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Basic Design Considerations for the Moscow 533 Metre T.V. Tower

Considérations principales pour le projet de la tour de télévision à Moscou, 533 m haute

Grundsätzliche Berechnungsprinzipien für die Projektierung des 533 m hohen Fernsehturmes in Moskau

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Construction of a new 533 m TV tower was terminated at the end of 1967 (Fig.1).

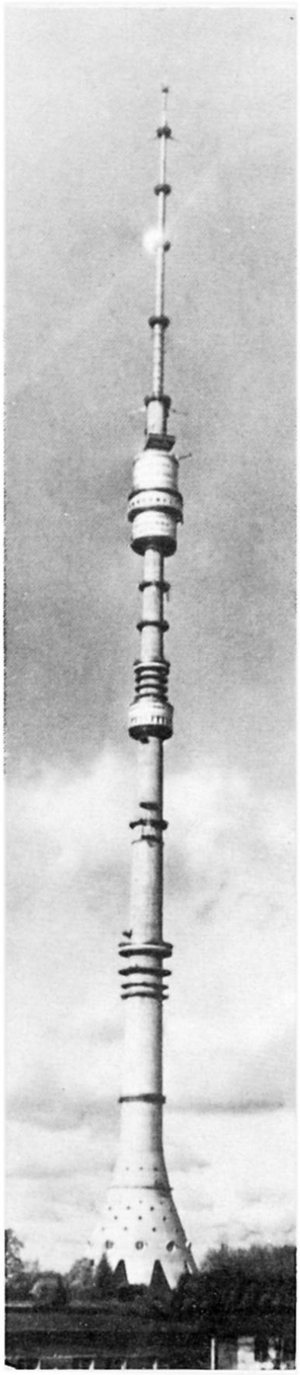
Up to the level of 385 m the tower is a cone prestressed reinforced concrete tube with a sharp break in its generating line at the height of 63 m. The lower part of the cone rests on foundation containing ten separate supports which form ten open arches 17 m high each.

Beginning from the height level 385 m to 533 m (148 m) a steel telescopic pipe shaft is situated which carries radio aerials as well as TV antennas.

Within the cone and the shaft of the tower 48 floors of various premises are situated, floors N°, 5, 6, 7 are occupied by the radio transmitting station. Partially the radio transmitters are located in the upper part on the level of 348 and 353 m. Simultaneous transmission of 5 television programs is secured (one of them is in technicolour) and 6 radio programs on ultrashort wave lengths. The height of the antenna guarantees secure reception of TV and radio broadcasting programs at a distance of 120-150 km.

Three floors at the height levels 328, 331 and 334 m are taken by the restaurant. Simultaneously the restaurant can serve 288 persons. The tables are placed on the circular revolving floor.

At the height levels of 147, 269, 337 and 341 m the observation towers are located. Three of them are closed and the upper one is open (Fig.2).



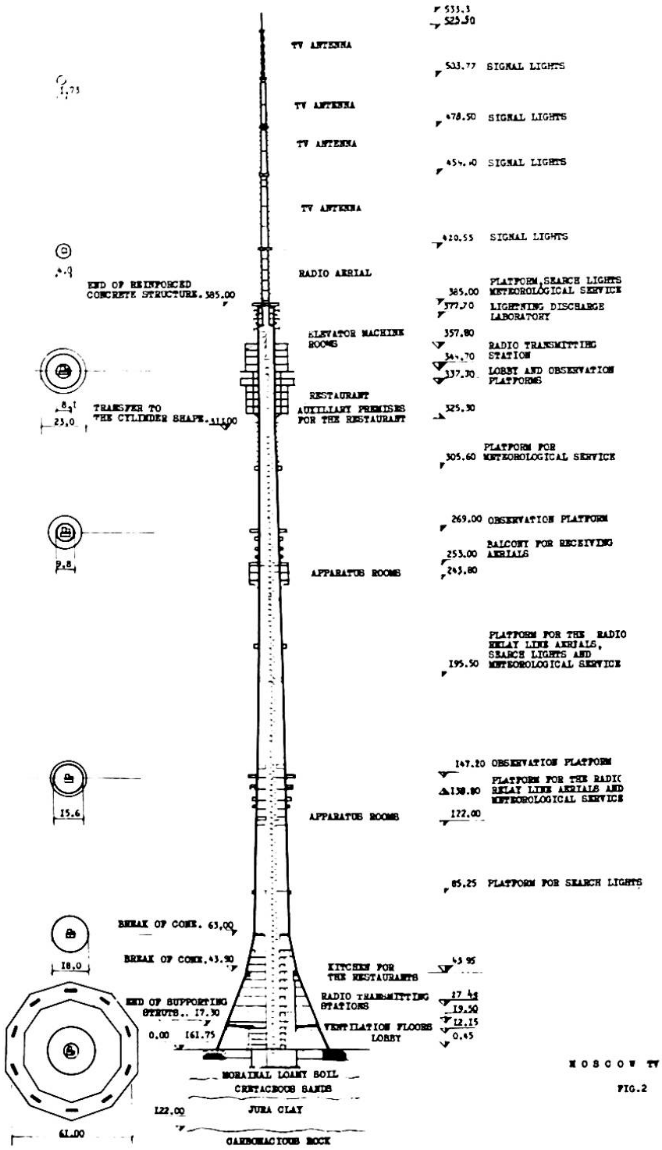
In the shaft there are 4 elevators. One of them serves the restaurant. The rise and descent takes 150 sec. at a speed of 7 m/sec.

In the shaft proper and around it on the platforms numerous technical services are located such as a receiving station for subsequent retranslation of the TV programs, meteorological apparatus, signal lights, laboratory for lightning discharges and radio relay line antennas. The shaft of the tower on top of this contains all kinds of communication, wiring for radio, TV transmissions, electric conduits, water supply pipes, sewerage and telephone.

The architectural outlook of the tower considerably is determined by the technical requirements. The lower part has large openings and being left free reveals the structural design. On this elevation the cross-section of the shell is represented. The concrete surface of the tower is left unpainted in order to better reveal the type of material used. The light aluminium constructions contrast well with rough concrete shell. The interior decoration contains modern materials: aluminium, glass, plastics.

The Basis and Foundation

On the building site the subsoil conditions are the following from top to bottom: 10-12 m of very dense loamy soil of the glacial period containing pebbles and bolders, water is at a depth of 5-6 m, further down 12-15 m more ancient deposite in form of fine dusty sands and sandy soil. Further down old dense clay and only at a depth of 40 m - rock. Under the above-mentioned circumstances it was decided to rest the tower on the upper loams with the minimum depth of foundation in order to leave as much as possible the layer of good soils between the lower part of the foundation and relatively weak lower sandy soils saturated with water.



MOSCOW TV TOWER

FIG.2

The foundation of the tower has the form of 10-sided polygonal ring slab with an average diametre - 60 m, width 9.5m and the thickness 3 - 5.5 m.

When testing the soils by the method of loading the punch with the area of 600 cm^2 in the pits we have received the following values of the modules of deformation: morainal soil 800-900 kg/cm^2, underlying sand soils, loamy soils 300-400 kg per sq cm and the lower jura deposits - 300 kg per sq cm. The sedimentations of the foundations were determined at a usual supposition that the stresses under the foundation are being distributed as in a elastic homogeneous half shere 5 - 6 cm, what at subsequent observations was found to be true. Such sedimentation has no importance as far as the construction is concerned. The statical design of the foundation was carried out assuming the design scheme to be a ring continuous 10 span beam with a hinged support with a given load as the reaction of lower soils with the flat distribution of the load.

As a principal scheme for designing such a beam it is convenient to adopt a 10-sided polygon with the hinges in the middle of each side (Fig.3).

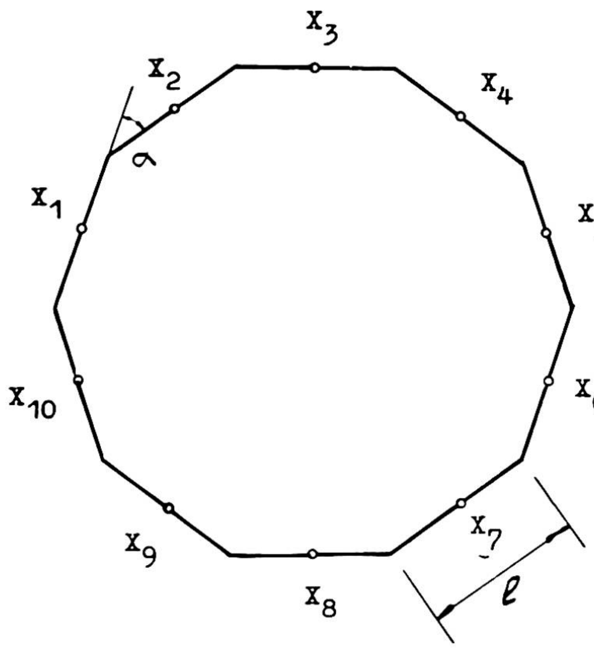


FIG.3

The hinges allow free rotation around the axis of the rod, that is they exclude the torsion moments but do not allow any shear or rotation around the axis perpendicular to the axis of the rod. In other words the hinge would allow the bending moments and the shearing stresses to appear. Under this assumption the design is reduced to solution of a system of five-member equations.

$$-x_1-2\cos\alpha\, x_2+(4+2\cos^2\alpha+f)x_3-2\cos\alpha\, x_4-x_5+$$
$$+a_{3p}+a_{35}+a_{3m}=0$$

where x are unknown torsion moments in the hinges

$$f=\frac{6\sin^2\alpha\, EJ}{GJ_k}$$

where E and G are modules of elasticity and shear of the concrete; J and J_k – bending and torsion moments of inertia; $a_{35}a_{3p}a_{3m}$ – the load members from the vertical loads, from the junction loads and from the uniformly distributed torsion moments.

The total weight of the tower is 31400 tons, the foundation 14500 tons soil on the foundation 5500 tons. All these loads produce stresses under the foundation equal to 2.64 kg/cm^2. The side stresses from wind loads are equal to 0.42 kg/cm^2.

When designing the foundation the necessary measures were taken to increase its durability and safety. The concrete used had according to our standards strength N^o 400, the main and cross reinforcement was increased. The protecting layer was increased to 10 cm. Along the perimetre of the foundation the reinforcement was prestressed and was formed by 10 strends according to the number of sides of the foundation polygon. Every strend has 104 wire cables containing each 24 wires 5mm in diametre. This reinforcement was prestressed by means of jacks each one developing the stress of 57 tons in the cable. Thus, in the foundation a prestressing force was created by compression equal to 5900 tons. This prestressing permitted to reduce the tension in concrete to such a value under which one does not have to expect appearance of cracks.

Supports, the Cone and Shaft

In the cone there are 10 round openings 4 m in diametre each as well as a considerable number of other smaller ones. Over the openings special reinforced concrete awnings are provided to protect from possible fall of icicles.

At the height level 63 m a very powerful diaphragme was introduced, further on up the shaft of the tower is a cone shell with the slant of the generating line equal to 2% compared with a vertical line. The diametre of the shaft is being reduced from 18 m at a height level 63 m to 8 m at the level 311 m, further up the shaft has a cylindrical form. The thickness of the shell of the shaft within the entire cone part is constant and equal to 40 cm. In the cylindrical part it is 35 cm.

The aerial consists of 5 cylindrical sections having the following diametres: 4.0 m; 3.0 m; 2.6 m; 1.72 m; 0.72 m with the lengthes from 19 to 36 m. The sixth section 8 m long has a square cross section 16x16 cm. The sections of the aerial have the thickness from 30 to 12 mm. Inside the aerial are located the feeders from the radio transmitting sets and a lift designed for one person. The last stop of the lift is at a height of 470 m. At the sections where the cross section of the aerial is changing the ring platforms are located. To these balconies special suspended platforms are affixed. By means of these suspended platforms the possibility to reach any external point of the aerial surface is secured. The aerial is protected from corrosion by means of galvanized zinc layer or plastic materials.

The total weight of the aerial is 360 tons.

The Strends for the Prestressing of the Structure

In order to increase the rigidity and to avoid appearance of cracks the shaft of the tower was specially prestressed to create compression in the concrete in the vertical direction. This was provided by means of a series of strends prestressed parallel to the inner surface of the shaft. Altogether there were 150 strends stressed. Thirty strends are fixed at a height of 63 m and 120 strends at a height of 43 m. At the upper part these strends are gradually fixed at 7 different horizons beginning from the height of 195 m and to the upper part only 60 strends are reaching.These strends are embedded into the ring cantilevers. The strend having a diametre of 38 mm consists of 259 wires 1.8 mm in diametre each. The wires have high quality zinc coating.

Each strend is prestressed with a force equal to 72 tons. The total stress of these strends in the lower part of the shaft reaches 10800 tons.

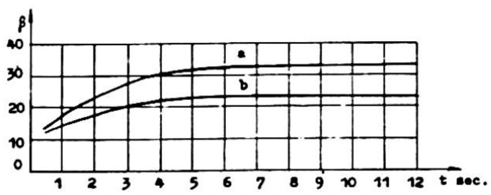
After the final stressing the strends were brought close to the walls of the shaft and affixed at the intervals of 7 m to the wall of the shaft. This is important since the affixed strends work as reinforcement and the strends not affixed to the walls keep their normal stress unchanged. This fact helps to gain around 10% in the safety factor.

Wind loads

When designing this tower the wind pressure was taken into consideration according to the usual norms for similar structures increased by 8%. The following wind velocities were considered:

Height above the ground level	10	20	40	100	350 and higher
Velocity in m/sec	24.7	28.7	33.1	36.6	42.7
m	0.35	0.35	0.32	0.21	0.10

Besides the statical wind load which corresponds to the above given velocities a frontal dynamic wind pressure is taken into consideration. The dynamic wind pressure is being evaluated through the coefficient β added to the statical pressure. By means of this coefficient the distribution of the dynamic part of the wind pressure along the height is calculated. The type of change of load pressure in terms of time is evaluated through a dynamic coefficient which is a function of the period of oscillation of the structure and its material. Having two different materials such as: reinforced concrete shaft and steel aerial the dynamic co-



a- for metal and timber structures
b- for reinforced concrete structures

FIG.4

efficient is determined by means of interpolation-proportional to the quantity of the potential energy stored during the oscillations of the steel and reinforced concrete parts of the structure (Fig.4ab).

$$Q_m = \int \frac{M^2}{2EJ_c} \, dx$$

Aerodynamic coefficient for the cylinder part is assumed to be equal to 0.6, for all sorts of protruding parts - 1.00. At that the wind load was assumed acting on all protruding parts along the entire perimetre.

Besides longitudinal oscillations in the direction of air pressure the resonance transversal oscillations were also considered which occur through periodical cessations of wind vortexes.

The critical velocity of air which leads to a resonance of the vortex cessation and proper oscillations of the structure can be determined by the formula:

$$V = \frac{5D}{t}$$

where D is the diametre of the tower

t - period of oscillation

The amplitude of the transversal aerodynamic force is determined by the formula

$$F = \frac{V^2 D}{80}$$

When determining the full resonance amplitude the damping of oscillations on account of friction is taken into consideration. The logarithmic decrement is assumed: for steel structures 0.10 and for the reinforced concrete structures 0.30. The mixed structure is being calculated through the interpolation similar to the design of frontal oscillations.

In connection with the very large period of fluctuation of transversal oscillation for the tower the latter ones were found to be considerably less than frontal oscillations although the resonance was considered to appear along the entire height of the tower.

The Dynamic Design

When making the dynamic design the tower was modeled in the form of a cantilever rod with the 24 concentrated loads (masses) and 24 elastic hinges in the same points where the concentrated loads were applied. By means of consecutive approach method the forms of free oscillations were determined together with the amplitude of acceleration at the top level of the tower which was equal to the acceleration of the gravity force further called unity oscillations.

In this case the inertia forces at the time of maximum deviations are determined by the formula:

$$T = G \frac{Y}{f} = G\lambda$$

where G is concentrated mass; Y - deviation of the mass;
f - deflection of the upper point of the rod.

Having assumed in the first approximation the diagram of deflections in form of a parabola the inertia forces T are determined and on the basis of these diagrams we determine the diagram of deflections. By means of the following procedure we gradually approach the diagram of deflections. After the fourth attempt we have come to a satisfactory precision.

After the unit form of oscillation has been determined the period of fluctuations can be determined by the formula:

$$t = \sqrt{\frac{4\pi^2 f}{g}} = 0.2 \sqrt{f}$$

where f is in cm.

When determining the second harmonic the basic curve was reduced to orthogonal position with the first one.

$$Y_1^{I} = Y_0^{I} - a\lambda^{II}$$

where

$$a = \frac{\sum T^{I} Y_0^{II}}{\sum T_1^{II} \lambda^{I}}$$

When determining the third harmonic the original curve was reduced to the orthogonal position with the two first ones

$$Y_1^{III} = Y_0^{III} - a_1\lambda^{I} - a_2\lambda^{II}$$

where

$$a_1 = \frac{\sum T^I Y_o^{III}}{\sum T^I \lambda^I} \; ; \quad a_2 = \frac{\sum T^{II} Y_o^{III}}{\sum T^{II} \lambda^{II}}$$

The amplitude of oscillations and the accompanying stresses can be determined by the formula:

$$K^n = \eta \beta k^n$$

where K^n is deviations and stresses according to harmonic n-power, k^n - the same deviations and stresses in the singular harmonic; β - the mentioned above dynamic coefficient; η - the influence coefficient

$$\eta^n = \frac{\sum P^n \lambda^n}{\sum T \lambda^n}$$

where P^n - the load applied at the point n; λ^n - relative deviation at the singular harmonic n.

Summing up of all the amplitudes of all harmonics is done by means of the square root:

$$K = \sqrt{(K^I)^2 + (K^{II})^2 + (K^{III})^2}$$

The Static Design

The static design was carried out on the basis of deformed scheme, i.e., all bending moments from the vertical loads were taken into consideration, bending moments which appear on account of deflection of the shaft. The deflection of the shaft taken into consideration gives us increase of the bending moments up to 10-15%.

Since the design was carried out on the basis of the deformed scheme separate design for stability was not performed.

A special design for the season fluctuations of the temperature was performed. It was assumed that the foundation has permanent temperature but the cone and supports can be heated up to + 30°(C) and cooled up to -30°(C). This design led us to the necessity to make the supports of the tower flexible in the radial direction.

Strength, Stability and Crack Resistance Design

The strength design of any usual structure is being carried out by means of comparison of stresses caused by the designing loads having considerably small probability together with the limit stresses which the structure can receive. In this case a reduced stress compared to the nominal strength of materials of the structure is taken into consideration.

For the tower such an approach was found to be nonconvincing. There is a definite assurance as far as the size of the normal stress is concerned, stress which is caused to 95% by the weight of the entire structure. The quality of building materials were under a very thorough observation and there is no doubt in total reliability of the concrete and reinforcement's strength. Still doubtful is the correct choice of the designing wind load as well as the correct determination of breaking stress in the ring section. In connection with this the following condition of the rigidity of the shaft was adopted: the breaking bending moment in each section was determined under the given permanent normal force, must be twice the size of the bending moment caused by the wind load. The breaking bending moment in the section of the shaft of the tower is determined under the assumption that in the part of the ring section the concrete is stressed and these stresses reached the prismatic strength; and in the reinforcement they reached the ultimate compression strength in concrete (0.2%) $R^1a = 4000$ kg/cm^2.

In the tensile zone the concrete resistance was not taken into consideration and the reinforcement was considered to have the yield point at $R_a = 4600$ kg/cm^2. This conventional yield limit somewhat exceeds the reject minimum (4000 kg/cm^2) and approaches the average statical. It is assumed that the following distribution of stresses will take place along the cross section (Fig.5). In the compressed zone (φ) the stresses in concrete and in the reinforcement are uniform (the orthogonal diagram); in the tensile zone (ψ) as well. Between the compressed and tensile zones it is assumed that a neutral zone is located in which neither concrete nor reinforcement are stressed.

It is assumed that the strends in the tensile zone have a stress limit that is $R_H = 19200$ kg/cm^2; and in the compressed zone

$$\sigma_c^1 = 19200 - 4000 \frac{1.5 \times 10^6}{2.0 \times 10^6} = 6200 \text{ kg/cm}^2$$

and in the neutral zone the stress in strends is assumed to be equal to original minus the losses $\sigma_c = 11050$ kg/cm^2.

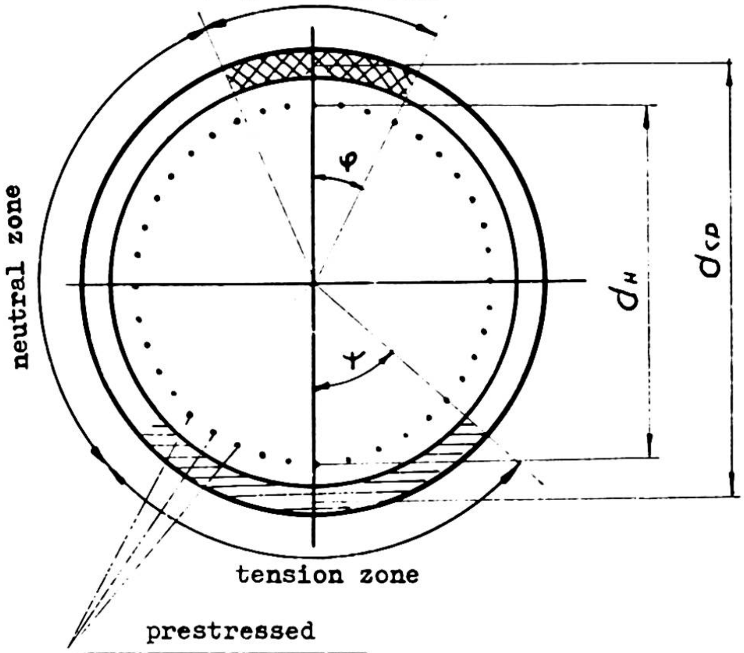


FIG.5

The size of the tensile zone is selected in such a way that it must satisfy the following conditions: $S_\delta = 0.8\, S_0$, where S_δ – the statical moment of compressed zone calculated in relation to the centre of gravity of tensile zone, S_0 – the statical moment of the entire area of concrete situated above the centre of gravity of the tensile zone, related to this centre.

From this condition we receive an equation to determine the size of the tensile zone depending on the size of the compressed zone

$$\varphi\left(\frac{\sin\varphi}{\varphi} + \frac{\sin\psi}{\psi}\right) = 0.8\left(\frac{\sin\theta}{\theta} + \frac{\sin\psi}{\psi}\right)$$

This equation is invariant to the reinforcement of the normal force/diametre.

Solving this equation we can compile the following table which gives us the values of the compressed and tensile zones:

φ	<1.28	1.29	1.31	1.33	1.35	1.37	1.39	1.41	1.43
ψ	1.55	1.46	1.37	1.28	1.19	1.10	1.00	0.89	0.77

The size of the compressed zone can be determined from the condition of equilibrium of all stresses acting in the cross section:

$$\varphi = \frac{N + d_{cp} f_a R_a \psi + d_H f_n\left[\pi\sigma_0 + \psi(R_H - \sigma_0)\right]}{d_{cp}\delta R_{np} + d_{cp} f_a R'_a + d_H f_H (\sigma_c - \sigma'_c)}$$

After this the breaking moment

$$M_p = \tfrac{1}{2} d^2_{cp}\delta R_{np}\sin\varphi + \tfrac{1}{2} d_{cp} f_a (R_a \sin\psi + R'_a \sin\varphi) + \tfrac{1}{2} d^2_n f_n \times$$
$$\times\left[(R_H - \sigma_c)\sin\psi + (\sigma_c - \sigma'_a)\sin\varphi\right]$$

where N is normal force in the cross section; f_a, f_n – the cross section area of unstressed and stressed reinforcement related to the unity of the perimetre. The crack resistance design was carried out in the elastic state. At the wind load equal to 0.75 of assumed there was no tensile stresses allowed in the concrete.

The Design Data and Observations

For the three forms of harmonic oscillations the periods of the fluctuations were determined at 13.1 seconds, 4.6 seconds and 2.7 seconds.

Actually the period of fluctuations was found to be 11.3 sec in the main form. This fact shows that the rigidity of the shaft was 1.35 times underestimated. When designing the fluctuations the module of deformation of concrete was also reduced; it should have been taken considering the age of concrete at the impact load equal to 390000 kg/cm^2, actually was taken 300000 kg/cm^2. Evidently all numerous elements filling the tower inside as well as external constructions participated in the total work.

As a nominal wind load it is assumed by the norms such a probable wind load which is equal to 1/7300, i.e., such a load which would be surpassed on the average during 1/7300 of the considered period of time, for instance, during 1.2 hours a year. At this wind velocity according to the design the statical deflection of the upper part of the tower will be equal to 5.8 m, the amplitude of fluctuations according to the first form 1.4 m, the second – 1.1 m and the third 0.2 m.

It can be expected that the actual deflections of the tower will be less than mentioned above since the rigidity of the entire structure was not taken into account in design.

A one-sided heating by the sun was also taken into consideration. According to design the deformation must reach 3.3 m. The small amount of observations carried so far give somewhat smaller value.

During the 24 hours the upper part of the tower moves along a very complicated closed curve with a maximum diametre of 2.5 m.

There were apprehensions that the visitors of the tower would experience large and unpleasant fluctuations of the tower. The experience showed that visitors do not feel any of those fluctuations.

A very vast program of observation of the condition of the structure was organized. The wind velocity is being measured at various heights. Deformations, oscillations of the tower, conditions of the concrete, lightning discharges, the status of the prestressed strends as well as sedimentation and deformation of the soil under the structure are also being observed and measured.

SUMMARY

In the design of the Moscow prestressed r.c. 533 m TV tower both static and dynamic effects of the wind load were taken into consideration. Three forms of harmonic oscillations were determined. The amplitudes of frontal and transversal fluctuations were also found. The design was based on the deformed scheme. The s strength computation was performed at the failure stage. Crack resistance and rigidity were determined at the elastic stage.

RÉSUMÉ

Pour le calcul de la tour de télévision à Moscou, 533 m haute et construite en béton armé précontraint, les effets statiques et dynamiques de la force du vent ont été pris en considération. Trois formes des oscillations harmoniques et les amplitudes des divergences frontales et transversales ont été déterminées. Le schéma déformé a été adopté pour les calculs. La résistance de la structure a été calculée pour l'état de destruction, la résistance à la fissuration ainsi que la rigidité pour l'état d'élasticité.

ZUSAMMENFASSUNG

Bei der Projektierung des 533 m hohen Turmes aus vorgespanntem Stahlbeton in Moskau wurden die statischen und dynamischen Wirkungen der Windbelastung in Betracht gezogen. Es wurden drei Formen von harmonischen Schwingungen festgelegt und die Amplituden der Frontal- und Querschwingungen bestimmt. Die Berechnungen wurden auf Grund eines deformierten Schemas vorgenommen. Die Widerstandsfestigkeit wurde auf dem Stadium der Zerstörung, die Rissfestigkeit und Steifigkeit auf dem Stadium der Elastizität berechnet.

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