

EXAMPLES FOR APPLIED DIAPHRAGM WALL TECHNOLOGIES

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Abstract

Building up vacant house-row plots in Budapest, on the left Danube-side and in the complicated inner city area. Bringing to a stop the tilt of an 80 m high reinforced concrete chimney-stack and rectification thereof into vertical position. Building up vacant house-row plots. Rectification of a tilt chimney-stack into vertical position.

Introduction

Deep foundation structures have to fulfil sometimes water proofing requirements beside their structural adequacy. A progressive development appeared in the past decades in the field of diaphragm wall technologies of which an extensive and frequent application can be observed among other deep foundation structures.

One frequently emerging field for diaphragm wall construction is the building of edifices on abandoned vacant plots in the row of houses. Characteristic for such an action is to build a new building amongst two one in the row — mostly to deeper foundation depths — whereby a strictly limiting factor is to protect the safe stability of the old houses.

In the internal downtown districts of Budapest one can find a lot of similar available grounds (due to the damage caused by World War II). Prices of these plots have increased extremely in the recent years because infrastructural installations are mostly available on the spot and the new houses might be used for any desirable purpose.

We were used to experience the growing of houses towards the sky, but nowadays demand has increased for spacious underground facilities.

Foundation, Supporting of Working Pits, Water Proofing

In most cases the condition of the foundation for the old houses on the border line of an empty plot is rather unfavourable. To the contrary, new buildings might be founded in a favourable manner by using deep slab or stripe, or block foundation methods on the deeper lying load bearing basic soil. Other deep foundation methods are sparcely applied, say, due to caused vibrations.

In technical terms the core of the problem arises where the walls of the working pit have to be supported, as this is the most demanding task in deep engineering practice, whereby the 'horizontal' pressures have to be balanced appropriately. In the endeavour to make use of the most available area, only vertical pit walls can reckon with appreciation, with steep slopes only in the vicinity of the ground surface. The slopes need always reinforcement either by retaining walls, shotcrete, or anchoring pegs.

An economical contract can be achieved only where the supporting structure of the working pit fulfils multi-functional purposes: it has to accept and carry as surface loading the various forces from the neighbouring buildings, it has to be continuous to take up earth- and hydrostatic pressures and has to transfer the superimposed loads of the new edifice to the underground. All these impacts have to be accommodated in a manner to restrain both vertical and horizontal displacement in the range of allowed and designed limits. Stress and strain requirements have to be treated properly even in the case of multi storey deep spaces, as in the cellars of public buildings, in deep garages, subterrain passages, etc.

In absorbing horizontal forces also injected and prestressed anchorage may gain role. This ought to be avoided, however, above a certain depth, in the view of time loss, expenses and sophisticated technology.

Following examples — extracted from the recently accomplished experimental practice — will shed more light on the actual approach of these technical problems.

A. The Water Engineering Construction Co. was the contractor in the completion of a typical supporting diaphragm wall structure for the HYATT Budapest and the FORUM hotels on the left side bank of the Danube in the downtown city district. The same company has been commissioned by the Austrian patron to make the executive plans for the foundation.

Two storey deep cellars have to be built below the ground level by using the novel technology of a supporting diaphragm wall. 80 cm wide prefabricated reinforced concrete elements and waterproof injected mortar finish were used to produce the diaphragm wall structure which had been

keyed in into the impermeable Kiscelli clay in the depth. From inside the cellars walls have not got any extra insulation — with the exception of the finishing torcrete wall cover — despite the frequently repeating 5 to 6 m water head over the floor level of the deep garage. Eventually seeping or oozing water can be collected in the cellar in grated gutters and discharged from sumps by the help of automatic pumps.

At the HYATT hotel the diaphragm wall had to be designed to carry different pressures from various superimposed loads, from varying water heads and from altering soil strata. The maximal vertical load attained at places 569.5 kN/lin.m, or 852.0 kN/m. Stress conditions have been calculated by using the conventional Blum method.

For gaining control over the vertical and horizontal stresses and strains, two-directional loading tests have been designed and carried out by the use of a vertical anchoring method (see Fig. 1).

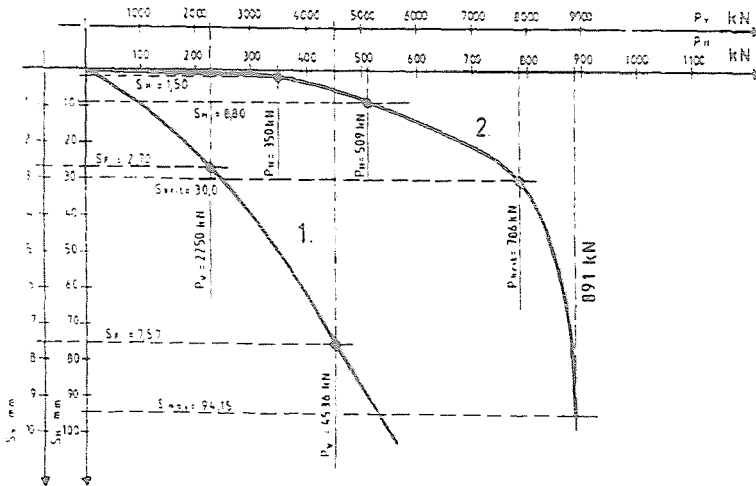


Fig. 1. Results of probing test

- 1. vertical probing p_v vertical loading s_v vertical settlement
- 2. horizontal probing p_H horizontal loading s_H horizontal displacement

Cellars and garages in the multi-storey depths were exposed to the action of the ground-water already in the construction period, so, the problem to prevent the influx of the ground-water had to be solved both in the design and the construction phase. Having a permeability coefficient of the water-tight diaphragm walls in the range of $k = 10^{-8}$ cm/sec and because the wall sections reach down to and are keyed into the Oligocene Kiscelli clay, the water was excluded from the working pit as soon as the

diaphragm wall was completed. The water from the enclosed space had to be exhausted then by the help of locally arranged temporary pumping wells.

There was no need to apply any type of insulation onto the inside cellar walls at the two hotels, because the materials of the incorporated reinforced concrete elements and the torcrete sheet are both impermeable in themselves. If any dewatering would eventually be needed, this was provided for the HYATT hotel by automatically operating pumps (*Fig. 2a* and *2b*).

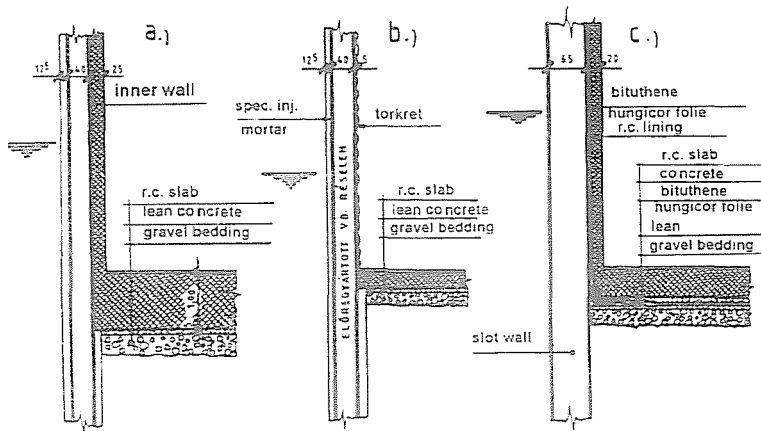


Fig. 2. Structural schemes

a) Forum hotel b) Atrium Hyatt hotel c) East-West Office bldg.

B. Foundation plans and the execution of work for the East-West Estate Office Building (opposite to the Astoria hotel in Budapest) were accomplished by the Water Engineering Construction Co. The monolithic reinforced concrete diaphragm wall structure had to take up both horizontal and vertical dead loads from outside and fulfil the requirement of ground water resistance.

A single sheet insulating bithuten coat had been adhered from inside to the diaphragm elements for providing water-tight conditions even in the case of the highest ground-water level position. For the protection of the insulating sheet a reinforced concrete lining wall of 15 to 20 cm thickness has been built in the inside space of the cellar, with structural conformity to the monolithic diaphragm structure (see *Fig. 2c*). So, the once shuttered internal cellar wall offers an aesthetic appearance and is absolutely safe for water tightness. Owing to the contribution of the diaphragm wall foundation, no sign of a differential settlement could have been observed in the

overstructure of the eight storey high building. Construction of the foundation was a quick affair and also the bithuten insulation has not caused unbearable expenses.

The foundation for the inside loads in the building consisted of a reinforced concrete slab supported by slotwall type piers on the locations of the inside pillars.

Some Conclusion Can Be Drawn from the Two Cases Above

Construction activity on the vacant plot sites in the city and the structural complexity of deep engineering tasks became ever harder in recent times in the course of continuously increasing demand. From building a slab as foundation and from underpinning the foundation of old buildings on the neighbouring plot, we have come to support the working pit walls with stressed and injected anchors in one or in several rows.

Free standing vertically cantilevered keyed in reinforced concrete walls are capable to support two storey deep earth walls to the most. Where we have to support three or more storey deep working pit walls, we have to consider to add some special structural element, or to apply particular building technology to restrain horizontal earth-, or hydraulic pressures.

Where we have to erect edifices on vacant plots, two, or sometimes three sides of the vertical walls of the working pit are loaded by the weight of the buildings in the close vicinity. Here, the horizontal forces might be balanced with prestressed and injected anchors, or by the so-called 'Milanese' method, whereby the floor levels are supported separately.

Due to given depths, in most such cases also the necessity arises that beside the load bearing capacity, the supporting structure should possess the capacity to provide for watertight conditions as well.

Technical progress seems to accept generally the tendency toward developing reinforced concrete structures. Primarily diaphragm type slot wall technologies seem to gain success, either in the form of prefabricated elements, or as monolithic structures. Reinforced concrete piles, or steel sheet piling methods are, however, also commonplace methods, but sometimes with restrictions.

C. A selected example for the demonstration of the above mentioned considerations might be offered by the deep engineering work that had been carried out at the I. B. C. Office Building in the Üllői street in Budapest. Two storey deep garage was designed below the ground surface and a monolithic reinforced concrete, water-tight, load bearing supporting wall for the delimitation of the working pit. The structure had to carry the loads from the adjacent old buildings as well.

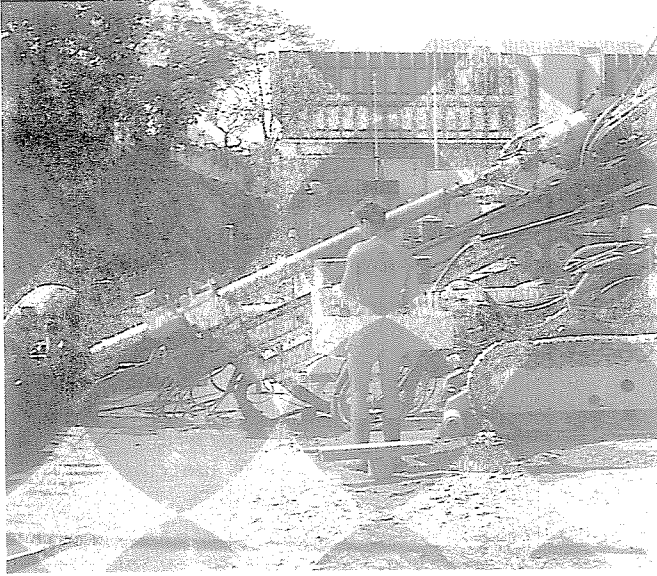


Fig. 4. Preparation of anchors

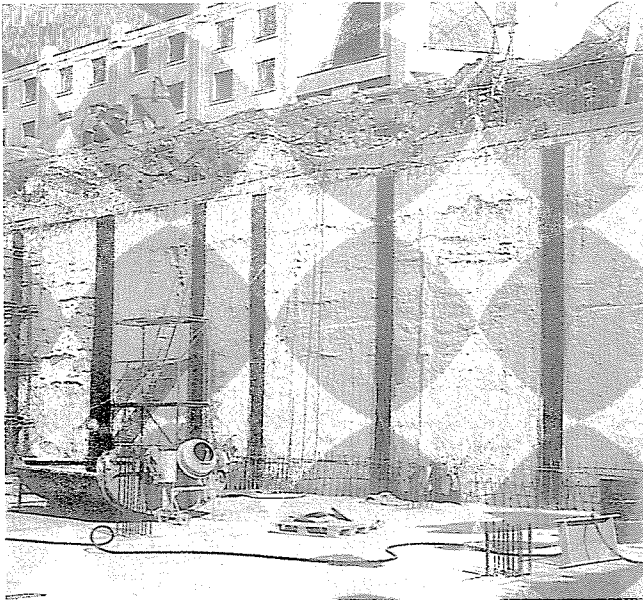


Fig. 5. Reinforced slot-wall with drains

The anchor rods were 15 m long, drilled in with 20 degree inclination to the horizontal (*Fig. 5*). Each rod had to carry a force of 800 to 900 kN.

An Extremely Complicated Engineering Job

Strengthening the foundation of an 80 m high tilted chimney and rectification thereof into the lead line.

The progressive tilting movement of an 80 m high reinforced concrete chimney of tapering cross-sectional area and 3.8 m diameter at the bottom (having been made by the sliding form method) has been first observed in 1980, at a plant of a factory of vital importance. Surveying registered a travel of the peak in the range of 30 to 50 mm per month, what meant that in about two more months time the chimney had to tip over with serious consequences in the neighbourhood and with marring the operation of the plant.

Direction of the tilt coincided with that of the dominant wind. Expert reports originated the cause of the unfavourable displacement primarily to a significant upheave of the ground-water level accompanied with the deterioration of the soil properties and the diminishing load bearing capacity of the underground below the circular reinforced concrete foundation slab.

In accordance with the suggestions in the expert report, the patron has decided to have the foundation strengthened, to readjust the chimney in the vertical position and to have the slightly corroded body of the chimney surrounded by a reinforced concrete jacket wall up to +55 m elevation at the outside.

On a competitive bidding basis the work has been awarded to the Water Engineering Construction Co., together with the commission to finalize the executive plans.

The Substitutive Foundation Structure

In contemplating the given task, it had to be taken into account that the strengthening of the foundation should be capable to carry the additional weight of the proposed jacket over the corroded body of the chimney up to +55 m elevation and to enable the positioning of the chimney into the vertical. So, the substitutive structure had to match the increased load and to respond to the requirement to protect the stability of the edifice despite the awaited strenuous effort caused by the rectification.

Any beneficial influence of the existing reinforced concrete circular foundation slab had to be omitted from the structural design of the new base, as — due to the unbalanced restricting conditions — the earth mass

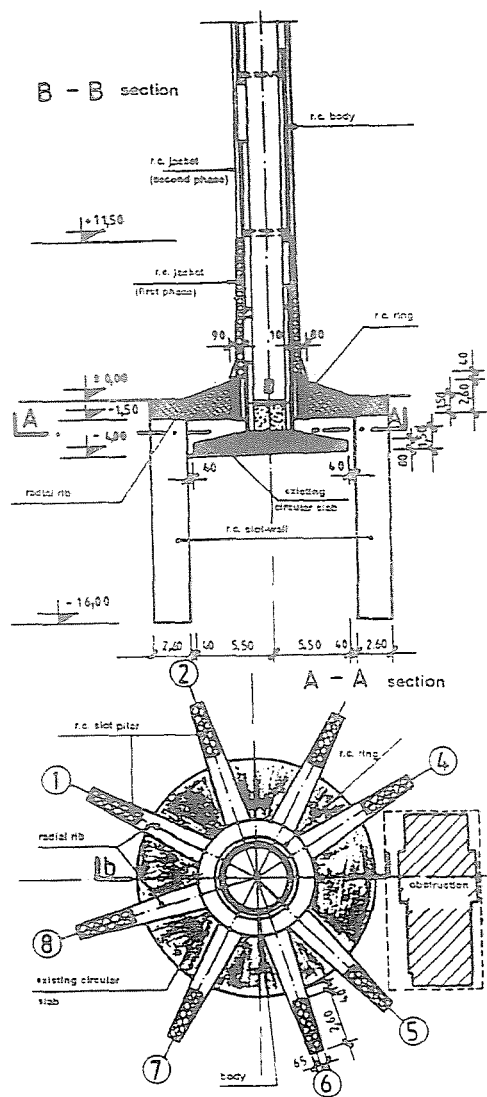


Fig. 6a, 6b. Substitutive found. system

above and next to the existing base had to be left intact. Namely, a wind storm during construction could have caused the complete deterioration of the chimney in the wake of such an interference.

In response to the above determined requirements the substitutive reinforced concrete structure consisted of a circular ring around the body of

the chimney, in which radially arranged reinforced concrete ribs of tapering cross-section were keyed in, supported from underneath by reinforced concrete diaphragm type pillars. The whole arrangement can be seen in *Figs. 6a and 6b*.

Structural Considerations: the Temporary Clamping Device

Work for the substitution of the old foundation has begun with the erection of the temporary clamping equipment with the aim to care for the stability of the chimney and to enable the correction thereof into the vertical position. One of the main requirements to be complied with was to avert any additional loading of the damaged reinforced concrete base. It was also an aspect to arrange or locate the elements of the temporary equipment in a tight space, whereby these would not impede the forthcoming activities.

A schematic view and main geometric particulars of the temporary clamping arrangement can be seen in *Fig. 7*.

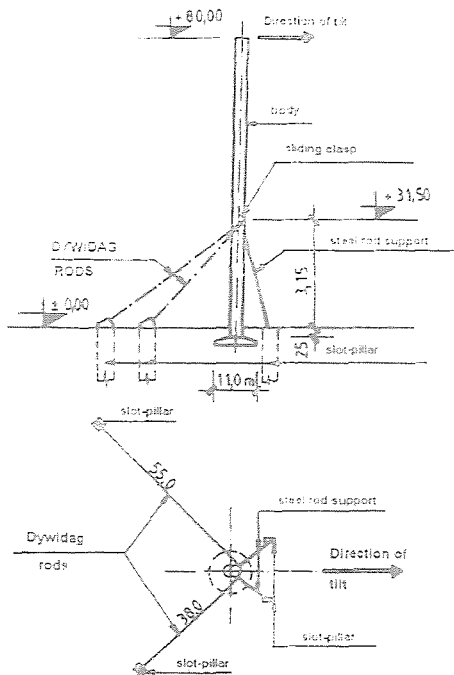


Fig. 7. Temporary clamping arrangement

The pulling rods of the Dywidag type joined up to the steel made clasping ring on the chimney at 31.5 m elevation with a certain tolerance for vertical displacement. This sliding clasping ring was also supported with a pair of jointed steel bars in the skew position, resting on a diaphragm type pier outside the working area. Stressing of the ribbed and threaded ST 85/105 type Dywidag pulling rods of 36 mm diameter took place through hydraulic presses mounted onto diaphragm type foundation blocks located to the available spots in the yard. Stressing forces were controlled and adjusted permanently during the work.

Diaphragm Type Slot-pillar Foundation for the Substitutive Foundation System

Design stresses for the substitutive foundation system were calculated in accordance with the loading forces given in the valid Design Code. In the central axis of the chimney the vertical component of acting forces was $N_{ke} = 9040$ kN on the surface of the existing reinforced concrete circular slab. In the plane of the tilt the bending moment was $M = 20400$ kNm (as calculated from the wind load and the surveyed displacement) and the shearing force was $Q = 250$ kN.

In the calculation of design stresses for the substitutive slot pillar foundation blocks also the additional loads from the proposed jacketing cover (of reinforced concrete with 0.5 m thickness in the average) on the chimney body were reckoned with the result of $N_{ko} = 4100$ kN.

Having assumed the possibility that a partial disloading of the circular foundation slab may occur during the process of rebalancing the chimney in the vertical position, 50 per cent of the slab's weight ($N_a = 1400$ kN) was also taken into account in the calculation of vertical forces.

This way, the following forces were reckoned for the mostly stressed slot-pillar foundation in the substitutive structure: normal force: $N = 14540$ kN, bending moment: $M = 20400$ kNm, shearing force: $Q = 250$ kN.

Symmetrical position could not be selected for the radial slot-pillar foundation blocks of the substitutive structure because of an obstructive foundation block of the ventilating plant in the vicinity (see *Fig. 6b*).

In contemplating the burden being taken by the mostly stressed slot-pillar in the direction of the tilt, it was assumed that the adjoining load transferring elements (like the radial girders and the reinforced concrete radial ring) had an utmost rigidity, so, for the mostly stressed pillar the calculated normal force was $N_1 = N_{\max} = 2650$ kN and the shearing force was $Q_1 = 31.25$ kN.

Calculated on the basis of actually encountered soil physical properties, the slot pillars of $2.60 \cdot 0.65$ m cross-sectional area and -16.0 m depths were capable to carry also the forces on the mantle surface in respect of both stresses and strains.

Each slot pillar for the substitutive structure was constructed individually, i. e. one in time, to its completion. To prevent the soaking influence of the applied slot fluid, in the vicinity of the existing circular foundation slab, in 0.4 m distance from it, cylindrically shaped steel sheet panels were sunk to cover the endangered side between -1.5 and 4.5 m depths below ground surface.

The reinforced concrete slot pillars were made of C. 16 grade concrete and B 50-36 type steel bar reinforcement by using tremie for underwater technology.

Reinforced Concrete Ribs of the Substitutive Foundation System

Loads from the chimney in its finally rectified vertical position were supposed being transferred to the slot-pillar foundation blocks by radially arranged reinforced concrete ribs of tapered cross-sectional areas. The geometric scheme of the arrangement can be seen in *Fig. 8*.

Each rib was constructed individually (one in a time). After having positioned the premounted armament for each rib, the weight of the excavated earth material was immediately refurnished by partially refilled concrete. In the region of the ground surface and at the interface to the reinforced concrete circular ring toward the chimney's base, permanent joints were inserted. The ribs were completed after the fitting of the armament of the circular ring.

The Reinforced Concrete Circular Ring

With a gap of 0.1 m thickness from the body of the chimney (filled up by Hungarocell plastic sponge) a reinforced concrete ring of 0.8 m width and 3.5 m height was constructed. The filling was disposed later and replaced with cement mortar, after the rectification of the chimney and the final completion of the work.

Stresses in the ring were calculated in an approximative manner, as a consequence of the asymmetric arrangement of the ribs and the developing uncalculable various forces in them during the rectification process.

Before having the ring concreted, Dywidag rods in protective tubes and the relevant load distributing plates for the pertinent presses were

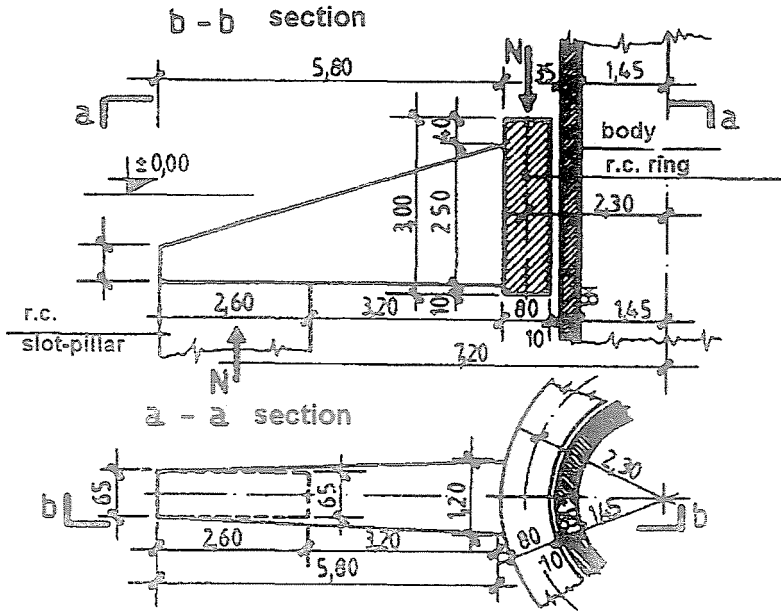


Fig. 8. Scheme of the radial girder and the ring

anchored in position for later application during the process to rectify the chimney in the correct vertical position and for the connection of the body of the chimney.

The Jacket around the Body of the Chimney

After the construction of the reinforced concrete structure for the substitution of the old circular foundation slab, a reinforced concrete has been built to cover the body of the chimney up to elevation 10.5 m. This part of the jacket has been utilised in the rectification process in transferring vertical forces onto the reinforced concrete body of the chimney. The conical jacket wall had a thickness of 0.5 m at the upper surface of the reinforced concrete ring and 0.3 m at elevation 11.5 m.

Connection between the jacket and the body of the chimney was provided by B. 60-40 type steel pegs of 20 mm diameter, arranged in a 0.25 m grid on the surface of the cleaned body of the chimney, where 20 cm deep holes were drilled for sticking the pegs into. Also a coat of emulsified sticking material was spread over the surface of the chimney to glue the two bodies to each other.

The bottom part of the jacket with doubled rows of reinforcement housed the compartments for the hydraulic presses. Between the lowest plane of the jacket and the uppermost plane of the reinforced concrete ring a gap of 50 mm height has been left open for inserting the steel plate wedges according to necessity.

Rectification of the Chimney into the Lead Line

The correction of the tilted chimney into the vertical has been accomplished by the help of hydraulic presses acting on the tilted side between the newly completed foundation and the jacket wall and by the contracting forces aroused on the opposite side by the incorporated Dywidag rods.

This manoeuvre took place in three phases. In each phase the maximal allowed returning movement of the chimney; this movement corresponded to 10 mm settlement at the base, in the plane of the tilt. A few days had to elapse between the phases to allow for the consolidation of the soil.

Significant role has been devoted for precise surveying and levelling technics inasmuch all relative movements of the jacket wall (settlement, upheave, etc.) were measured in the plane of the distortion. Eight reference points were measured simultaneously on the upper plane of the reinforced concrete ring. The adjusting equipment has continuously followed the movements of the chimney.

The whole rectifying process lasted for 18 days, including the elapsed intervals for the development of consolidation. The manoeuvre was finished when the axis of chimney coincided with the theoretical lead line at the 80 m elevation.

References

- NAGY, J.(1992): Foundation Solutions of Some Edifices Built up on Vacant House-row Plots during the Recent Decades in the Capital. (In Hungarian). *Közlekedés és Mélyépítéstudományi Szemle*, Vol. 9, pp. 346-351.
- BANCSIK, CS. - JUHÁSZ, B. - NAGY, J.(1992): Foundation Strengthening and Rectification to the Vertical Position of an 80 m High Reinforced Concrete Chimney-Stack. (In Hungarian). *Közlekedés és Mélyépítéstudományi Szemle*, Vol. 10, pp. 370-380.