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FOUNDATION STRENGTHENING AND GROUTING BY MEANS OF JET PILES FOR A 9+1-STOREY BUILDING WITH STRENGTHENED-CONCRETE FRAMING

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Abstract

The reinforced concrete frame building on the area of 17×150 sq.m consists of a cellar, ground floor and $9 + 1$ storeys. The pillars were founded on solitary reinforced concrete blocks. The given vertical supporting system is horizontally connected by flexible, reinforced concrete sheets for ceiling. This configuration was not adequate to withstand, or balance the ensued uneven settlements arisen by the response of the alternating subsoil under the different loads from the pillars.

At frames 14. to 27., however,– which were founded on weak silty fine sand of varying thickness (0.5 to 3.0 m) – the safety factor against failure was calculated to $n = 1$, instead of $n = 2.77$ that had to be prescribed.

The goal was to transfer the loads from the silty fine sand onto the Kiscelli clay, which has excellent bearing capacity at 5 to 6 m deeper levels.

The first part of the reading to be presented deals with the unavoidable geotechnical exploration, laboratory testing, determination of the intermittent and final time dependent parameters of the jet-propulsion work, the evaluation of the in situ bearing capacity trials and, of the strength analysis of the undisturbed samples having been taken from the jet piles. The second section informs about the design and accomplishment of the work, and concludes with the settlement monitoring results that verified the success.

The measurements proved that the settlements stopped at the prescribed limits and, the reckoned safety factors against failure widely surpass the $n = 2.77$ value required by the standard for this case.

Keywords: foundation strengthening, jet piles.

1. Summary

The building 16–18 at October 23. street in Budapest (Hungary) on the area of 17×150 sq.m consists of a cellar, ground floor and $9 + 1$ storeys, is 33 m, respectively 36 m high with the conference room on the 10^{th} floor. Speciality of the reinforced concrete frame building is that the four rows of columns for the three alleys of storeys 1 to 9 join up on the ground floor to two rows of blade-pillars through wall supports on the first floor. The pillars were founded on solitary reinforced concrete blocks. The given vertical supporting system is horizontally connected by flexible, head girder-less, reinforced concrete sheets for ceiling. This configuration was not adequate to withstand, or balance the ensued uneven settlements arisen by the response of the alternating subsoil under the different loads from the pillars. The foundations of the building sunk continuously, and even worse, differently since the building was inaugurated and its usage begun in the year of 1979. (Next to the western side of this building, the house was handed over for the residents in 1982.). Cracks on the walls, breakage of fittings and tightening of doors demonstrated the misfortune. Due to these miserable conditions jurisdiction commenced and the authority ordered uninterrupted surveillance with the monitoring of settlements. Measuring the settlements started unfortunately only in 1988 when the most important period of movements was already over. The settlements continued as long as 1996 when the improving rehabilitation work was finished. In 1994 the intensity of settlements begun to increase, therefore the experts Dr. B. Juhász, Dr. I. Halász and Dr. L. Szerémi, who investigated the superstructure of the building stated in their report, that the building is unable to endure any further additional settlement. Thereafter the careful underground exploration and soil testing has began in the laboratory. The outcome was that the safety factor for frame sections 1 to 13 were acceptable, because the foundations there rested on gravel. At frames 14 to 27 however, – which were founded on weak silty fine sand of varying thickness (0.5 to 3.0 m) – the safety factor against failure was calculated to $n = 1$, instead of $n = 2.77$ was had to be produced according to the relevant standard for such constructions. In addition, the measured settlement on the street side has not attenuated to the least by that time.

The goal to be attained was to transfer the loads from these latter foundations with planes on the inadequate silty fine sand onto the Kiscelli clay of excellent bearing capacity at 5 to 6 m deeper levels than the actually existing level was. In the meantime it was prohibited to let the existing bearing capacity still further, to exceed 15 mm additional total settlement at the end of the improving work, and to surpass 3 mm differential settlement among the new foundations.

The first part of the reading to be presented deals with the unavoidable geotechnical exploration, laboratory testing, determination of the intermittent and final time dependent parameters of the jet-propulsion work, the evaluation of the in situ bearing capacity trials and, of the strength analysis of the undisturbed samples having been taken from the jet piles. The second section informs about the design and accomplishment of the work, and concludes with the settlement monitoring results that verified the success.

Thus, the performed improving work has gained its aim, inasmuch the still ongoing measurements proved that the settlements stopped at the prescribed limits and, the reckoned safety factors against failure widely surpass the $n = 2.77$ value required by the standard for this case.

2. Data, Antecedents

The building at number 16–18 23 October street has a basic area of 17 × 150 m, it consists of the cellar, the ground floor, and 9 + 1 stories, its height is 33 meter, and 36 meter at the heightened 10-storey conference hall. The special feature of the reinforced-concrete framing of the building is that the transverse three-winged four pier rows of the 1–9 stories assemble by means of wall joists built-in the first story in two ground-floor blade pier rows, being founded on separate solitary reinforced-concrete foundations. This vertical structure system is only connected by the flexible reinforced-concrete flat slabs without heads, which can neither balance nor counteract the foundation settlement motions originating from the loads of different extent transferred to the piers and from the dissimilarity of the subsoil. The building is stiffened against horizontal loads by the reinforced-concrete walls of the lifts. At the piers 14, 15 of the superstructure of the building dilatation was designed and built in but this was not continued across the foundation.

The building was built between 1972 and 79. In the year 1973 one part of the structure tumbled down. Then, the pillar heads were strengthened with mushroom heads of steel structure, and further reinforced concrete stiffening walls were built in. After re-planning and strengthening, the building was finished in the year 1979. *Fig. 1* shows the structural framing of the building.

Fig. 2 shows the soil strata along the longitudinal axis of the building relaying on data of the subsequent soil explorations accomplished for the strengthening design.

The clay of Kiscell of high loading capacity appears in a depth of ∼9 meters on the 95 mAf (meter above the Adriatic Sea) level. Above this level yellow small gravel is to be found in changing layer thickness.

The upper level of this is near the foundation level (100.70 mAf.) from the Budafoki út down to the breaking point in the ground-plan of the building, however, on the western end of the building the upper level of the small gravel is deeper, near the 98.00 mAf level. Soft loose yellow sandy sludge of low loading capacity settles down this small gravel, as the top layer, that is a made-up group filling with broken building materials in 2.1–3.4 m thickness. Thus the building foundations rest upon the yellow gravel from the breaking point in the ground plan up to the building end from the Budafoki út, while they rest upon the yellow sandy sludge in ∼ 50 cm – 2.5 m thickness of low loading capacity and compressible at a high extent.

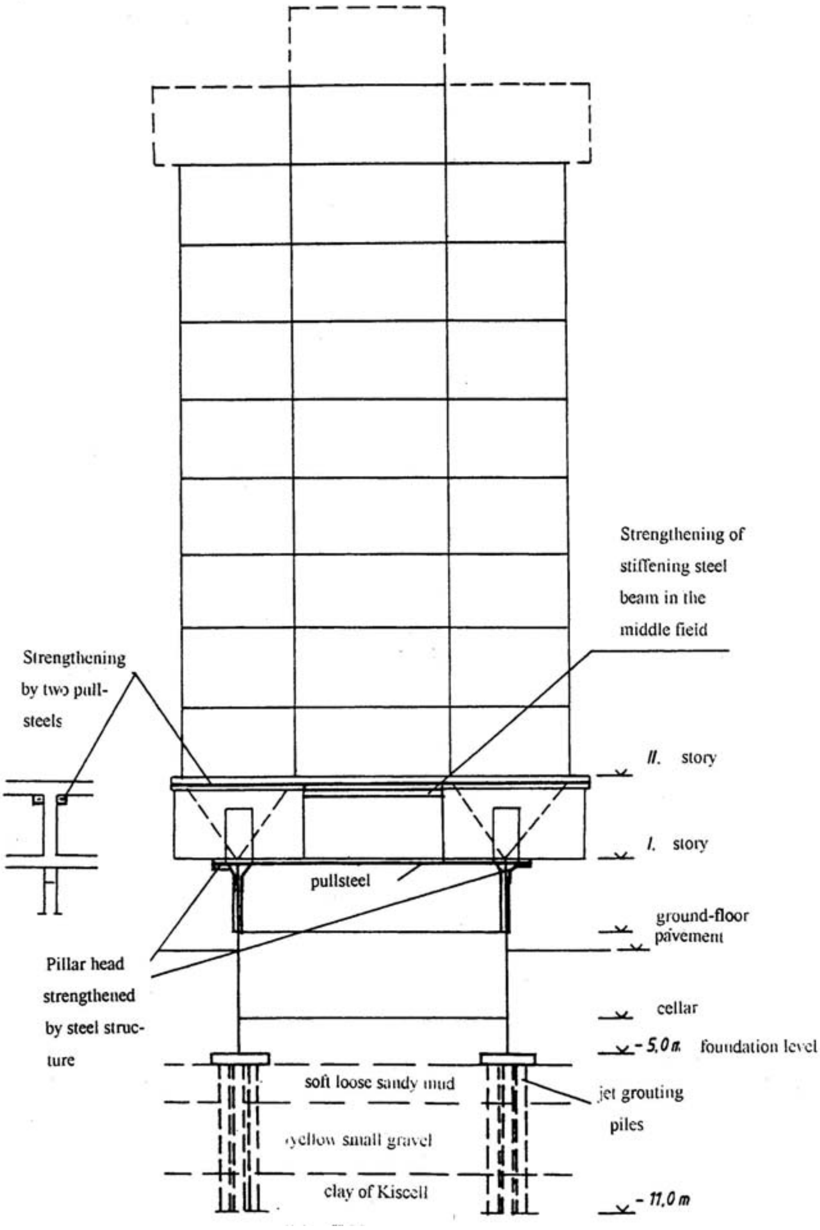

Fig. 1. Structural framing of the building

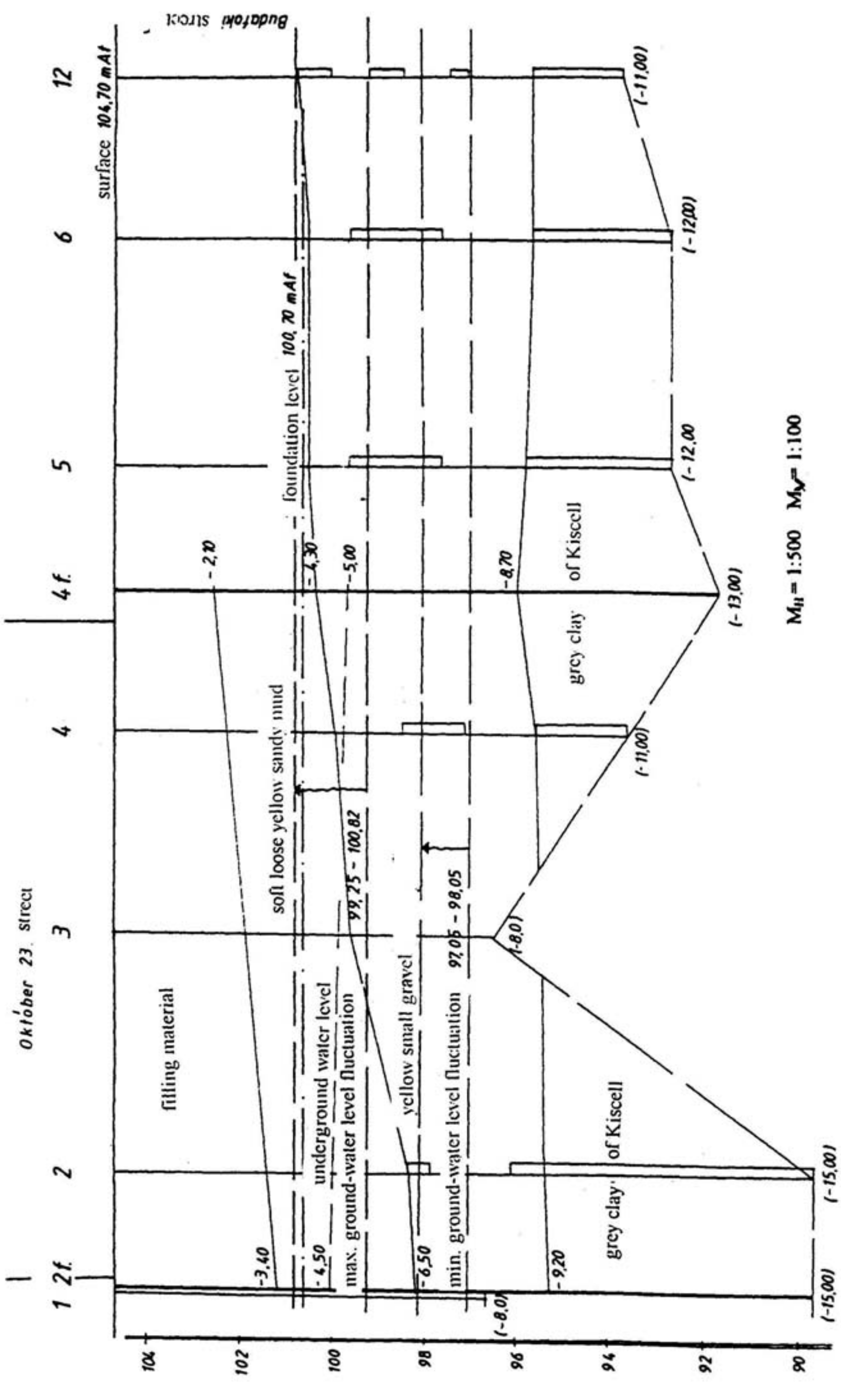

Fig. 2. Soil strata along the longitudinal axis of the building

As early as after having finally approved the building in its part lying west of the ground plan breaking cracks and door tightening movements referring to settlement have been appearing at increasing extent towards the end of the building. In the year 1982 when a residential building standing upon reinforced concrete foundation was built at the western end of the office building being also built upon the soil of sandy sludge, the signs pointing to settlement have been increased. The cracks appeared in the residential building already in 1983, then at the end from the residential building of the office building have also appeared the signs referring to the increase in deformation. This is why building settlement measuring at 8 points of the building was ordered in the course of legal proceedings instituted on account of building damages. It came to the basic measuring on July 3, 1988, when the great part of the settlement motions resulting from loads of the head office building and the residential building built next to the former, had already taken place, since the office building and the residential building were finally approved in the year 1979 and 1982, respectively.

Six further measurements have been accomplished up to October 25, 1994. Having elaborated the measured data, it can be stated that the settlement of the buildings has been increased during the whole measuring period and their intensities did not even decrease. On the basis of levelling data we have also determined the bending of the first story corridor. These data verified that settlement movements have taken place in the building gradually increasing from the frame station 12, inasmuch as in this area below the foundations the compressible sludge-and-sand of low loading capacity and $\sim$ 50 cm – 2.5 m thick is to be found. At the foundations of the official building block at the street front the settlement rate increased. The question comes to hand – what can be the reason for that the settlement of the office building has been still continued 15 years after its finished construction and 12 years after the final approval of the residential building built beside it (1994) and the settlement rate did not decrease.

Table 1. Settlement rates at the street front (mm/month)

	number of measuring points			
	5	6	7	8
1988–1990	0.044	0.033	0.050	0.067
1990–1994	0.071	0.078	0.090	0.0936

On the basis of results of the soil exploration and laboratory testing works we stated that the continuous settlement of the building, still having been going on in the year 1994, and the cracks as a consequence of that, are the aftermath of the sandy sludge to be found below the foundations, existing in a flow state and in changing thickness (0.5–2 m).

In the triaxial tests – except the specimen preloaded with the 400 kPa load of the foundations – a relevant breakage picture did not evolve. From the very beginning of the loading the specimen engaged in swelling out and this has been continued in the course of the testing. This is why we have determined the breaking value on the basis of Mohr's circles falling under the 20% vertical deformation. We have determined, however, the breaking value falling under the 10% deformation too.

We composed the following table from the results of the tests:

Table 2. Results of the triaxial tests

	$\phi°$	c kPa	σ_{breaking} kPa	σ_{measured} kPa	n safety
$\varepsilon < 20\%$ deformation open system	24	23	11911	413	2.88
$\varepsilon = 20\%$ closed system depending on effective stress	28	0–69 very variable	906	413	2.1
$\varepsilon = 10\%$	16	0	423	413	~ 1.0

The settlement movements having run their courses till now fall nearly under the 10% specific deformation. So the state of the year '94 shows a safety of about $n = 1$ and the sandy sludge under the foundation was really in a flow state. Although it's true enough that the more deformations and settlements increase the more consolidated sandy sludge but the sludge strength changes significantly only about 20% deformation. However, the building can not bear a deformation of such extent, in special consideration of great differences in settlement caused by the inclined deposit of the sandy sludge.

The question comes to hand: is there any role of the level fluctuation of underground water level which was in a 15 years' average 2.4 m/year and that fluctuation has been in the soil stratification just under the building foundations enacted.

We have considered this process in a large-size consolidation apparatus. We have built in undisturbed sludge specimen upon the sand-and-gravel stratum below the sludge and loaded with 4 kP/cm^2 load corresponding to the building load. As for the sand-and-gravel, we have changed the water level in it. Having passed a 5-day loading the specimen let us see the push-in of the sludge into the sand-and-gravel, however, elution owing to water level change was not observable. We stated that the sludge stratum pushed into the sand-and-gravel because of the building load but the fluctuation of the ground water level was only able to cause elution at a small extent and so was the settlement.

We estimate this value to be 1–5% of that of the total value of settlement.

Dr. László Szerémi, Dr. István Halász and Dr. Bertalan Juhász have fulfilled the control of the building superstructure and stated that the building superstructure is not capable of standing further settlement, moreover, for the sake of assuring the adequate safety the superstructure has to be strengthened.

On the basis of the strength and deformation tests performed, related to the foundations, we established that the breaking safety of the foundations from the frame station 12 up to 27 of the office building was not satisfying, the relevant stresses approach to a great extent the breaking stress value. In consequence of that at that building part the settlements have also been in progress in the year 1994 and their intensity did not decrease. Thus the foundation strengthening in the suggested section is by all means necessary for the sake of safety against breaking of the building, the assurance prescribed by MSZ, and stopping the continuous settlement, in particular in such a way that the settlement caused by that reason the minimum should be, the safety should not decrease, and the building should not cause further change in bending. That is to say the strengthening works have to be effected without taking up the floor concrete above and below the foundations and earth excavation up to the basic level. For the sake of safe work performing the strengthening of superstructure has to take precedence over the foundation strengthening works. Figure 1 shows the summing up of the superstructure strengthening.

We describe one of the three solutions for the foundation strengthening which has been executed.

3. Determination of Cement Feeding, Pile Strength and Diameter of Experimental Jet Piles

The sandy sludge soil under foundations can be well grouted by means of high-pressure injection (jet grouting). The essentials of the process are, that the soil is disintegrated in aqueous soil slimes by means of high-pressure – 200 – 500 bar – water jet, coming from a central injection hole, having a diameter depending on the soil quality, then cement is mixed through a direct connecting valve into the soil slimes, by means of cement mortar jet, at a pressure of about 20 bar. The mixing is to be assured by the rotating and axial motion of the injection rod. The solidification process of the cement soil slimes is the same as that of the grout setting solidification. The final strength of the grouted soil depends on the soil. Expected values of the 28-day strength in the sandy sludge $\sigma_t^i = 5.0$ MPa, in the sand-and-gravel $\sigma_t = 15$ MPa. The grouting-hole sequence has to be chosen that at the same time under one foundation body cannot be more than a single hole in which non-solidified cement soil slimes take place. In this case, according to our calculations, while effecting the grouting the solitary foundations settle max. 2–3 cm as it is to be expected. By means of correct choosing the injection sequence, however, the settlement differences between the neighbouring foundation bodies can be maintained at a value of ≤ 1 cm. For the sake of reliable determination of data required for the design of the foundation strengthening by grouting test jet making was effected in the courtyard near the building.

We wanted to establish in the course of the pre-tests the following:

- Increase in solidification and rigidity of cement treated sandy sludge as a function of time
- Cement feeding required for achieving the 28-day 5000 kPa uniaxial compression strength of
- Expectable diameter of the jet pile.

We prepared soil exploration boring at the place of test jet making and established that the soil stratification is similar to that of the soil section made for strengthening.

The execution firm designed a 450 kg/bore running meter cement feeding and gave 80 cm for the expectable pile diameter. The voids in the sandy sludge are $n = 40\%$, so one bore running meter contains 795 kg soil. In the case of the designed 450 kg/running meter cement feeding the weight ratio of cement soil is $450 : 795 = 0.57$.

Taking into consideration the water content $w = 25\%$ of the soil we have the following composition of the soil grout used for the laboratory tests:

371 gr soil
225 gr cement
336 gr water
932 gr

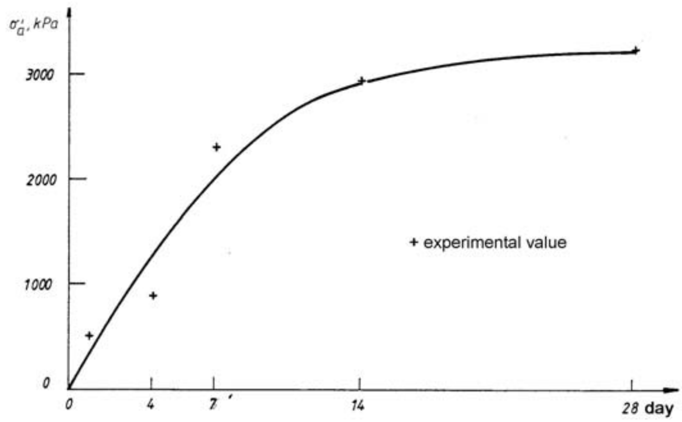

Fig. 3. Uniaxial compression strength depending on time

The cement weight ratio in this mixture $225/371 = 0.61$ is nearly corresponding to the ratio (0.57) that develops when the voids in undisturbed soil specimen are filled up by cement. We determined the uniaxial compression strength of the specimens at ages of 1; 4; 7; 14; 28 days. We crushed 6–6 specimens in each period. Excluding imperfect specimens containing air bubbles we summed up the average

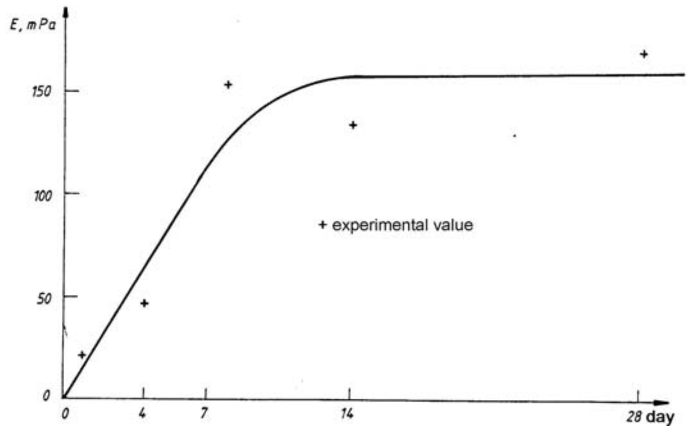

Fig. 4. Modulus of elasticity depending on time

values of strength and modulus of elasticity in the function of time in *Figs.* 3–4. We used for calculating the moduli of elasticity the values of specific deformation belonging to the stress amounting to 70–80 % of the average compression strength.

According to MSZ 15004-89 the present compression stress of the ungrouted sandy sludge below the foundation is as follows:

$$\phi = 16^\circ;\ c = 10 \text{ kPa; foundation surface } B \cdot L = 3.5 \text{ m} \cdot 4.5 \text{ m;}$$
$$\gamma_n = 20 \text{ kN/m}^3;\ t = 1.5 \text{ m;}$$
$$N_B = 1.53;\ N_t = 4.34;\ N_c = 11.69;\ a_B = 0.74;\ a = 1.37;$$
$$\sigma_t = 0.74 \cdot 20 \cdot 1.53 \cdot 3.5 + 1.39 \cdot (1.5 \cdot 204.34 + 10 \cdot 11.63) = 79.3 + 342.63 = 422 \text{ kPa;}$$

so the uniaxial compression strength

$$\sigma_{ny} = \frac{2 \cdot c \cdot \cos\phi}{1 - \sin\phi} = \frac{2 \cdot 10 \cdot \cos 16^\circ}{1 - \sin 16^\circ} = 26.5 \text{ kPa.}$$

We can state the following on the basis of the result of strengthening as the function of the time, to be seen in *Figs.* 3–4:

- The aimed uniaxial 28-day 5 MPa compression strength can not be achieved even by means of 450 kg/running meter cement feeding under laboratory circumstances. Still less, in reality, namely one part of cement gets also away together with the overflowing mortar. So the cement will be certainly less in the jet 'column' than 450 kg/running meter.
- The solidified sludge at 10 days of age reaches $\sim$ 80% of its 28-day strength and $\sim$ 90% of its rigidity (E). An important part of the strength and rigidity increase takes place so during the first 10 days.

- The uniaxial compression strength of the grouted soil is already in its 1 day of age $\sim$ 25 times higher than the strength of the soil strength without grouting ($\sigma_{ny} = 26.5$ kPa). Consequently, a decrease in load bearing capacity below the foundation can only occur in the first $\sim$ 12 hours – if it is observed that 1 jet 'column' will be prepared at most every day, in the course of the day. From the 1 day of age of the first jet column, the load bearing capacity below the foundation can but increase as long as after having finished the grouting, the strengthened soil does not reach its total load bearing capacity.
- Rigidity of the grouted soil against compression (E, modulus of elasticity) is already in 1 day of age $\sim$ 4 times higher than the 5.87 MPa modulus of elasticity of the sandy sludge. Decrease in the soil rigidity (max. 3%) below the foundation caused by the 'jet' making is to be expected in the first 10 hours after having made the first jet 'column'. Beginning from that the rigidity of the soil below the foundation against compression increases continuously as long as the grouted soil will not reach the 28-day value of its rigidity. We calculated the modulus of elasticity of the sandy sludge on the basis of the $E_s = 8.7$ MPa modulus of elasticity determined in laboratory on an undisturbed specimen assuming the $\mu = 0.33$ value. $E = 0.675 \cdot E_s = 5.9 \sim 6.0$ MPa.
- Owing to the continuous increase in the elastic and grouting properties of the sandy sludge grouted by jet making, depending on time, and the extent of its property values, settlement on the building caused by compression of the grouted sandy sludge is impossible and not to be even expected, given that the construction sequence prescribed by designs and the technology, determined for the test jet making, will be observed.
- The settlement on the building – as detailed as follows – will take place only for that reason that the building loads will be directly transferred by the grouted blocks – shallow flat fundaments – to the surface of the clay of Kiscell, so compression, settlement will take place in it.

We repeated the laboratory tests with 550 kg/running meter cement feeding also but the desired 5000 kPa designed uniaxial compression strength could not even be reached by that. Partly relaying on laboratory pre-tests 5 pieces of test jet 'piles' were made. The jet 'piles' reached down to $\sim$ 50 cm into the clay of Kiscell and ended $\approx$ 50 cm above the level of the foundation bodies in the made-up ground. The piles were made with different parameters, the constructor disclosed the cement feed among them. The test jet 'piles' were finished 24 and 25 June, 1996. The cement feeds were as follows:

A 520 kg/running meter;

B 625 kg/running meter;

C 440 kg/running meter;

D 500 kg/running meter;

E 575 kg/running meter.

The cement was one of high strength, rapid hardening Portland cement (350 min).

Core drilling specimens with $D = 45$ mm diameter were taken out of jet piles 4 and 5 July.

The core specimens have been delivered in the Institute for Geotechnics, University for Technical and Economic Sciences of Budapest (BME), where they have been stored under water up to their 28-days of age.

8 July, 1996 the jet piles marked D and E have been dug out and measured. Diameter of both of 'piles' (D and E) reached the desired 80 cm.

The Constructor took $\sim \phi 125$ mm core specimens out of the jet piles marked 'A' 'B' and 'D' and on 19 July, 1996 delivered the total bore length in the Laboratory of Institute for Geotechnics, BME, for effecting the tests on these specimens by the Institute for Geotechnics. The specimens reached their 28-days of age together with those taken out earlier from the jet piles E and D on the 22 July and then we completed the uniaxial compression and tensile strength tests.

According to the results of the breaking tests the average uniaxial compression strength of the specimens taken out of the jet pile marked 'B' approached the aimed 5000 kPa value, which had 625 kg/running meter cement feeding.

$$\sigma_{Ba} = \frac{4867 + 5133 + 4713 + 5133}{4} = 4962 \text{ kPa}$$

The value of the average splitting tensile strength: $\sigma_{ta} = 574$ kPa.

And the modulus of elasticity $E_{ba} = 198000$ kPa.

So, on the basis of the pre-tests the jet piles were made with 625 kg/running meter cement feeding.

We determined the strength parameters of the jet piles (ϕ, c) according to MSZ 13285/3-79 7.8:

$$\phi = 24.55^\circ$$
$$c = 893 \text{ kPa was obtained.}$$

We present the checking of the load bearing capacity of the grouted foundations and the calculation of the expectable settlement because of grouting by means of a standard foundation body with $3.5 \cdot 4.5 = 15.75$ sizes, the relevant load of which is 7000 kN. The jet piles of strengthening are to be seen in preparation sequence in *Fig. 5*.

4. Strengthening Design and Construction

For checking the load capacity we assume the following:

The strengthened deep foundation bears the total foundation load and transfers it upon the clay of Kiscell. This approach neglects the fact for the advantage of safety, that a very small part of the loads will always be borne and transferred upon the clay by the ungrouted soil.

We assume that the strength of the grouted block, that is the foundation body of the shallow flat fundament, equals the value obtained from the uniaxial compressive strength.

The compression strength on the surface of the clay of Kiscell is calculated from the lower values obtained for the clay in the course of the laboratory experiments, whereas we know from other investigations, on the one hand that these parameters (ϕ and c) are essentially higher, and on the other that the grouted block penetrates ≈ 50 cm below the clay surface, so the load transfer does not take place on the accidentally cracked crumbled clay surface, but it does on the clay to be considered massive and homogeneous.

Checking the load bearing capacity on the level of foundation before grouting:

Transferred 7000 kN relevant load, designed grouted surface (see *Fig.5*).

On the level of foundation $A_{as} = 5.25$ m^2.

Load transfer area on the clay surface $A_{ag} = 5.5$ m^2.

Since the jet piles reach 50 cm down at least into the clay of Kiscell, the load transfer takes place on this deeper level and the load transfer surface increases because of the $\sim 45^o$ stress extension assumed to the effect of the co-operation of clay and grouted soil. The presumable minimum value of the surface increase is the grouted perimeter multiplied by 50 cm. That is $A = A_{ag} + 5.37 = 10.9$ m^2.

The strength of the grouted soil block – foundation deepening – $\sigma_{hy} =$ 4960 kPa.

Compression load bearing capacity $P_t = 4960 \cdot 5.25 = 26040$ kN, relevant load of a single foundation block is 7000 kN.

So the safety against breaking on the level of the strengthened concrete flat foundation

$$n = \frac{26040}{7000} = 3.72.$$

For our case, the safety prescribed by the foundation standard:

$$\alpha_1 = 0.85;\ \alpha_2 = 0.85;\ \alpha_3 = 0.5;\ \alpha = \alpha_1\alpha_2\alpha_3 = 0.36125;\ n = \frac{1}{\alpha} = 2.77$$

so the safety realized against breakage is of higher extent than that prescribed by the standard. Checking the load bearing capacity in the case of load transfer upon to the clay of Kiscell that is the load bearing capacity of the shallow flat foundation.

On this level, the load bearing capacity of the clay of Kiscell is the lower one, so that is to be examined if the clay could take the loads coming from the shallow foundation, being to be borne by it with adequate safety.

The load transfer surface:

$$A = A_{as} + 0.5 \cdot 10.75 = 10.9 \text{ m}^2.$$

The compression stress of the clay on the level of load transfer, that is the level of shallow foundation, according to MSZ 15004-89, see *Fig.* 5 is:

$$\begin{aligned}
\sigma_t &= a_B \cdot \gamma_1 \cdot B \cdot N_B + a \cdot (t \cdot \gamma \cdot N_t + c \cdot N_c);\\
a_B &= 0.74;\ \ a = 1.38;\ \ B = 2.7\text{ m};\ \ L = 3.5\text{ m};\ \ \phi = 15^\circ;\\
c &= 250\text{ kPa};\ \ \gamma_1 = 22.0\text{ kN/m}^3;\ \ N_B = 1.32;\ \ N_t = 3.94;\\
N_c &= 10.98;\ \ \Sigma t_i \cdot \gamma_i = 89.31\text{ kPa};\\
\sigma_t &= 3849\text{ kPa}.
\end{aligned}$$

So the compression load capacity of the foundation on the level of the peaks of jet piles.

$$P_t = A \cdot \sigma_t = 10.9 \cdot 3849 = 41860\text{ kN}$$
$$n = \frac{41860}{7000} = 5.98$$

Disregarding from that the jet piles reach down ~ 50 cm in the clay of Kiscell and examining the load bearing on the clay surface, the following safety is obtained.

$$P_t = A_{ag} \cdot \sigma_t = 5.5 \cdot 3894 = 21417\text{ kN}$$
$$n = \frac{21417}{7000} = 3.0 \geq n_e = 2.77$$

That is to say that the safety against breakage is higher than prescribed by MSZ for this case even in the case, if disregarding from the ~ 50 cm deepening of the piles in the clay, we assume the load transfer to be taken place on the clay surface.

The building transfers its load at the parts to be strengthened upon the strata of sandy sludge and gives rise to a strain approaching the compression stress $\sigma_a = \dfrac{7000}{3.5 \cdot 4.5} = 444.4$ kPa. Taking into consideration the basic surface of 3.5×4.5 m of the foundation bodies and ~ 5 meters depth location of the clay below the foundation level, the stresses spread and only a stress of lower extent falls from the loads of building upon a bigger surface of the clay. $\sigma_{ag} = \dfrac{7000}{8.5 \cdot 9.5} = 87$ kPa, so the stress falling upon the clay surface from the building amounts is scarcely more than 2–3% of the compression stress. This situation changes after the grouting, since the significant part of the loads will be transferred directly upon the clay through the strengthened block that is the shallow foundation. Actually, that is just the destination of the grouting. The question is that the load falling upon a single foundation body from the building in what proportion is transferred through the grouted surface and in what proportion continues to be transferred through the sandy sludge.

Having finished the first jet ‘pile’ the supporting effect of the soil will decrease by 3% below a foundation body and this will give rise to minimum extra stresses. Proceeding, on the occasion of coming to the preparation of a newer jet ‘pile’, the great part of 3% load falls upon already finished jet ‘piles’ having significantly higher rigidity than that of the sandy sludge. Following this order of ideas, we can assume that after having accomplished jet making the load falling upon the ready jet columns amounts that part of the total load which was borne by these foundation surfaces before the grouting.

The total load on the foundation is 7000 kN, the basic surface of the grouting is $A_{as} = 5.25\ \mathrm{m}^2$, the total basic surface $A = 3.5 \cdot 4.5 = 15.75\ \mathrm{m}^2$. While reinforcing the soil the load transferred upon the grouting, without effect of slow deformation:

$$P_{sz} = \frac{7000}{15.75} \cdot 5.25 = 2333.33\ \mathrm{kN}.$$

We have under the foundation after the grouting a compressed column with two different rigidity values as follows:

Table 3. Rigidity values of the column

sandy sludge without grouting	strengthened soil
$A_{ni} = 10.5\ \mathrm{m}^2$ $E = 6$ MPa	$A_{as} = 5.25\ \mathrm{m}^2$ $E = 198$ MPa

During the slow deformation $7000 - 2334 = 4666$ kN force loads this ‘column’ of combined rigidity. Lasting long, parts of the soil column bear this load according to their rigidity against compression (E.A), assuming that originally horizontal plains remain flat ones. So the load falling upon the strengthened part 4399 kN and the load falling upon the part without grouting will be 267 kN. Hence, the load transferred onto the clay surface from the total 7000 kN will be $4399 + 2334 = 6733$ kN and on the $10.9\ \mathrm{m}^2$ surface of the shallow grouting deepened down to 50 cm in the clay on the level of deepening down to 50 cm in the clay, respectively. The stress developed after grouting on the surface of load transfer under the shallow foundation

$$\sigma_0 = \frac{6733}{10.9} = 617.7\ \mathrm{kPa}.$$

Before grouting the average stress originated from the building load was here $\sigma_{ag} = 87$ kPa. The stress increment $\Delta\sigma = 617.7 - 87 = 530.7$ kPa. From this extra stress, during and after the construction in the course of slow deformations further deformations and settlements came into being.

We have to know the compression modulus of the clay of Kiscell, to be able to establish the expectable value of the settlement.

For our calculations we use the result of the B100 drilling made at the Bartók Béla street, as regards the place, this is the nearest one to the building:

$$E_s = 106500\ \mathrm{kPa}.$$

Utilizing the limit depth and stress theory of Jáky for calculating the approaching value of settlement, we obtained $y = 7.7$ millimetres.

According to soil section (*Fig. 2*) foundation bodies 1–13 are standing on sand-and-gravel, while the others on a soil of sandy sludge of variable thickness

Seeing that in the superstructure of the column station 14 dilatation was constructed, experts in agreement with the designer decided that foundation bodies of the column station 14 will be the last strengthened ones.

In such a way the flat fundaments of the framing stations 14–27 of the building, having below them the grouting made by means of 'jet making', are transformed into shallow foundations standing on the clay of Kiscell, which will be practically motionless after having finished the consolidation movements. The state without motion of these fundaments and the movements ($0 - 2$ mm) of the foundation bodies 1–13, in all likelihood, will not cause cracks on the building on account of dilatation.

Certain data of the jet making machines delivered for construction were different from that reported in design phase. This is why we changed the places of the jet columns while maintaining the principles elaborated in the course of design. In the final report, we summed up the geometric places of the jet 'piles' effectively made at the individual foundation bodies, the sequence and dates of preparation, the results of levelling made during the construction period, the end values of measured settlements, and the load transfer surface on the level of solitary foundation bodies (-5 m), on the clay surface, and on the lower lever of the jet 'piles' reaching down $\sim$ 50 cm into the clay. From the former, we present now the data series of the fundament AB 25 (*Fig. 6*).

- The levelling has been started. Later at several foundation bodies has already been constructed a few jet 'piles'. Therefore there were minimum settlement values
- In the case of foundation bodies, where the gravel stratum was thicker and the sludge stratum with sand filling was thinner, the movements and settlement values originating from foundation grouting were lower than those at the foundation bodies, where the sludge stratum was thicker and the sand-and-gravel stratum was thinner. From the foundation bodies at the street front of 23 October street geodetic basic measurement was only made for fundaments marked AB 22; AB 24; AB 25 and AB 26 before deepening of the first jet 'pile'. It can be stated from this data series that the construction of the first jet pile caused $1 - 1.5$ mm settlement, that of the first three piles caused $\sim 3.9 - 4.5$ mm and that remained on this level at the fundaments marked AB 24 and AB 25 until the construction was not continued after 30 September. After having constructed the further 7 pieces of jet piles the $\sim 9 - 12$ mm values were obtained for the total settlement.

The 16.6 mm settlement of the foundation body marked AB 26 can be explained by the small thickness of the gravel stratum and its high content in sand, silty sand, further the softness of the sandy sludge above the gravel.

We summed up in a single figure the construction sequence and dates of the 229 jet piles constructed during the foundation strengthening works. On the basis of that, we can establish that the building contractor observed the cyclical construction sequence prescribed by designs and in the course of consultations, both in longitudinal and transversal directions, further within the individual fundaments. This is the reason for that although settlements approached and reached respectively the predicted value (7–9 mm), differences in settlement values did not exceed the prescribed $\Delta = 3$ mm maximum differences as for limit value of settlement values. So visible important damages and glass breaking cases did not take place neither in the superstructure of the building, nor in the neighbouring building.

5. Settlement and Strength Safety of the Strengthened Foundation Bodies

For the proposal of designer Olajterv (Design Centre for Oil Industry) the investment company pointed out the jet piles marked 4 below the foundation body marked CD 26 and piles marked 3 below the unit marked AB 26 for checking the uniaxial compressive strength. Eurosound GmbH effected the core drilling sampling on behalf of Soletanche Hungaria. Their photos relating to that are to be seen in *Fig.7*. We endeavoured to see out the specimens with $\sim 1:2$ diameter: height ratio from sound parts without inclusions.

We stored the core specimens prepared for compression in a moist surrounding saturated with humidity until beginning the execution of tests. We marked the specimens cut out of bore drilling 26AB/3 $A_1 - A_7$. The specimens cut out of bore drilling CD 25/4 were marked C1–C8 and one specimen not identified by depth was marked CX. The broken samples are to be seen in *Fig. 8*.

The compressive strength of the stone according to MSZ 18285/1 6.3:

$$\bar{q}_u = \frac{\sum_{i=1}^{n} q_{ui}}{n} = \frac{83.328}{15} = 5.55 \text{ MPa}$$

Splitting tensile strength $\sigma_t = 1.345$ MPa.

The shearing strength parameters calculated according to MSZ 18253/3-79 7.8:

$$\phi = 20^\circ \; c = 911 \text{ kPa}.$$

Modulus of elasticity: $E = 700000$ kPa.

We tested the load bearing capacity and safety against breaking of the foundation bodies on the lower level of the strengthened concrete foundation bodies and jet grouting strengthening. We took $q_u = 5.55$ MPa uniaxial compressive strength of the jet piles as basis on the lower level of the fundaments and $\sigma_t = 4322$ kPa compression stress of the jet piles on the seating surface of the clay of Kiscell.

$$P_t = A_B \cdot \sigma_t = 10.75 \cdot 4322 = 46460 \text{ kN}; \; P_{\text{measured}} = 7000 \text{ kN}$$
$$n = \frac{46400}{7000} = 6.63 > 2.77 = n \quad \text{(prescribed according to MSZ).}$$

On the level of foundation

$$A_{as} = 5.25 \text{ m}^2; \; q_u = 5.55 \text{ MPa};$$
$$P_t = 29137 \text{ kN}; \; n = \frac{29137}{7000} = 4.16 > 2.77.$$

Table 4. Safety against breaking

Sign of foundation	safety			effective safety/ prescribed safety	
	on the basic level	on the jet pile bottom	prescribed		
AB14	4.16	6.63	2.77	1.5	2.39
AB16-23 CD20-23 AB26 CD26	3.94	9.7	2.77	1.42	3.5
AB24-25 CD24-25	3.94	9.47	2.77	1.42	3.42
CD14-15 and 17	4.63	9.2	2.77	1.67	3.32
27	5.1	13.41	2.77	1.84	4.84

So we can state on the load bearing capacity of the strengthened foundation bodies that they have safety against breaking essentially (1.42–1.48 times) higher than the $n = 2.77$ value prescribed by the standard.

MOL Geodézia and Eurosond measured the settlement of the foundations, having been taken place during construction.

We summed up the settlement values from the summing up of the part-measurements, the probable values on the basis of the former, and the measurements of Eurosond as follows in the *Table* 5.

On the basis of the table it can be stated that the settlement values increase up to 4-10 mm according to the frame numbering, that is tantamount to say, according to the increase in the thickness of the sandy sludge stratum.

The settlement differences did not exceed anywhere the permissible 3 mm value. As regards the foundation, the jet grouting achieved the aim, since the safety

Table 5. Settlements of the fundaments

Number of frame station	fundaments AB on the street front mm	fundaments CD in the courtyard mm
14–15	4	4
16	3	no courtyard equivalent
17	4	4
18	4	has not been grouted
19	5	has not been grouted
20	5	5
21	5	4
22	6	6
23	6	6
24	10	7
25	9	6
26	12	10
27	10	12

against breaking of the foundation bodies exceeds the $n = 2.77$ value prescribed by MSZ and the slow continuous settlement of the fundaments originating from the continuous deformation of the overloaded sandy sludge soil came to end.

The levelling having been started September 11 1997 and has being continued till now justify the above said. So for instance, 27 March, 2002, on the occasion of the measurement No. 17, the presented foundation body marked 25 showed 0.7 mm settlement.

6. Contributors and Sources

Successful execution of the strengthening works of the complicated building superstructure and foundation of such a great volume was made possible by the concerted work of the contributors. Project manager: Béla Lányi, Chief designer: Lajos Horváth:

Contractor: Soletanche Hungaria, Chief engineer: Dr. Pál György, Building Engineer: József Tóth certified civil engineer.

Architectural experts: Dr. István Halász
Dr. László Szerémi
Dr. Bertalan Juhász

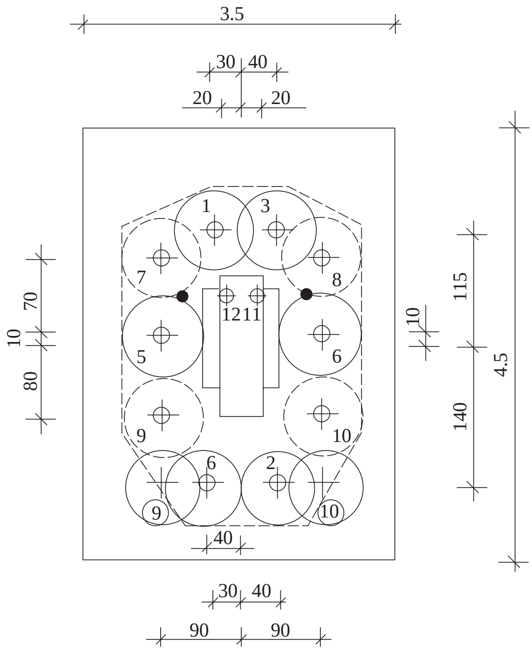

Fig. 5. The strengthened deep foundation

Remarks:

1. The jet piles 1–8 are vertically arranged, and their planned diameter amounts to 80 cm
2. The jet piles 9, 10 are inclined. Their place in the foundation level is marked 9 10, and those on the clay surface 9, 10.
3. The diameter of 9 and 10 jet piles must be widen out to 1.1–1.2 meters under the foundation at a length of about 2 meters.
4. The place of the inclined piles 11 and 12 is marked •11, •12, those on the clay surface + 11, + 12.
5. Load transfer surface on the foundation level: $Agl = 5.2\ m^2$, on the clay surface: $Ac = 5.5\ m^2$.
6. Contour of foundation on the clay surface: 10.75 m.
7. Load transfer surface at the end of jet piles being in a depth of about 50 cm in the clay: $10.9\ m^2$.

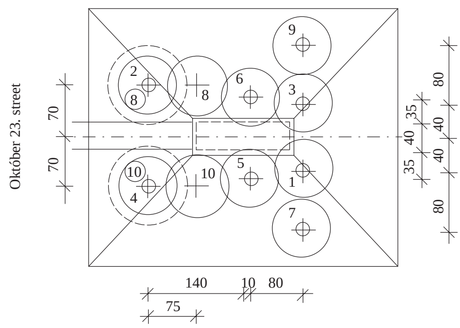

Fig. 6. Settlements during the foundation strengthening works

Construction date: 1996

Jet number	Foundation number AB 24	AB 25
1	09.04	10.11
2	10.09	09.04
3	08.28	09.04
4	09.03	10.09
5	09.27	10.03
6	10.03	10.14
7	10.11	10.15
8	10.17	10.21
9	10.14	10.17
10	10.15	10.22

Measured statements 1996 08.27

measure date	Measured settlements AB 24	AB 25
09.02	– 1.1	– 1.8
09.04	– 1.5	– 2.5
09.06	– 3.9	– 4.5
09.09	– 4.5	– 5.0
09.11	– 4.6	– 5.1
09.13	– 4.4	– 4.6
09.16	– 4.5	– 4.7
09.20	– 4.4	– 4.7
09.24	– 4.8	– 4.6
09.27	– 4.4	– 4.5
09.30	immeasurable	– 4.5
10.04	– 6.1	– 7.6
10.07	– 6.6	immeasurable
10.11	– 7.6	– 7.1
10.15	– 7.7	– 8.5
10.17	– 8.4	– 9.6
10.18	– 9.0	–10.2
10.21	– 8.5	– 9.6
10.22	– 8.9	– 9.4
10.28	–10.4	–11.3
10.31	liquidated	–10.0
11.01	liquidated	–10.8
11.04	liquidated	– 9.3
11.05	liquidated	– 9.7

Total settlement: $s = 9.0$ mm

Remarks:

The piles 8 and 10 are inclined. Their place in the foundation is marked 8 10 and those on the calay surface 8; 10. These piles (8; 10) have to be made with a dia of 1.1–1.2 mm in a length of 2 m directly under the foundation level: $Agl = 4.9\ m^2$. Load transfer surface at the end of jet piles: $Ac = 15.2\ m^2$.

Fig. 7. The core specimens

Soil mechanic laboratory tests were directed by Dr. György Horváth.
The author as for expert civil engineer contributed beginning from the first

exploration up to the realization. Those have been delivered in the paper, were taken from the reports of Dr. Miklós Müller BME university reader (Budapest University of Technology and Economics).

- Expert appraisement on the foundation load bearing capacity and settlement of the office building of the MOL residential building at number 16–18 23. October street Budapest, XI. and the necessity of its strengthening. November 7, 1994.
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- Expert study on the execution of the foundation strengthening of the MOL residential building, on the strength of the soil treated with the jet grouting procedure, on the strength of the strengthened soil, on the load bear capacity of the strengthened foundation bodies and on the settlement values. Budapest, 5 December, 1996.

Fig. 8. The specimens after the breaking tests