PERIODICA POLYTECHNICA SER. CIVIL ENG. VOL. 38, NO. 1, PP. 29-40 (1994)

FOUNDATION OF A 157 M HIGH TRANSMISSION TOWER

József FARKAS and László MARCZAL

Department of Geotechnics
Technical University of Budapest
H-1521 Budapest, Hungary

Received: March 25, 1994

Abstract

In a city of Hungary known to have a weak subsoil, a microwave transmission tower 157 m high, with a cylindrical reinforced concrete shaft was built. In its design, possibilities of both spread foundation and deep foundation arose. Beyond dead and live loads, also other effects (wind load, seismic effects, uniform and nonuniform settlement, daily and seasonal temperature variations) had to be reckoned with. In case of spread footing, that should be 25–30 m diameter because of the extreme height, uneven settlement and tilting, rather than soil failure, were the main problems. Effects likely to cause eccentricity like wind load and building inaccuracies had to be analyzed. In case of a deep foundation, which is technically much better, the point of the large-diameter bored piles was suggested to be taken in a clay beginning at 25 m below the ground level, which is in a much better condition than the overlying one. At last, the tower was built on spread foundation. During and after construction, stresses under the slab footing were recorded, horizontal and vertical displacements of the tower observed.

Keywords: spread foundation, deep foundation, settlement, slab footing, stress measurement.

1. Introduction

A microwave transmission tower 157 m high, with a ribbed cylindrical r.c. shaft of 7.8 m inner diameter, 35 cm wall thickness, had to be erected in the Rókus district of Szeged, city in Hungary, The r.c. shaft 110 m high was to be topped by a steel structure 1.4 by 1.4 m in ground plan, supporting an aerial. Subsoil exploration laboratory testing and proposals for the foundation were made by the Department of Geotechnics of the Technical University of Budapest.

Load data:	Permanent load (without weight of footings):	45281 kN
	earth backfill:	35000 kN
	working load:	11793 kN
	wind load (horizontal):	2935 kN
	fixed endmoment:	192834 kNm
	Permissible deflection (tilt):	005 degrees.

The subsoil of the site was classified as rather poor in geological descriptions. A loess stratum 3–4 m thick below the ground level is underlain by thick layers of compressible soft clays and silts. For the purpose of soil exploration a borehole 30 m deep was made at the tower axis, and three boreholes along the perimeter of a circle of radius $R = 15$ m about the axis. The borings confirmed the geological descriptions.

2. Subsoil Conditions

Below the ground level, topsoil and mixed fill was found to a depth of 0.5 to 1.7 m, followed by loess with a plasticity index of $I_p = 12$ to 15% in a thickness of 1.4 to 2.8 m ($\Phi = 13 - 14°$, $c = 48$ to 78 kPa, $E_s = 4.2$ to 6.0 MPa). From a depth of 3 to 4 m below ground level, high-plasticity ($I_p = 38$ to 52%) saturated rich clays were disclosed down to 11 to 16 m ($\Phi = 2 - 9°$, $c = 55 - 75$ kPa), followed by highly soaked, soft silt down to 23 m ($I_p = 9 - 15\%$, voids ratio $e = 0.81 - 0.86$), then down to the borehole bottom (30 m) rich clay with lime clumps ($I_p = 24 - 30\%$) was traversed.

Compression tests were made on undisturbed soil samples to determine parameters needed for settlement computations. Variation with depth of compression moduli for the load range of 100 to 200 kPa is seen in *Fig. 1.* Modulus of elasticity in rich clay at 3 to 16 m is seen to vary about linearly with depth; the silt below it is more compressible, and a less compressible 'competent' soil was found only below a depth of 25 m.

The permeability coefficient of clays is of the order $k = 10^{-7}$ cm/s. Water table in boreholes settled uniformly at a depth of 4.2 m below ground level.

3. Foundation Alternatives

In designing the transmission tower foundation, the following effects have been reckoned with:

a) Permanent and working loads
b) Wind loads
c) Seismic effects
d) Uniform and differential settlements (tilt)
e) Unilateral heating of the tower due to sunshine, causing bending and flexural moments.
f) Seasonal temperature variations in the raising cylindrical shaft, while the foundation slab temperature remains about constant.

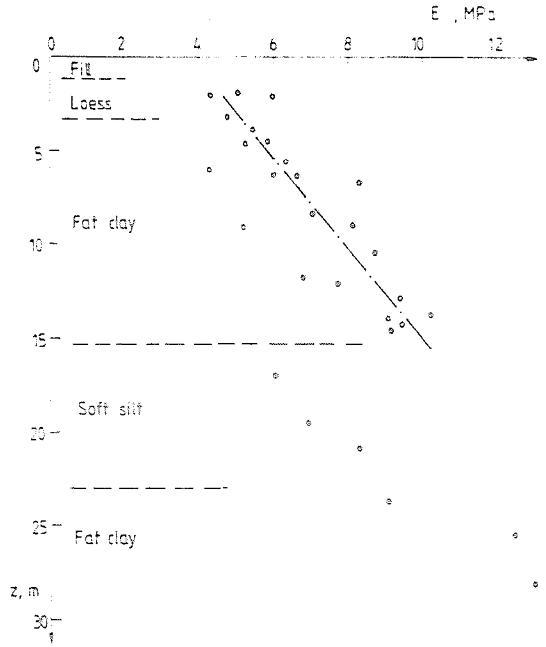


Fig. 1. Variation of the modulus of compression with depth

The tilt of the tower causes additional flexural moments. Tilt has also to be reckoned with as a factor affecting the angle of inclination required for the regular operation of aerials. For a tower 157 m high as a structure with an elevated centroid there are increased requirements for the foundation. In case of a spread foundation — with a foundation diameter of 25 to 30 m because of the significant height — crucial considerations are settlement and tilt rather than ground failure. In spite of the low soil stresses, important uniform settlements may occur because of the great thickness of the compressible soil stratum.

Differential settlements due to the elevated centroid are mostly caused by the moment arising from wind load, but uneven, slanting soil stratification, as well as compressibility varying inside the strata, and constructional inaccuracies may also contribute to it.

A choice between spread or deep foundation may be based on a complex investigation of soil and structure.

According to earlier construction experiences, tower-type buildings are economically constructed with spread foundations — even under not too favourable subsoil conditions. Spread foundations for towers are generally circular or annular r.c. disks. A deep foundation (pile or slurry trench pier) is only used when otherwise stability and deformational conditions are not met due to adverse soil conditions.

4. Spread Foundation

The disk footing planned to support the tower of a height $H = 157$ m had a diameter of $H/6 \approx 27$ m. The required minimum plate thickness v was determined according to an empirical relationship applied in design practice:

$$v = 0.6 + \frac{h - 30}{67},$$

where h = height of the r.c. shaft; yielding a disk thickness of 1.8 m.

The ultimate load of the soil under the foundation disk was determined according to Sokolovski's theory, reckoning also with the reduction in bearing capacity due to non-verticality and eccentricity of the resultant load:

$$\sigma_t = 466\,\mathrm{kPa}\,.$$

Taking the worst possible case of a moment due to an eccentricity of $e = 0.5$ m resulting from wind load, construction faults and other deformations into consideration, the expected maximum soil stress at the disk edges was $\sigma_m = 331$ kPa, giving a safety factor of $n = 1.4$ against ground failure, which was considered as satisfactory with a view on the extreme value.

In settlement calculations it was taken into account that the maximum settlement depends on the stress distribution, which in turn depends on the stiffness of the base plate. According to our experience, even disks considered as rigid undergo larger settlements in the middle, whereas the edges get somewhat cambered to form a 'saucer'. This is, however, not a problem, and is not to the detriment of stability.

Computations assuming a 'Limiting depth' of $m_0 = 0.75\ D$ led to a uniform mean settlement due to a centric vertical normal force

$$s_a = 13\,\mathrm{cm}\,.$$

Investigation of differential settlements due to wind load took into consideration that the dynamic effect of wind load may occur partly as wind blasts of instantaneous effect, and partly — under permanent flow conditions as turbulence.

Beside wind loads, eccentricity may arise from construction inaccuracies. Its expected value ($e = 0.5$ m) was given as a fundamental design value.

It has to be reckoned with, however, that — rather than being constant — wind loads are recurrent at random. Repetitions may add up to an assumed substitutive wind load, whose effect considered as constant produces the same compression of the subsoil as that due to a wind load of higher intensity, acting at short intervals. Repetitive loads cause more compression in the soil than would the same load acting permanently. Compression depends on the structure, and cohesion of subsoil, and also pore water pressure is an important factor. Theoretical results of compression tests made by repetitive loading are seen in *Fig. 2.* Successive but not uniform load repetition cause gradual vertical deformation. With an increasing number of repetitions of the same load, the compression increments will gradually decrease. The kind of load variation is such that it is never reduced to zero. With a high number of repetitions a condition is eventually arrived at where the compression practically ceases to increase.

As mentioned above, the moment due to wind load, causing eccentricity, acts for but a slight period as compared to the prolonged consolidation in the clay. Thereby only part of the total deflection value computed from the wind effect has to be taken into consideration. On the other hand, some accumulation of permanent deformations due to repeated loads has to be reckoned with.

According to results of repetitive loading and in compression test on clays made earlier at the Department of Geotechnique, within the range of soil stresses assumed for the considered tower — load repetitions increase the compression by about 20 to 25% as compared to the value obtained from conventional compressive tests.

The planned transmission tower being an especially slender structure, stability problems were devoted a special attention. It was taken into consideration that the tower with its foundation and the underlying soil form an 'integer structure', therefore in the phenomenon of deflection, also the elastic rotation of the foundation interacts.

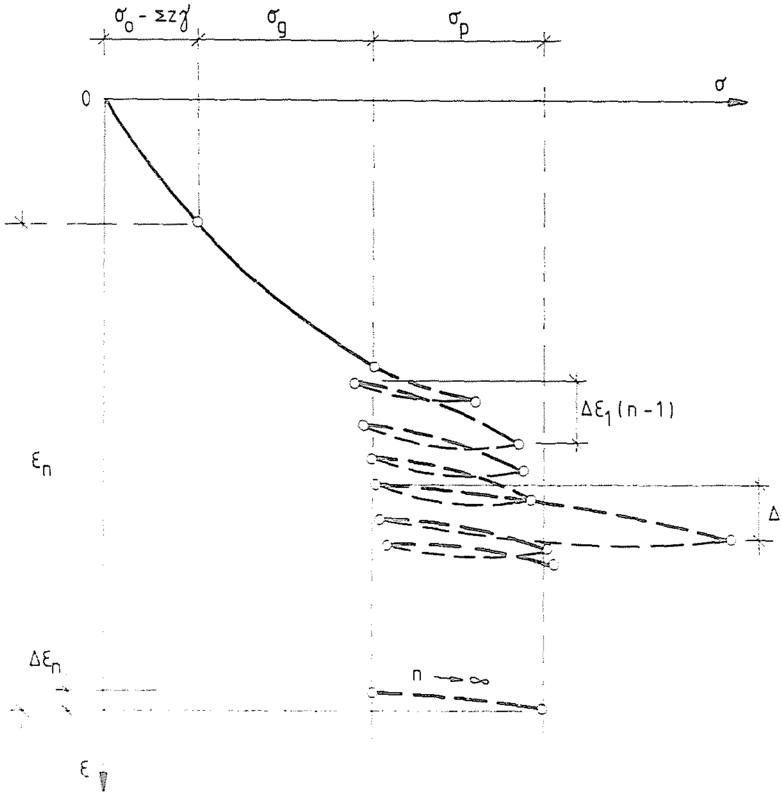
In conformity with these considerations, only $\alpha = 0.3$ times the moment due to wind load was reckoned with. According to computations, safety with respect to turning $n_0 = 6.2$, and to slip $n_s = 3.8$.

Also the possibility of a horizontal force due to seismic effects had to be reckoned with. The expected seismic intensity of that site may be classified as 6 according to the MKS scale agreed in Europe.

The computed seismic force for an acceleration of $a = 0.10-0.25\ \mathrm{m/s^2}$:

$$S_r = 1500 \quad \text{to} \quad 1700\,\mathrm{kN}\,.$$

According to the literature, in design, superposition (simultaneity of wind force and earthquake may be excluded. In conformity with our investigations, the Kishida criterion of dynamic soil liquefaction is not met under



$\Delta\varepsilon_1$ - specific deformation upon the first load variation

$\Delta\varepsilon_i$ - specific deformation upon the first peak variation

$\Delta\varepsilon_n$ - specific deformation after a very high number of load repetitions

ε_n - overall specific deformation after a very high number of load repetitions

σ_0 - stress due to dead load at the considered depth

σ_g - vertical stress due to dead load of the structure

σ_p - mean stress due to wind load

Fig. 2. Compression due to load repetitions at random in confined compression test

the given subsoil conditions. Thus, in final account, since the horizontal seismic force is less than the maximum horizontal force due to wind load simultaneity being excluded, horizontal stresses due to wind load have to be considered as critical.

In our investigations the time-dependant process of settlement (consolidation) has also been studied. According to our computations, 50%

of compression occurs in 18 to 20 months, consolidation to 80% takes 5.6 years, while to 93%, 9 years.

5. Deep Foundation

With a view on subsoil and groundwater conditions, as well as on critical loads and allowable ultimate value of deflection, from engineering aspects, deep foundations were suggested to be taken at a depth of 25 m below ground level, in the clay of harder consistency of the options of pile foundation and slurry wall foundation, cased Benoto piling was given preference. Here structural loads are transferred to the piling via a r.c. annular pile cap.

Loading test results for Benoto piles made earlier near the site, in similar, water saturated areas of impermeable subsoil have been compiled. Load capacity values of piles *Phi* 90 cm for 5 mm settlement ($P_L^{5\,\mathrm{mm}}$) have been plotted vs. pile length in *Fig. 3.* Taking the lower straight boundary into consideration, for a pile length of 26 m, an ultimate load capacity of $P_L = 1730$ kN could be predicted. (Mean values led to an ultimate load capacity of 2280 kN.)

Foundation was suggested to be made with 70 piles, supporting the tower with the intermediary of an annular r.c. headplate of an outer radius $R = 13.3$ m, and an inner radius $r = 3.2$ m.

6. In Situ Measurements During and after Construction, Checking for Settlements, Base Stress and Tilt

For structural reasons, and also to eliminate the upper, soft layers, the designer of the tower structure chose spread foundation, with a foundation level 4.00 m below ground level. Instead of the originally planned circle, for constructional reasons, an octagonal base plate was made, with the same surface area as that of the original circle. The plate with a diameter of 18.0 m, was 1.10 m thick at the edges, but below the tower shaft proper, twice as thick, (2.20 m thereby significantly increasing the stiffness in the middle area).

The tower shaft having a square cross-section with corners rounded by circle arches of 8 m diam is a concrete prism 100 m high, with 35 cm wall thickness, and double ribbing on every side, supporting the steel structure of the aerial. Overall ground plan layout is seen in *Fig. 4*.

While constructing the base plate, strain gauges were built in at five spots; at the centre, and at four extreme points at the plate edges arranged at 90°.

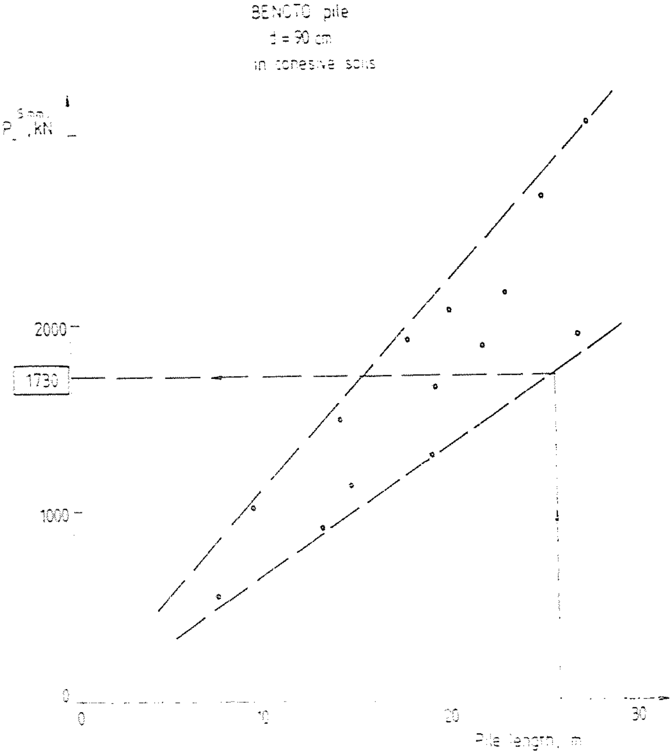


Fig. 3. Ultimate load capacity of the pile vs. lenght

Strain gauges were built in as seen in *Fig. 5.* Each gauge was put in a steel tube of larger diameter, the space around the gauge was filled with cement mortar, and the cables were led in an inner, narrower plastic tubing to above the plate level.

At the bottom, the gauge was placed on sand bedding, shielded around lest the concrete from the base plate contacts the gauge from below. Casings were fastened to the plate reinforcement.

Pressure-deformation relations were checked by calibrating the staring gauges. After installed at the site, measuring capabilities of the gauges were checked and adjusted to the initial value.

The frequency of measurements was adapted to the progress rate of the construction. Measurements were made at the end of significant building stages, and at every 20 m rise during the construction of the tower shaft. Settlement measurements at eight points were made simultaneously with the base stress measurements. Four of the eight measurement points

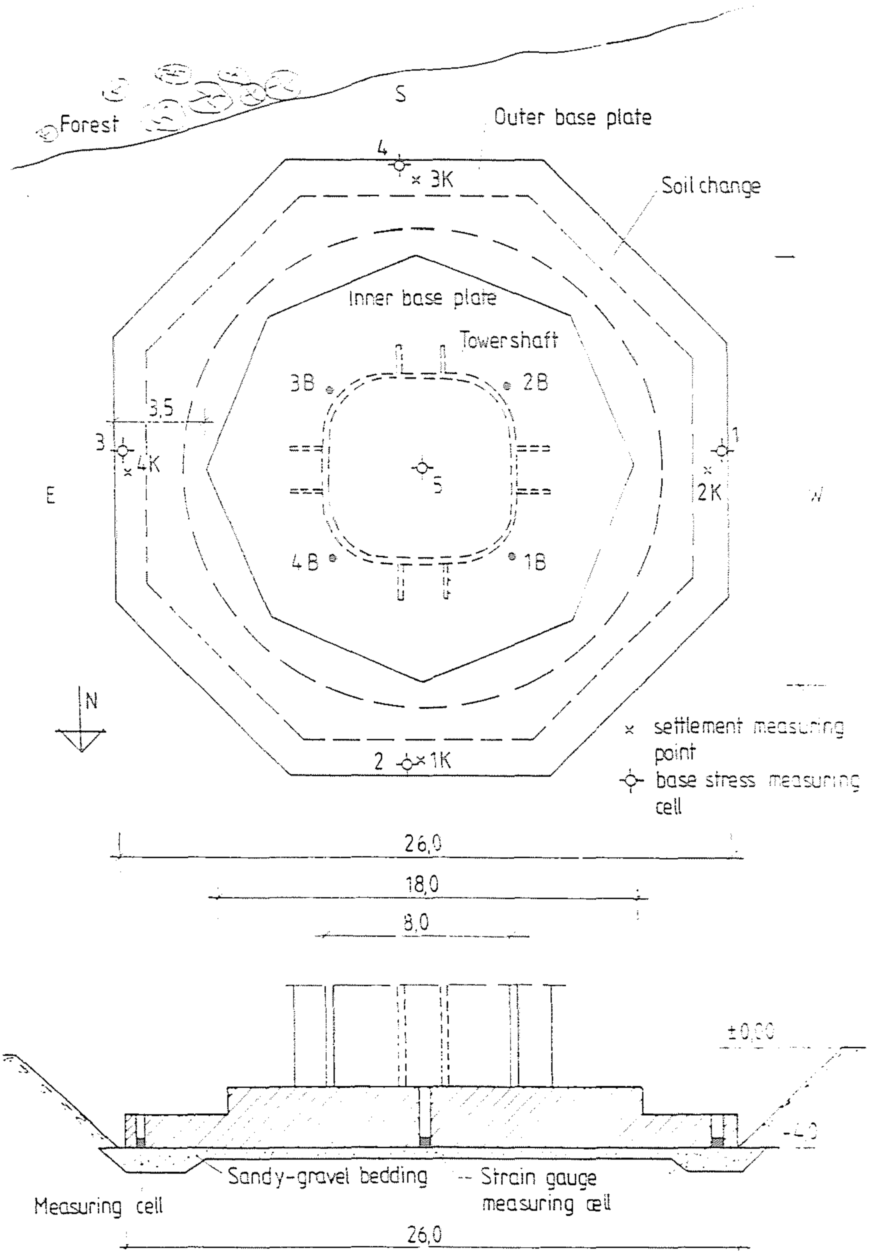


Fig. 4. Ground-plan layout of the tower, measuring point locations

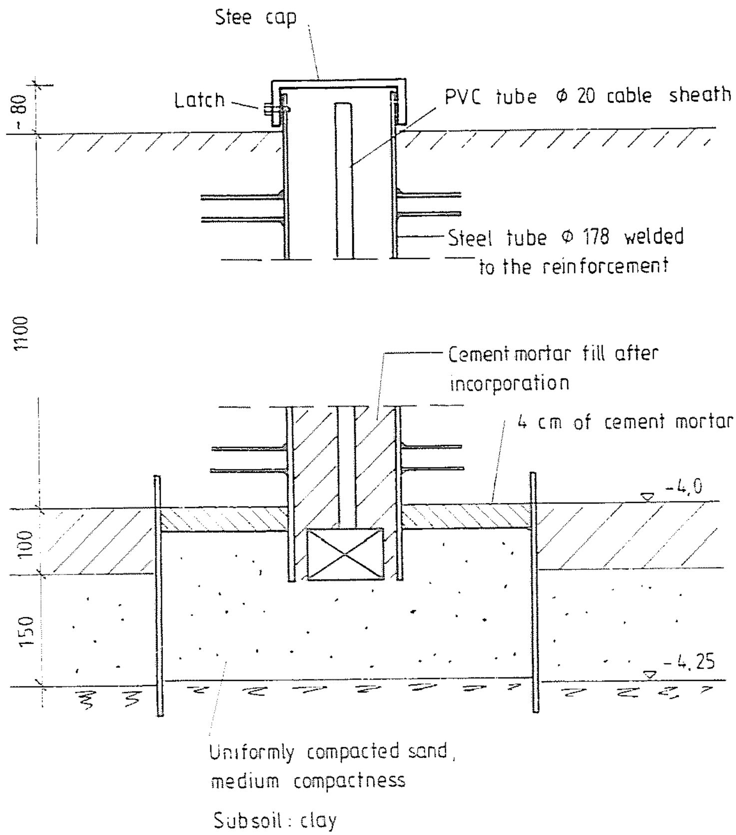


Fig. 5. Incorporation of base measuring cells

were located at the thinner part of the plate, near the base stress measurement points, while other four at the inner, thicker part, near the tower shaft. The central point was not seen from outside.

Deflection of the tower shaft during construction was checked by means of an optical plumb line, and even the least deflection corrected. Thereby the tower axis kept its verticality and when completed the deflection was less than 0.01°.

For the post-construction observation of tilting, aiming points were fixed at every 20 meters on the shaft and checked by precise geodetic goniometry.

Measurement results have been compiled in *Fig. 6,* showing load increases at successive stages of construction, computed and measured base stresses at base level and the simultaneously measured settlements.

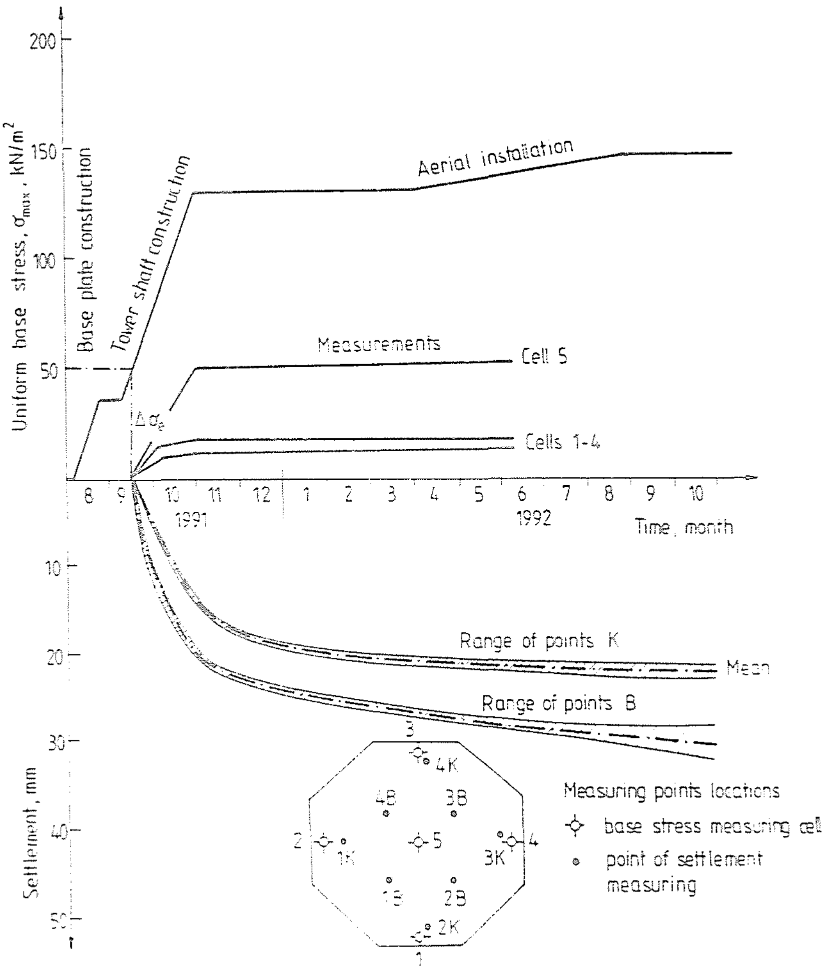


Fig. 6. Measurement results

7. Evaluation of Measurements, Conclusions

The first comment refers to the start of the measurements: namely, much of the load is due to the weight of the base plate, but practically, settlement measurements started only when the base plate was complete, and its outer part received soil backfill. As seen from the figure, the load at this stage amounts to 30 to 40% of the overall load, but the corresponding portion of settlements is missing from the settlement measurement. The part of subsidence not measured may be determined by computation. According to such calculations, 3–4 cm of the total computed settlement do not appear in the measurements, that is, in the figure, every measurement should be increased by that much. The base stress measurements truly reflected the increases in the tower shaft load. But the measured base plate stress values agree with computed values only below the stiff central slab, while strain gauges under the slab edges of lesser stiffness indicated 30–40% of the average base stress. Consequently, it may be assumed that

- the base stress is proportional to the stiffness;
- because of the sand and gravel bedding layer, the base stress distribution is parabolic despite the fact that the subsoil under the base plate is clay;
- the sensitivity of the gauges is somewhat less than the effect of soil compression, so they give correct values only for marked pressure variations, as seen for the middle gauge;
- the deflection of the tower may be computed from the differences in settlements which in turn are in fair agreement whit base pressure differences indicated by the gauge over 25 m base length, the maximum deviation is 8.8 mm. This value is much less than the design prediction.

At the top edge of the 100 m concrete shaft, this tilt means a deflection of 4 cm from the vertical. Measurements showed that in a wind of medium intensity, the tower had a sidesway of 2 cm. Thus, the deflection of the tower hardly exceeds the magnitude of an ordinary sidesway.

There has been no measured data on the effect of stresses due to lasting, intensive wind gusts, because of favourable weather conditions during construction.

The final value of settlement may be estimated from consolidation computations. The construction time was somewhat less than foreseen. At the writing of this paper, 14 months have passed and about 45% of consolidation has occurred, thus the actually measured max. 32 mm may eventually increase to 70 to 80 mm. Also this is less than computed, but allowing for the initial settlement missing from the measurements, estimated at 30 to 40 mm, our computations seem justified.

Anyhow it may be concluded that for the settlement of a large foundation slab of such proportions, it is exaggerated to assume the ultimate depth according to a geostatic pressure of $0.2\ t\gamma$, of even the Jáky's value of $m_0 = D$, since comparative computations invariably yielded higher, in the extreme case, twice this value.