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"Can The Tower Be Retained"

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Abstract: *This paper focuses on the analysis of two towers of an industrial plant exhibiting extreme deflection during service loads under heavy wind conditions. The towers are 90 m and 35 m in height, respectively and are interconnected with structural steel operating platforms.*
The nuts have flown off at some bolted joints in the interconnecting steel structure due to high stress induced by deflections.
The deflections measured at the structural steel towers had nearly twice the value permitted by the respective standard in the case of the 90 m high tower and approached the value permitted by the standard in the case of the 35 m high tower.
The herein detailed complex study – covering the strength analysis of the towers, the analysis of wind effects, and the review of the foundations – has been elaborated in order to determine the causes and consequences of the experienced deflections at the plant as well as to conclude the eventual actions to be taken.
The primary consideration for the conduction of the tests and analyses the determination of the eventual actions to be taken was to retain the towers and not to have them demolished.

Keywords: *Dynamic wind effect, Karman effect, change in groundwater level, deflection, high towers*

1. INTRODUCTION

Significant deflections were observed in an industrial plant under the conditions of strong winds tower 1/A which measure 90 m in height and the tower 1/B of 35 m in height, which are connected to each other by structural steel walkways.
The deflection measured at tower 1/A was nearly twice the value permitted by the respective standard and on the other hand it approached the value permitted by the standard at tower 1/B. Similar phenomena were observed already at the tower-like constructions in another unit of the same plant as well as at the tower-like constructions in another plant.

The structural steel towers 1/A and 1/B, mounted on a common foundation, were built more than 30 years ago, in the 1970s.
The common foundation of the towers is a reinforced concrete slab that is 2.50 m thick, with diameter of 19.00 m, supported by 52 reinforced concrete piles.
Tower 1/A is connected to the foundation by 48 ø75 mm anchor bolts and tower 1/B by 16 ø65 mm anchor bolts. The two towers are interconnected by structural steel operating platforms; the centre-to-centre distance between the towers is 8.50 m. The height of tower 1/A is 90.00 m and its inside diameter is 3.80 m while the height of tower 1/B is 35.00 m and its inside diameter is 1.60 m. The side view of the tower is shown in Figure 1.

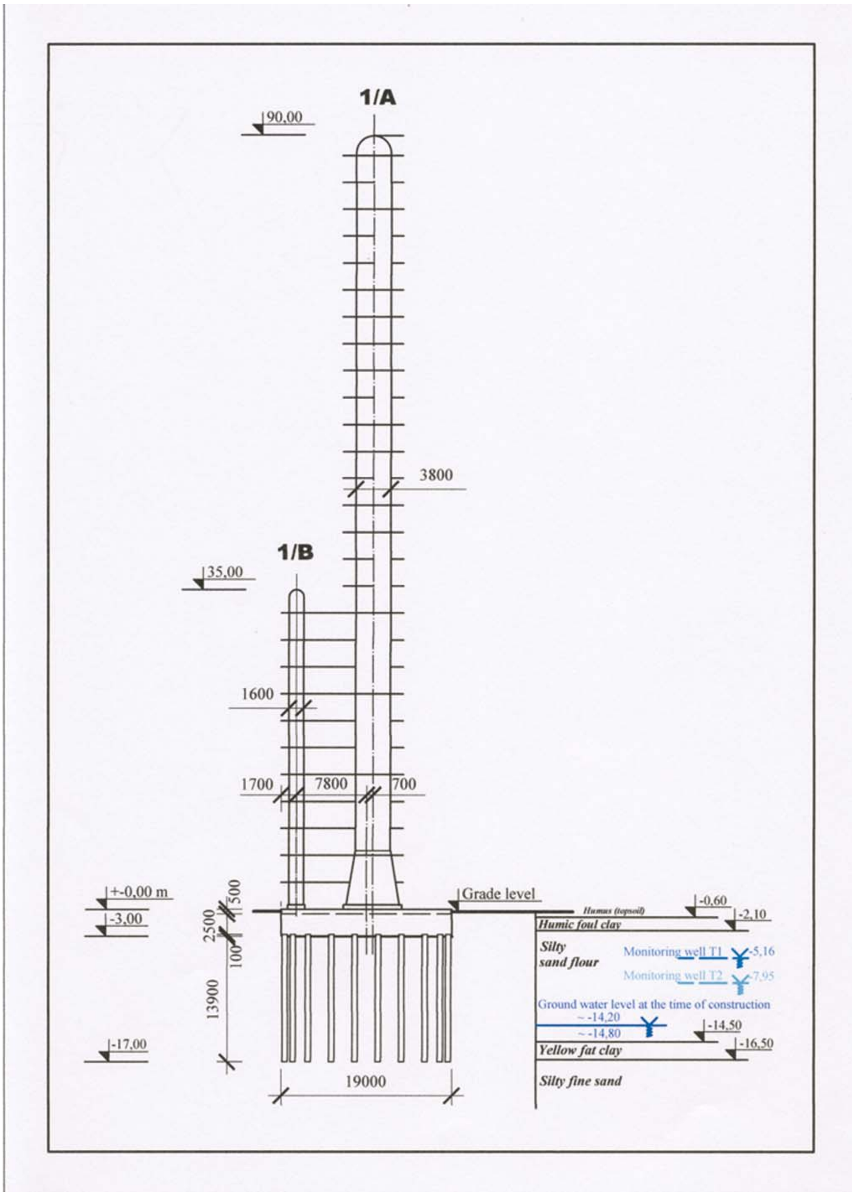


Figure 1. Side view of towers 1/A and 1/B, soil and ground water conditions

A complex study – covering the strength analysis of the towers, the analysis of wind effects the review of the foundations – has been elaborated in order to determine the causes, effects consequences of the frequently observed large deflections at the plant as well as the eventual actions to be taken.

The study covers the following areas:

- analysis of wind effects,
- strength analysis of the towers,
- static analysis of the foundation.

This paper describes the analysis of the wind effects and the static analysis of the foundation. The primary consideration for the conduction of the tests and analyses the determination of the eventual actions to be taken was to retain the towers not to have them demolished.

2. DESRIPTION OF REVIEWS

2.1. *Analysis of wind effects*

Combined wind velocity deflection rate measurements were conducted several times in order to determine the exact magnitude of the significant deflections experienced often under heavy wind conditions at towers 1/A and 1/B.

On the first occasion ±25–30 cm deflection of tower 1/A was recorded by bidirectional measurement conducted at ~40–50 km/h wind velocity. On the second occasion the deflection of tower 70–80 cm 1/A measured at ~50–60 km/h wind velocity along with 6–8 cm deflection at the top of interconnected tower 1/B.

Various wind effects act on the towers since the function of flow velocity varies around the tower shells. The wind loads affecting the towers were studied with the measured deflections, the measured wind velocities and the wind velocity used as basis of design.

We have determined the critical wind velocity at the towers ($v_{critical}$= ~47,6 km/h), the value characteristic for the flow around the towers, the Reynolds number as well as the natural frequency and vibration period of the towers.

The wind velocity measured at large lateral deflection was equal to the calculated value of the critical velocity.

We determined the magnitude of the wind loads in directions both parallel with and perpendicular to the wind direction.

Based on the calculated and measured values it could be established that Karman effect occurred at the towers with lateral vibrations resulting from periodical vortex shedding (Karman turbulence) occurring at the towers at the critical wind velocity.

The Karman type vortex path developing around the circular cylinder is shown in Figure 2.

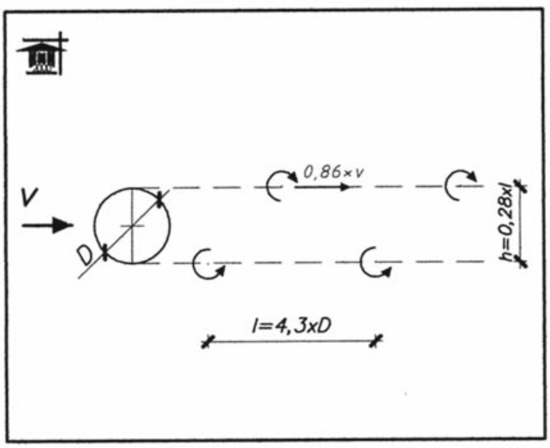


Figure 2. Karman type vortex path behind the circular cylinder

2.2. *Static analysis of foundation*

2.2.1. *Soil and ground water conditions*

The findings of the expert opinion on soil mechanics prepared on the basis of exploration drillings are as follows (Figure 1):

- The surface is covered by a 0.60 to 1.50 m thick layer of brown humic clay.
- Below this, down to ~ 13.80 m depth from grade level yellow silt mixed with floury sand and silty floury sand layers, respectively, of Aeolian origin, formed in the Pleistocene period – are located which change to foul clay at some places.
  These layers are in extremely dry condition and have a macro-porous structure according to the odometer tests with watering (i_m= 2.9% – 11.7%, i.e. susceptible to slumping)
- Under the above layers a strata set comprising mainly clays of varying cohesiveness, formed in the Pannonian epoch, with the intrusion of silt, silty floury sand and sandstone shelves detectable at some places.
- Ground water was detected only in deeper boreholes at ~14.50 to 15.00 m depth.
- The study recommended locally drilled Franki pile foundation as the foundation design for the towers and specified P_H= 95 Mp as the load-bearing capacity of the piles.

Soil probing tests have had been conducted with the use of three (3) CPT probes at ~16.00 m depth and five (5) dynamic probes at ~16.00 m depth for the review of the tower foundation. The tests with the CPT probes were conducted in the surroundings of the towers, with two of the dynamic probes under the tower foundation and with three of them in the surroundings of the towers.

The following conclusions could be drawn from the results of the probe tests:

- The strata under 13.00 to 14.00 m, formed in the Pannonian epoch, have an unchanged good load-bearing capacity.
- The ~10.00 m thick layer of silt mixed with floury sand and silty-floury sand, respectively, above the strata formed in the Pannonian epoch is soil soaked by water – due to the rising and fluctuation, respectively, of the ground water level and its physical properties in respect of the load-bearing capacity have deteriorated.

Figure 3 shows the typical averaged tip resistance values of the CPT probes and Figure 4 shows the typical averaged values of impact counts for dynamic probes.
In the case of the CPT probes the values indicated in red colour are below 5 MPa and the value of the tip resistance dropped even below 1 MPa at some places.
In the case of the dynamic probes the values indicated in red colour represent impact counts less than 5 and even 0 and 1 values were recorded at some places.

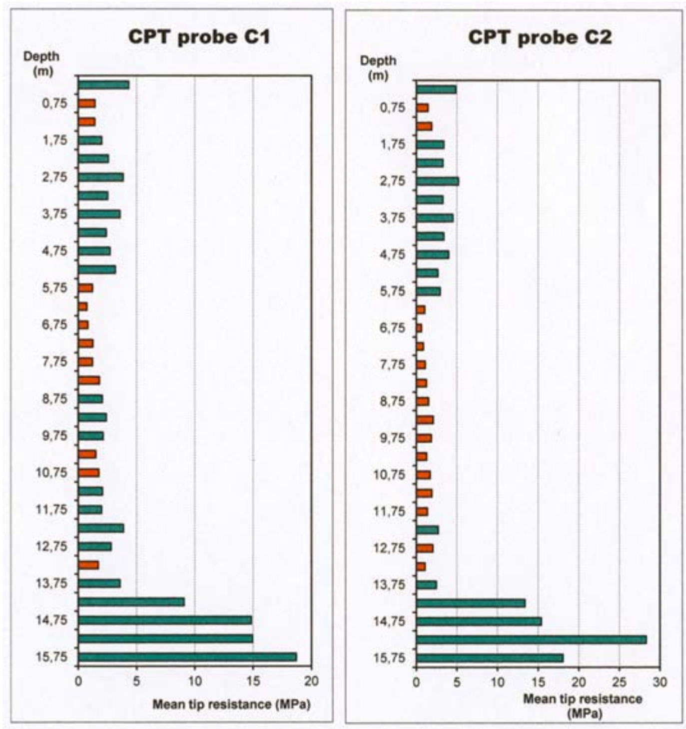


Figure 3. Typical mean tip resistance values of CPT probes

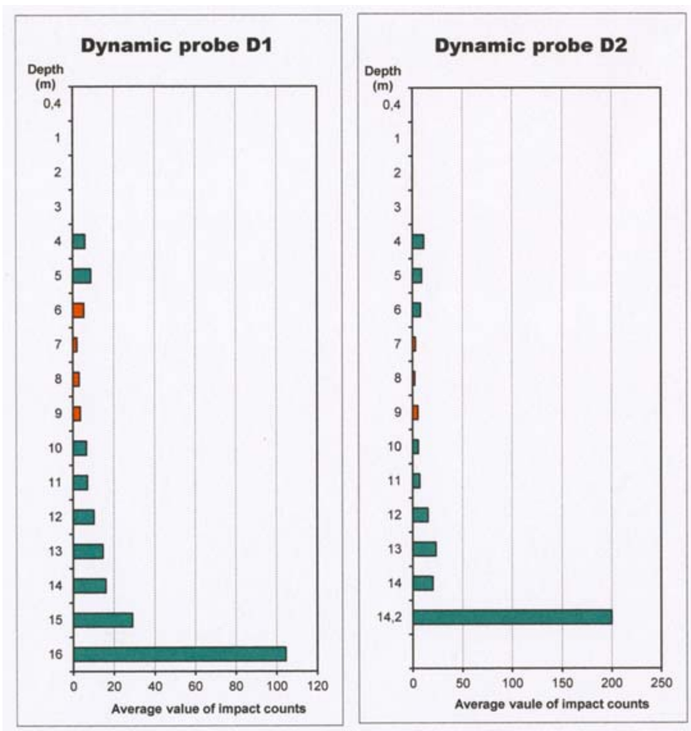


Figure 4. Typical averaged values of impact counts for dynamic probes

Figure 5 shows the averaged values of the dynamic probes in the section taken by the dynamic probes under the tower foundation proving that the soil properties have deteriorated significantly under the foundation.

The results of both the CPT and dynamic probe tests indicated that the ground water level varied between ~6.00 m and ~8.00 m below grade level.

The ground water approached to ~1.80 m the bottom plane of the foundation slab. Figure 6 shows the rise in ground water during the past nearly thirty years and the readings of the dynamic probe tests under the tower foundation.

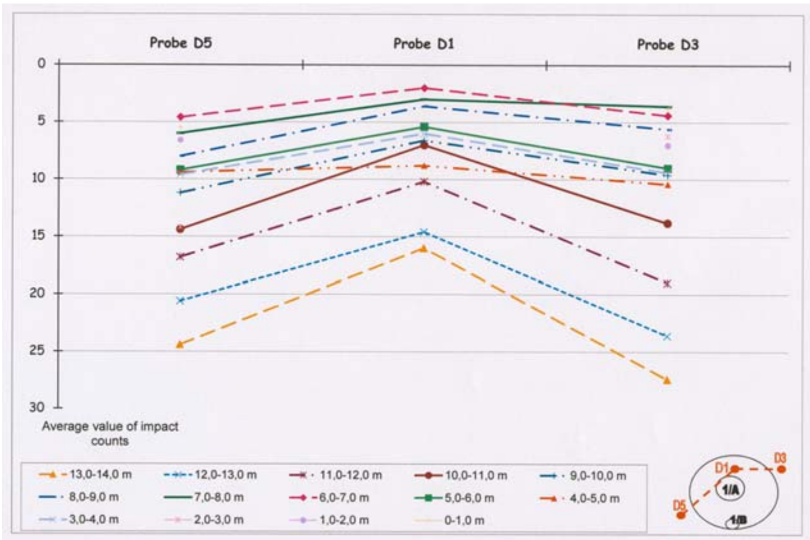


Figure 5. Section taken by dynamic probes D5-D1-D3 (mean impact counts)

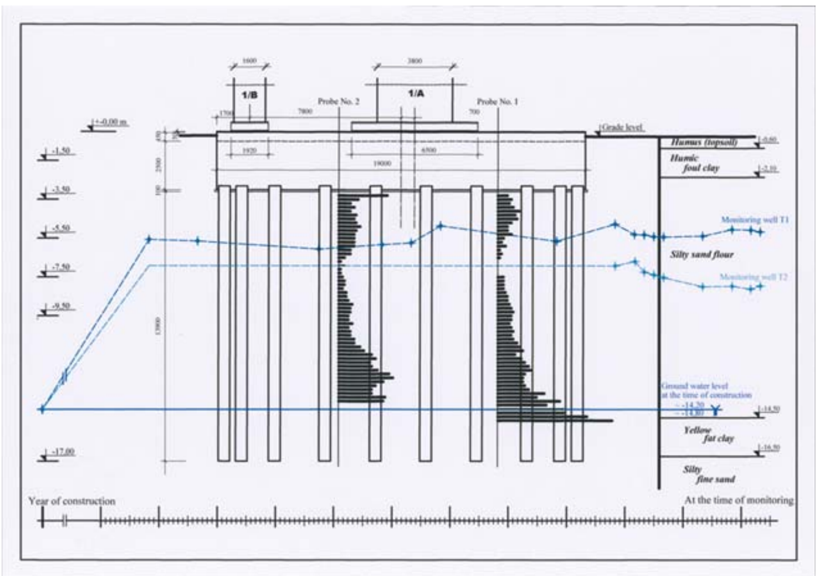
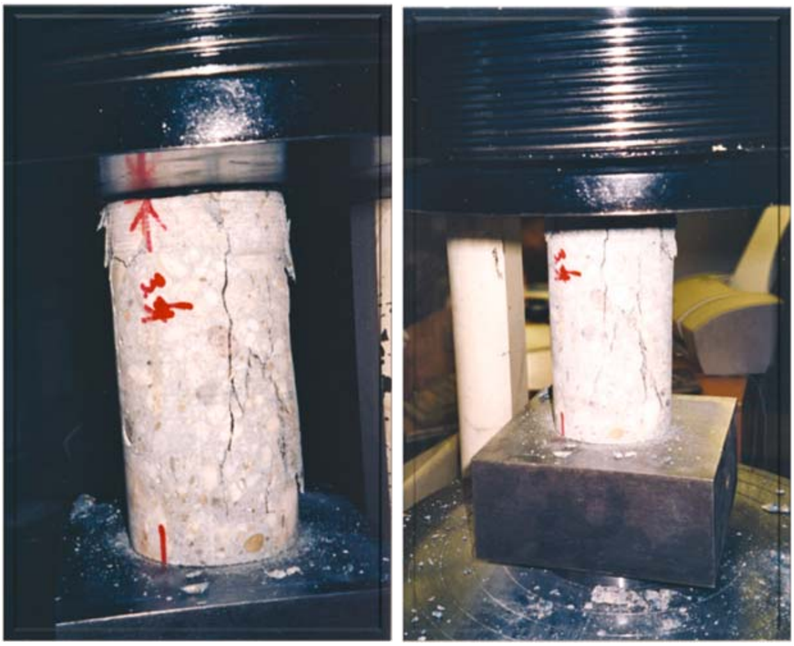


Figure 6. Rise in ground water level as determined by dynamic probes under the tower foundation

2.2.2. *Analysis of concrete quality*

Concrete core samples were taken by drilling through bores at two locations in the foundation and cutting out along 70 cm length at one place for determining the quality of the reinforced concrete foundation.

Based on the compression and tensile strength tests of the concrete specimens the strength class of the concrete is C40 while B200 (~C16) was the designed class.



Photos No. 1 and 2: Concrete tests

2.2.3. *Control analyses of foundation*

The analysis of the common foundation of the towers was performed with the results of the strength tests conducted on the concrete samples and by assuming that the reinforcement is compliance with the design.

The analysis was performed for three loading cases (operating, construction and hydrotesting), nine load combinations and three wind load (X, Y and diagonal) directions with the soil properties recorded on the basis of the soil mechanical probe tests and those existing at the time of construction, respectively, taken into consideration.

The calculations were performed by the Axis VM7 finite element computer program and computed manually. The finite element model built for checking the foundation comprises 2133 nodes and 4058 surface elements (see Figure 7).

Based on the results of calculation the load-bearing capacity of the piles and the foundation slab was evaluated and the anchor bolts and deflections were investigated severally.

The results from the actual standards and the ones from the standard being operative at the time of design were also compared.

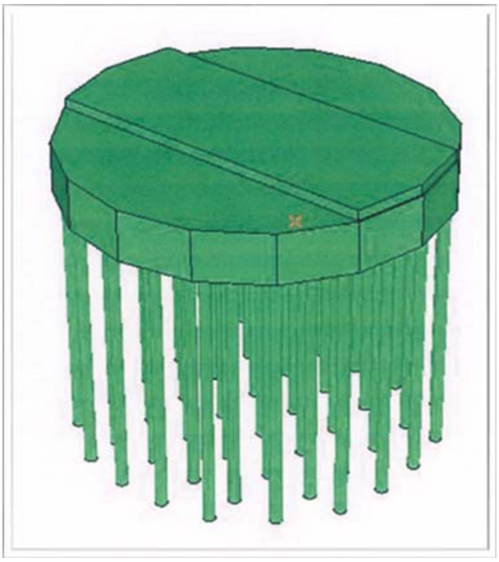


Figure 7. The finite element model built for checking the foundation

2.2.3.1. *Analysis of the piles*

52 Ø600 mm size and 14.0 m long Franki system reinforced concrete piles were designed to support the 19 m diameter foundation slab.
The soil mechanics study prepared at the time of construction specified the load-bearing capacity of a pile as 95 Mp.
The comparison of the limit load-bearing capacity value of a pile obtained by control calculation with the load values calculated at various loading conditions is summarized in Table 1 below.
The variation in load-carrying by shell friction due to the rise and fluctuation of ground water level means ~20% reduction in the total load-bearing capacity which is still within acceptable limits.
The rise in ground water level entails ~1–2 cm post-subsidence withdrawn over a long period of time.

Loading conditions	***Limit load-bearing capacity [kN]***	***Acceptability***	***Load [kN]***
Operating	958	> Acceptable	656
Out of service	958	> Acceptable	646
Hydrotesting	958	> Acceptable	773
Hydrotest + 0.6 wind	958	> Acceptable	652

Table 1: Checking of piles

2.2.3.2. *Analysis of the foundation slab*

The 19.00 m diameter, 2.50 m thick reinforced concrete slab joins the grid of piles.
The concrete grade of the foundation slab is C40 as determined by tests, the design rebar quality is B60.40, its reinforcement is bottom and top bar-mat reinforcement composed of ø22/250 mm size rods.

Based on the results of the completed calculation it could be established that with the assumption of the current soil properties, the design reinforcement and the concrete grade found by the tests the load-bearing capacity of the foundation in operating and empty condition is at the limit of permissibility (see Table 2 below).

Loading condition	*M_H [kNm]*	*Acceptability*	*mv [kNm]*	
Operating	2188	> Acceptable	2162	mvy
			1524	mvx
Empty	2188	> Acceptable	2102	mvy
			1462	mvx

Table 2. Checking of foundation slab

2.2.3.3. *Checking of anchor bolts*

Tower 1/A is connected to the foundation by 48 Ø75 mm anchor bolts and tower 1/B by 16 Ø65 mm anchor bolts.
No data were available in respect of anchor bolt qualities therefore we have had the material quality of an anchor bolt for tower 1/A determined by tests and found it to be Grade 4.6.

Determining the limit tensile force (with the presumption of faultless material) of the anchor bolts for tower 1/A the cross-section of the bolt was found to be acceptable for the calculated design stress: $F_{design} = 428$ kN < $F_{limit} = 696$ kN.

In respect of break-out the bolts are acceptable with the assumption of concrete grade C40.

2.2.3.4. *Tipping and subsidence*

The constant obliquity of the towers was measured: the maximum resultant skewness is ~285 mm. The obliquity results in a constantly acting eccentric load.

The maximum value of the subsidence calculated for various load combinations is ~60 mm (see Figure 8) and the maximum difference between the subsidences calculated for various points is 54 mm which may cause 300 mm tipping but was acceptable according to the standard specification effective at the time of design.

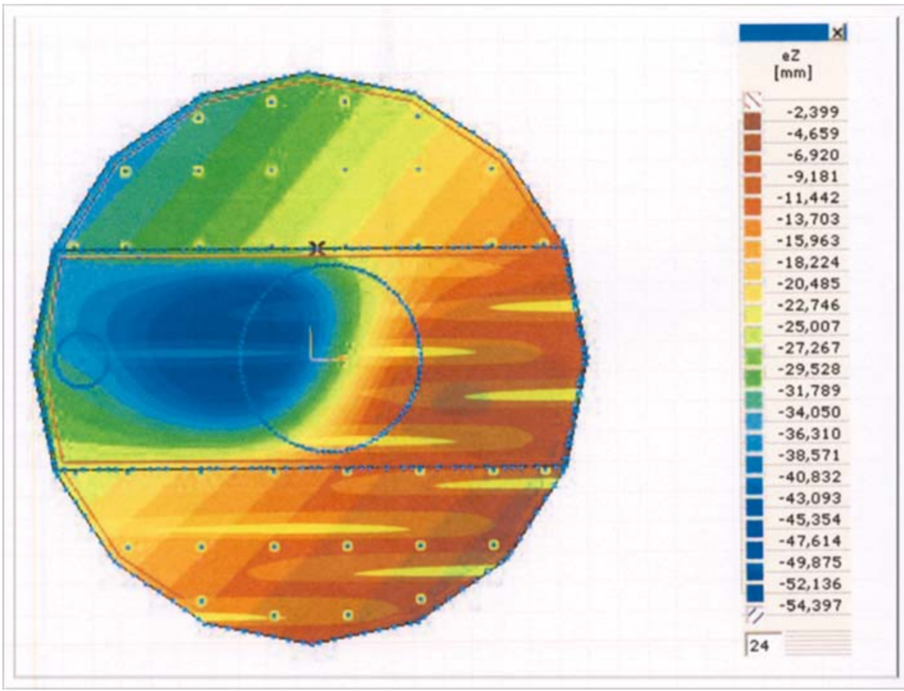


Figure 8. Subsidence

3. REDUCTION OF DYNAMIC EFFECTS

Large deflections and extra stresses occur at towers 1/A and 1/B due to periodically recurring vortex shedding at lateral dynamic wind loads in the case of $v_{critical}$ wind velocity.
The applicability of several solutions for the elimination of dynamic effects was investigated.

3.1. *Attenuation of lateral vibrations*

The vibrations of the towers are caused by periodically shedded vortexes.
The vibrations can be eliminated by making the air flow around the tower irregular and thereby stopping the shedding of vortexes.

If reduction of the dynamic effects is achieved by a solution resulting in the foundation being subjected to additional stresses, then the foundation has to be reviewed in respect of such additional stresses and strengthened if necessary.

3.1.1. *Modification of static skeleton*

The large lateral displacements can be reduced by modifying the static skeleton of the tower with bracings applied at one or more level (see Figure 9).

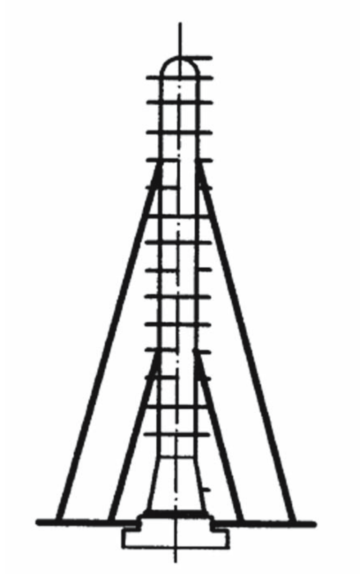


Figure 9. Modification of static skeleton for tower 1/A

3.1.2. *Aerodynamic solutions*

In the case of some aerodynamic solution various types of baffles are attached to the cylindrical shell of the tower (see Figure 10).

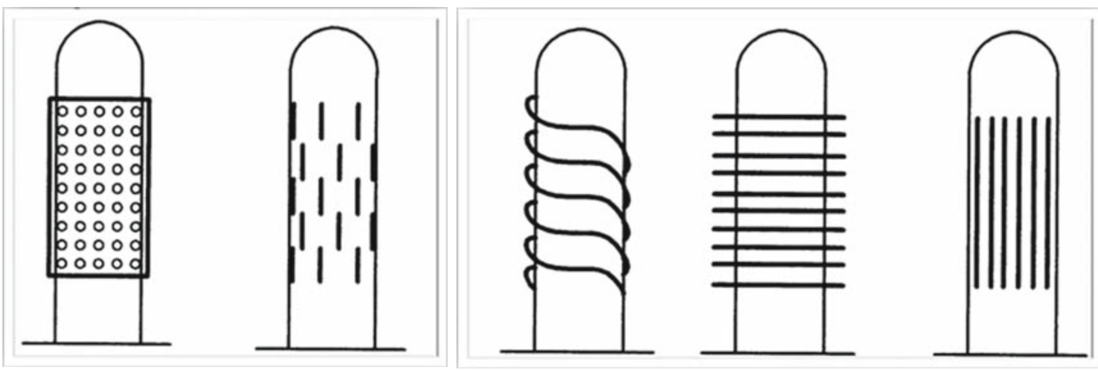


Figure 10. Aerodynamic solutions

3.1.3. *Dynamic attenuation*

Dynamic attenuator can be divided into two main groups: the groups of active and passive attenuators.
For the case of using a solution with a passive vibration damper we have determined by approximate calculation the mass and length of the damping pendulum for tower 1/A.

3.2. *Modification by reducing slenderness of the towers*

As a solution for the reduction of dynamic effects we investigated the possibility of modifying the technological system (internals) of tower 1/A.

3.2.1. *Reducing the height of tower 1/A*

In order to reduce the dynamic effects the top 24 trays are removed from tower 1/A and its height is reduced by 12.00 m to 75.50 m (see Figure 11).
Tower 1/B remains on the existing foundation in unchanged condition and tower 1/A with its height reduced to 75.50 m.

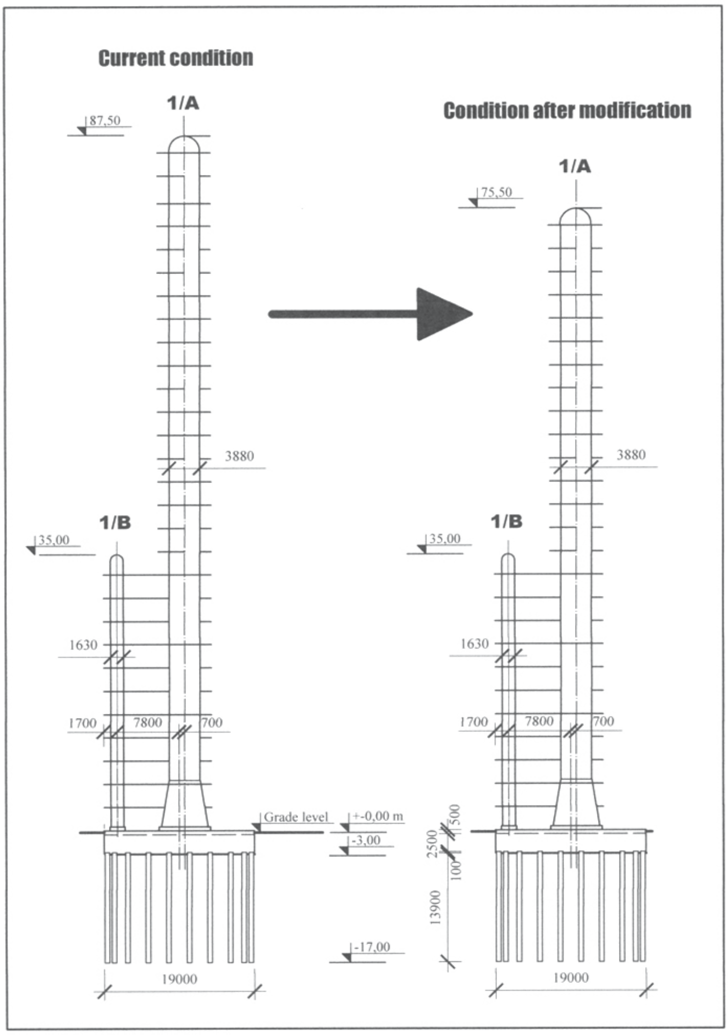


Figure 11. Reduction of the height of tower 1/A

The existing foundation was reviewed with the design reinforcement, the concrete grade determined by tests and the soil-physical characteristics recorded on the basis of probe tests taken into consideration.
The load-bearing capacity of the piles, the foundation and the anchor bolts were found acceptable. The reduction of the height of tower 1/A by 12.00 m resulted in the reduction of the dynamic problems but additional vibration dampening and foundation strengthening may become necessary.

3.2.2. *Division of tower 1/A into two columns*

Tower 1/A is cut into two columns for the reduction of dynamic effects. Tower 1/A/a remains on the existing foundation together with tower 1/B while column 1/A/b formed by the cut off half is erected on a new foundation creating thereby twin towers (see Figure 12).

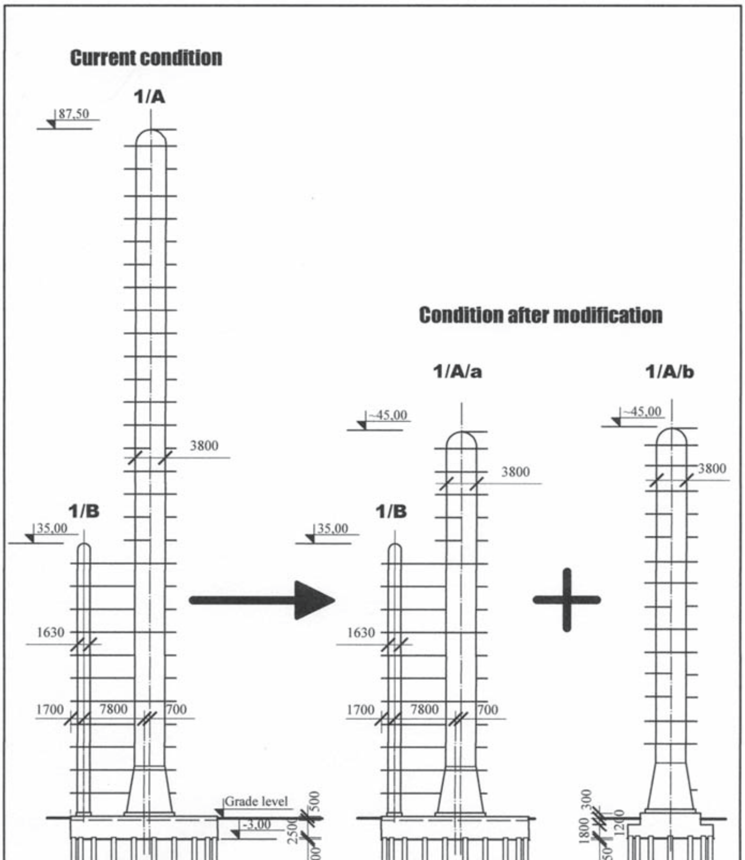


Figure 12. Modification of tower 1/A into two columns

4. SUMMARY

In the design of tower-like constructions the stresses arising parallel with and perpendicular to the wind direction due to the dynamic wind effects must be investigated in all cases.

A vibration dampener or baffles have to be included in the design already at the time of construction if necessary.

If the reduction of dynamic effects is achieved by a solution resulting in additional loads/stresses acting on the foundation, then the strengthening of the foundation may become necessary.

The rise and fluctuation of the ground water level occurring with the passing of time must be taken into account for the design of foundations.

The solution chosen for reducing the dynamic effects acting on tower 1/A according to the comparison of the potential solutions determined on the basis of the tests, analyses and calculations performed was the installation of a pendulum swinging in opposite phase with the tower.

The resultant displacement measured in recent years was 1.0 to 5.7 cm. In the more than five years elapsed since the installation of the pendulum the continued observance of the tower has verified the effectiveness of the solution chosen: the large deflections of the tower observed previously have been eliminated and the tower remained unchanged and not dismantled.

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