that three structures of corresponding three different leg layouts could be attached (the first and second towers cross-sections were triangular and the third was square). In order to minimize the expenses and material use, it was decided that the typical tower structures foundations: a monolithic concrete pad footing under each of the tower legs were not used. The slab foundation consisting of widely available concrete plates was constructed instead.
The foundations structure consisted of the steel frame, which was supported on concrete slabs for more uniform distribution of the stresses (Fig. 8). They were placed on a layer of compressed sand of 50 cm.

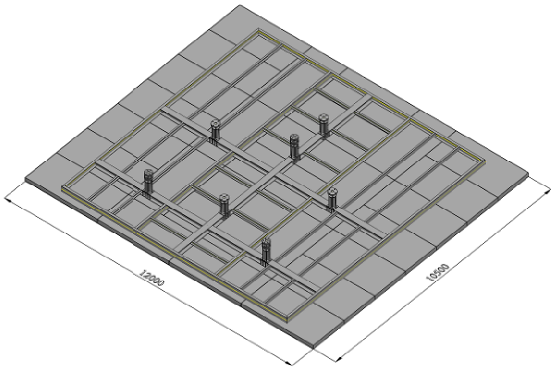

Fig. 8 Steel frame in the foundation base

The foundation anchors were attached to the steel frame in both geometric leg layouts corresponding to the tower types; this foundation was subsequently uploaded with concrete slabs. The structure was placed in foundation trench of about 1.0 m depth in order to protect against moving. The concrete slabs shown in Fig. 9 were also inserted between the walls of the slope to ensure the additional stabilization of this structure against possible rotation.
The elevation of foundations occurred during the testing of the second tower, which had the same geometry but larger element cross-sections than the first one - it took place for the total weight of about 110 tones. It is important to the future tests that such a mass was not sufficient to ensure perfect stability of the foundation itself. Therefore, the experiment was repeated after additional load was added to foundations, which resulted in the final weight of 170 tons.

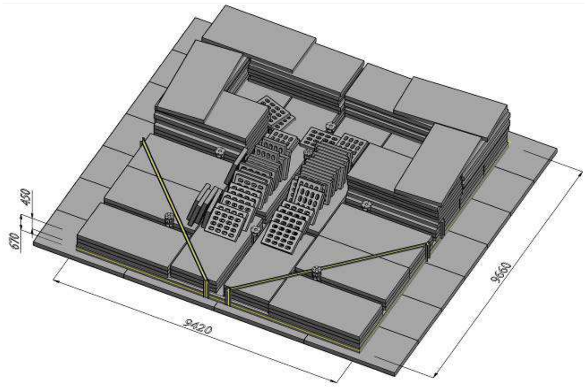

Fig. 9 General idea of the concrete plates foundation

Fig. 10 Final foundation with asymmetric mass distribution

5. DESCRIPTION OF THE TOWERS GEOMETRY

The experiments were performed for two towers with triangular cross sections and heights equal to 40 meters each. The basic structural concept is the same for both towers; the differences were noticed for the cross-sections of particular members only. The detailed description of one of the tested towers is presented below. This tower has been manufactured as the three-dimensional steel truss of a triangular cross-section and height of 40.0 meters divided into seven separate sections. Its upper part is of a triangular cross-section and the bottom part (up to 34th meter) forms a pyramid frustum with a constant 5% convergence.

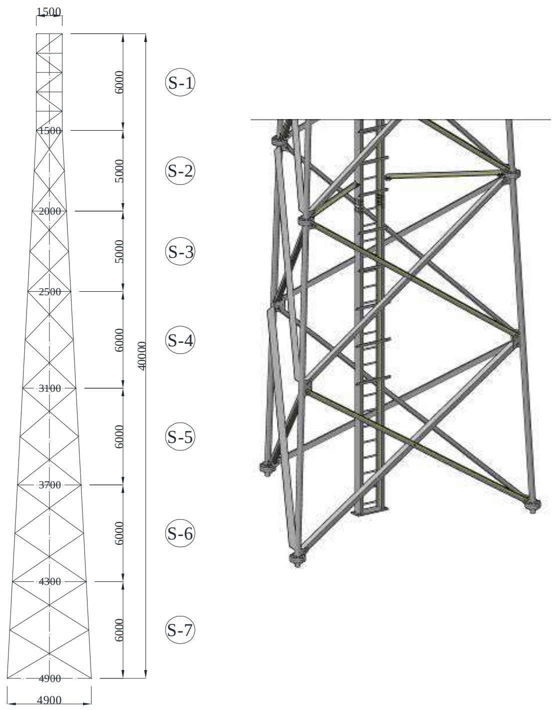

Fig. 11 Scheme of the structure (left), and view of section S-7 (right)

The centerline dimension is 4.90 m at its base and 1.50 m at its top. The upper part of the tower is a parallelepiped of a height equal to 6.0 m with the cross-section of an equilateral triangle of side length equal to 1.50 m. The segmentation of the tower, the heights, the names of particular sections and the 3D view of bottom section S-7 with a climbing-cable ladder is shown in Fig. 11 above.

The leg members in each section consist of the round solid bars, while the bracing elements are hot-rolled symmetrical and non-symmetrical angle sections. The diagonal bracing system of this tower is of type X. The bracing elements are continuous in this structure and the joints at their intersections are made with a spacer and just a single bolt. Their connections with diagonal bracing of the lattice were manufactured with gusset plates and bolts, two in each node. The profiles of particular elements of the tower are presented in Tab. 1 below.

Tab. 1. Selected tower element profiles; dimensions in mm

Section	Legs	Diagonal bracings
S-1	Ø 65	∟60x60x5
S-2	Ø 65	∟60x60x5
S-3	Ø 80	∟60x60x6
S-4	Ø 80	∟90x60x8
S-5	Ø 90	∟90x60x8 ∟100x75x8
S-6	Ø 90	∟100x75x8
S-7	Ø 100	∟120x80x8

6. GEOMETRICAL IMPERFECTIONS

One of the more important activities before the experimental test in full-scale manner is a correct identification and measurement of the geometric imperfections. It is particularly important in case of the steel structures, where geometric nonlinearities cannot be avoided and where precision plays significant role.

These geometrical imperfections had been measured in case of the second tested tower for each structural element, prior to the final installation of the structure. The measurements were not performed for the climbing-cable ladder, because these imperfections were assumed as uninfluential in this particular experiment. The ladder was added to the tower body to reflect the reality as accurately as possible, although being not very important for the overall structural response. The diagonal bracing members made of hot-rolled L-sections were produced as the independent elements without any welded gusset plates, brackets, etc.; therefore no bulging, bending or other deformations were noticed in case of these elements. The legs had the gusset plates welded at centers of section spans (10 mm thick rectangular plates located asymmetrically in addition to the longitudinal leg axis), where the greatest imperfections were observed. They had been created during the manufacturing process (residual and welding stresses) and take the form of imperfections in direction perpendicular to the welded gusset plates as shown in Fig. 12.

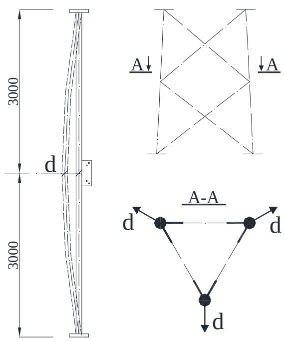

Fig. 12. Geometrical imperfections of the tower legs in S-6 and S-7

These imperfection values labeled in Fig. 11 as *d* oscillated between 8 and 15 mm. The measurements of a curvature were taken after the horizontal tower assembly. A line connecting section flanges was taken as the reference base for measurements of the leg imperfections.

7. DISPLACEMENT MEASUREMENTS

There is no doubt that the most significant structural response in this pushover test was the horizontal displacement of the structure under external load. A measurement of this displacements was taken with total station tachymeter and leveling instrument. Geodetic displacement measurements of the structure were taken at points A, B, C on the compressed leg and the points E, F, G in the supporting nodes (see Fig. 4 and 13).

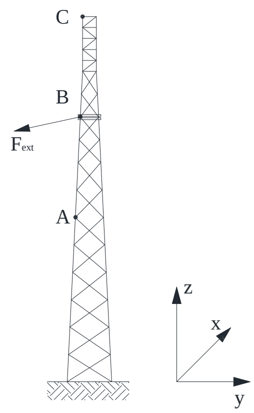

Fig. 13 Points A, B, C and the coordinates system orientation

The main goal of these measurements was an observation of the displacements as a function of the external load applied as well as a verification of nonlinear fluctuations of these displacements when approaching to the failure including the geometrical imperfections. The measurements were taken for two settings of lunette of the geodetic instruments. The force in the line decreased every time when the hoisting winch was stopped (to take measurements, every 10-15 kN) because of apparently elastic character of the steel line behavior under the given load. Thus, the final measurements presented here are the arithmetic mean of the two different values of displacements and the force in the line read with about 1 kN accuracy. The absolute value of the total deflection of the top for the second tower was about 475 mm. The displacements graphs for directions *x* and *z* versus the force applied to the structure are presented in Fig. 14.

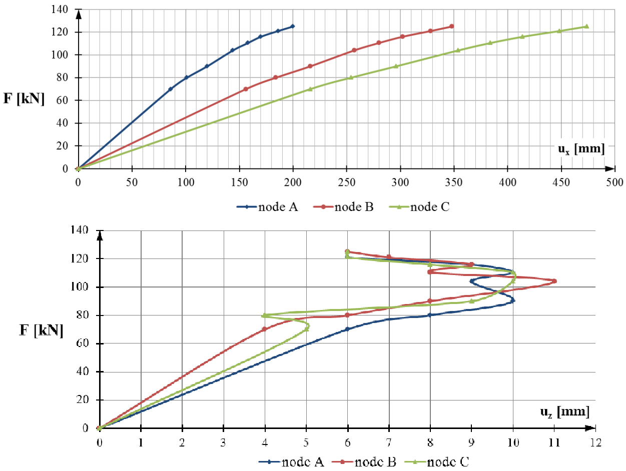

Fig. 14 Displacement of nodes A, B and C (second tested tower)

It is seen that after reaching the external load value of 90 kN, all the values started to decrease in *z* direction (the points moved downwards), while all these values increased systematically in *x* direction. It was therefore revealed that, in agreement with stability theory for large systems with geometrical imperfections, the displacements of points rapidly increased when approaching the value of the failure load. It should be noted that there were also vertical displacements of the supporting nodes under the given external load. The values noticed for the compressed leg seem to be of the paramount importance. The results of vertical displacements for the supporting nodes E, F, G *versus* the external force for the second tower are given in Tab. 2 below.

Tab. 2. Vertical displacements of the supporting nodes

Force in the line	Vertical displacement (Z axis) [mm]		
[kN]	Node E	Node F	Node G
0	0	0	0
70	-6	5	3
80	-8	6	3
90	-10	7	4
104	-11	9	6
115	-13	10	7
116	-14	11	8
121	-14	12	9
125	-15	14	10
after the experiment	-36	18	18

From the perspective of a global behavior of this structure under failure load, the differences in the vertical displacements before and after the destruction were relevant. It is important to mention that the permanent deformations in the supporting frame were relatively large, in particular below the compressed leg.

8. STRAINS MEASUREMENTS

Strains measurements within the selected structural members of the towers (compressed legs in particular) were the next focus during the execution of the tests. The results presented in this subsection were taken directly from the experiment on the second tower. The strain measurements were made with the electric resistance strain gauges. Every measuring point had four independent gauges placed on the opposite sides of the cross section and the additional results have been presented in Fig. 15. These strain readings corresponding to parallel measurements of the force in the pulled line allowed for

- normal stresses identification and axial forces identification when the cross-section of an element is known,
- determination if particular elements are under compression or under tension,
- validation of the results for the overall capacity obtained using standards definitions.

The strains of the particular tower members are presented in a graphical form as a function of the external load (force in the line). Fig. 15 presents these strains values for three measurements points located in the compressed leg of the tower in section S-7. The graphs correspond to the setting of the element:

- the bottom graph corresponds to measurement point at the bottom of the element, just above the supporting joint,
- the middle graph corresponds to the measurement point at the center of the element just under welded gusset plates,
- the top graph corresponds to the measurement point placed under the joint between the leg of the sections S-7 and S-6.

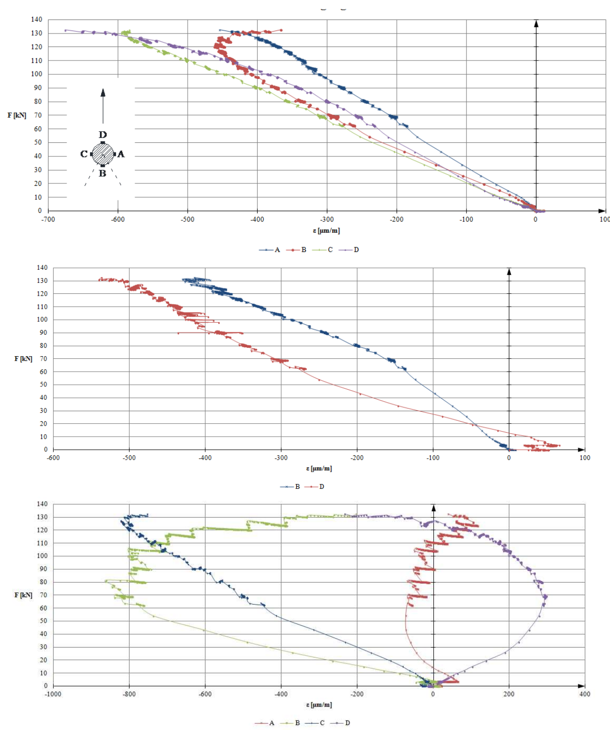

Fig. 15 Strains of the deformed leg of the tower

It can be stated that the distribution of stresses just under the supporting joint indicates that the element was being bended at this spot, which had been caused by its attachment to the anchors. The two remaining strain rates indicate clearly compression of this element. The maximum value of the strains in the leg amounted to -676.76 µm/m. This value was reached for the force in line of 132.5 kN. The value above corresponds to maximum stresses that leg members in this section (solid round bar Ø 100) can transfer. Yielding of this cross-section takes place leading to buckling of the leg immediately above this limit.

9. FAILURE MODES

Due to the fact that the towers analyzed had different geometry, the cross sections of the legs, and also the diagonal bracing members, the final stability loss occurred at different heights and different sections. Exemplary failure mode of the second tested tower is depicted in Fig.16. The attached pictures show that the main extreme deformations occurred in the compressed leg at the section S-7 in case of the second tested tower.

Fig.16 Buckling of the tower leg in section S-7 (second tower)

The stability loss occurred in the section S-5 in the first case. For all cases, the buckling of the legs took place almost perpendicularly to the loading force. Plastic hinges occurred at the centers of the bracing panels, at 1/3 and 2/3 of section span. It should be noticed that the joints connecting the neighboring legs remained rigid due to significant thickness of the connecting flanges, and no so-called leverage effect was present. Taking damage schemes of the particular researched structures into consideration, it can be stated that the reliability of that structures is based on carrying capacity of the legs. The strength test of the real full-scale structures, whose particular elements are in real scale allows for authentic measurements of the large deformations. The advantage of those kinds of tests over the tests on laboratory models, which do not have real scale, is particularly visible in this case.

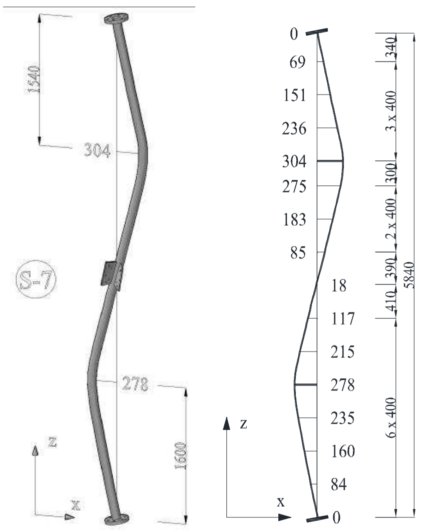

Fig. 17 Large deformation measurements
in the most deformed leg for the second tested tower

The measurements of the deformed element of the second tower destroyed leg are presented in a graphical form in Fig. 17. The data may serve the adaptation of FEM models to failure analyses or analytical behavior descriptions when such elements are under the failure load.

10. CONCLUSIONS

The article contains the basic information gathered during the full-scale tests of steel telecommunication skeletal towers. A particular attention was devoted in this work to

- the difficulties in finding an appropriate test locations, which would meet all the experimental requirements,
- a description of the fundamental equipment and the auxiliary structures employed to perform these tests.

The above information considerably enriches the existing knowledge concerning the full-scale testing of high skeletal steel structures. Taking into account the fact that a scientific literature contains only the few examples of such a full-scale structure research, this article may serve as the concise guidelines for researchers, practicing engineers as well as telecommunication companies employers who want to carry out similar experiments on their own account in the future. It is seen in particular that the post-industrial urban locations meet perfectly the needs of such full-scale experiments.

It can be also stated that the very expensive full-scale tests allow for obtaining the results that are unavailable in any other way. The results concerning displacements included in the article allow for stating that the serviceability limit states (the displacements measured at the top of the tower were equal to 0.475 m) would be practically reached before the loss of bearing capacity in the section S-7 leg occurs. The failure modes analysis supported the view that standard provisions are not accurately described and defined, which had been already proposed in some earlier work [7]. These results will serve for further both analytical and computational studies concerning quasi-static inelastic failure analysis of such structures to validate some numerical models applied frequently in steel structures designing processes.

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