TC 207 'Soil Structure Interaction'.
CASE HISTORIES

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Saint Petersburg

Collection of monitoring data on deformations of existing buildings

Introduction

Developments in computational apparatus have made it possible to start using calculations of soil structure interaction (SSI) in everyday design applications. As evidenced by the totality of experience of such calculations available to date, this approach allows significant refinement of load definitions in foundations and superstructures. However, SSI calculations unequivocally exact a higher degree of accuracy of geotechnical predictions, which with usage of SSI calculations will determine not only settlement values but also structural parameters, bearing in mind also not infrequent influence of SSI calculations on structural solutions of principal importance.

Correspondingly, one of the primary objectives is refinement of settlement predictions methodology. The existing analytical settlement calculation methods owing to the assumptions present therein permit merely of conjecture with respect to expected settlement values. The reason for such inferior settlement predictions will be found not so much in inadequate site investigation, but rather in an oversimplification of the physical nature of the involved phenomena.

Currently there exist technical possibilities of using appropriately complex nonlinear models of soil mechanics in subsoil calculations. Therewith a most promising direction lies in taking into account actual physical soil properties as displayed and identified by means of site and laboratory investigation programmes.

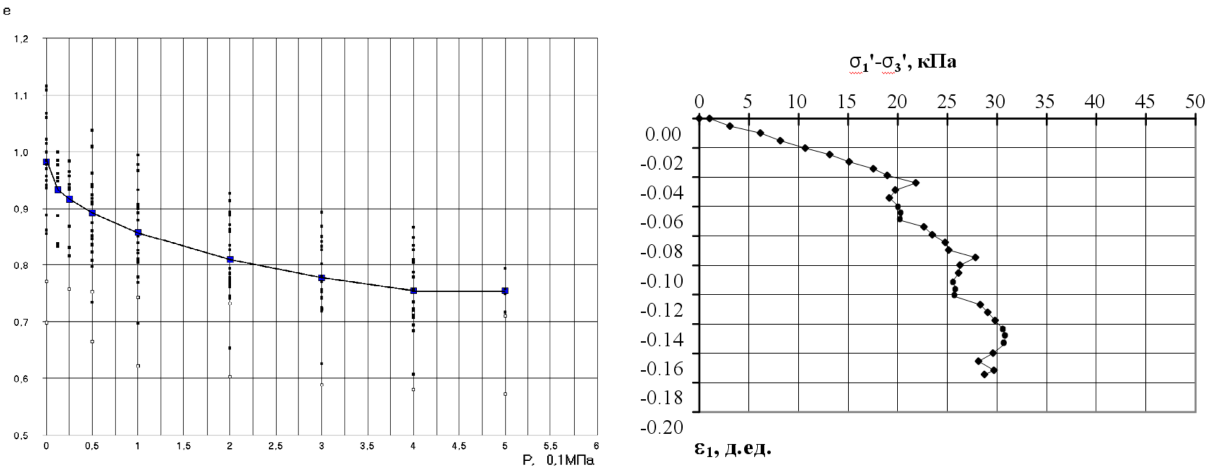
Additionally, application of complex models of subsoil behaviour presumes that a host of precise parameters needs to be defined, wherewith to avoid such eventualities when usage of inadequate input data in complex models and numerical calculation methods is detrimental to the desired accuracy.

In view of the above, it is of utmost importance that various approaches to settlement calculations be evaluated by means of comparison with available *in situ* monitoring data. Consequently, collection, review and evaluation of the available data throughout the world are extremely significant.

This report contains settlement materials that have heretofore been collected from buildings located in complex ground conditions of St. Petersburg.

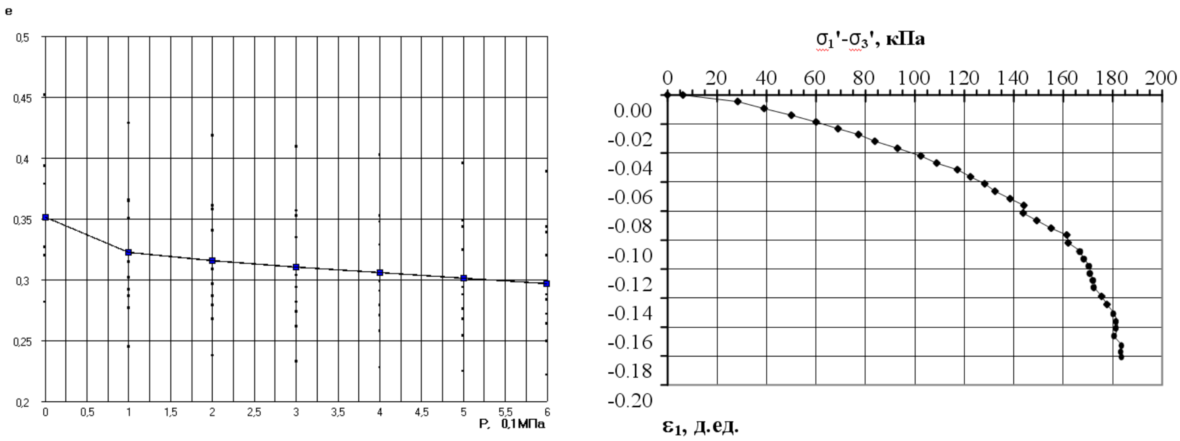
1. Specifics of soft soil behaviour in St. Petersburg.

Thick deposits of soft saturated clays are typical for St. Petersburg, where thickness of clay strata may reach 20-30 m. Figure 1 contains an example of characteristic test results obtained for soft soils (fluid sand with clay). Those deposits may be overlain by sand layers, primarily being the subsoils for many historic buildings and structures. The soft strata are usually underlain by stiffer moraine deposits, whose behaviour in a laboratory test displays itself in the manner shown in Figure 2. Between the moraine deposits there often are intra-moraine soils resembling the weaker upper layers by nature and composition. Underneath the moraine there will usually be Cambrian and Proterozoic deposits of hard clays (please refer to Figure 3 for a characteristic test result).



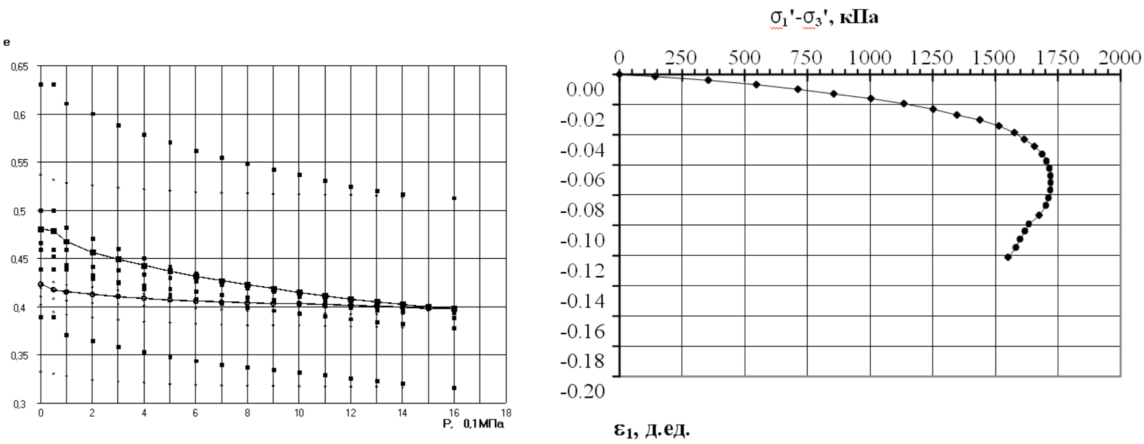
a) b)

Figure 1. Characteristic test results for soft soils (quick sand with clay):
a –oedometric test results (void ration e versus pressure P (0.1 MPa); b – results of the unconsolidated-undrained triaxial tests (deviatoric stress (KPa) versus vertical deformation ε_1).



a) b)

Figure 2. Characteristic test results of moraine deposits (quick sand with clay):
a – oedometric test results (void ration e versus pressure P (0.1 MPa); b – results of the unconsolidated-undrained triaxial tests (deviatoric stress (KPa) versus vertical deformation ε_1).



a) b)

Figure 3. Characteristic test results for hard clays: oedometric test results (void ration e versus pressure P (0.1 MPa); b – results of the unconsolidated-undrained triaxial tests (deviatoric stress (KPa) versus vertical deformation ε_1).

The clay soils underlying the hard clays display predominantly plastic failure in triaxial tests, the physical and mechanical parameters of the moraine being more often than not rather low. Additionally, for all these soils shear resistance does not grow with depth, which is clearly visible from CPT and vane-test results. Following traditional classification, the soils in question are ***underconsolidated***, however, not showing any significant signs of potential consolidation in the foreseeable future. On the other hand, when conditions conducive to consolidation are present those soils undergo quick consolidation, as exemplified by the tests conducted on soils underlying St. Petersburg flood prevention structures, where it was observed that the settlements were initiated by installation of drains at no applied loads whatsoever. This proved the underconsolidated nature of those soils being liable to consolidation given the conditions for drainage.

When performing laboratory tests on St. Petersburg clays it is impossible to identify either their, so called, 'structural strength' or their OCR. Laboratory tests performed on this type of soils give rather low values of permeability ratio, *i.e.* in the order of $10^{-7}…10^{-8}$ cm/sec. It becomes clear from the simplest calculations that even at such low values of permeability ratio these soils should have consolidated under their own weight over the thousands of years of their existence. Such consolidation, however, did not occur. The reason therefore is the non-linear dependency of permeability ratio on the water pressure gradient. Professor R. Dashko has often reiterated in her publications that water pressure gradients during laboratory tests as a rule exceed those observed in subsoils *in situ* by 2-3 factors of magnitude. Correspondingly, values of permeability ratio derived from laboratory tests are over-evaluated. The *in situ* water pressure gradients thus prevent consolidation from happening within any appreciable (non-geological) timeframe.

Over the time the clay deposits in question were accumulating, their consolidation reached a state in which pore water seepage practically stopped. This was followed by hardening of their structural bonds leading to the pore water being immobilized within. Resulting from that, as further deposits were added on top, subsequent consolidation of the clays never occurred, producing their persistent quasi-stable condition. This condition is most likely the reason for the constant physical and mechanical characteristics within the confines of that sensitive stratum. Certain site investigations in St. Petersburg have identified soft moraine strata at depths of about 80 m in paleo-valley areas, having retained high water content and fluidity despite the long age and high pressures. Clay characteristics predominant in St. Petersburg obviously suggest that volumetric deformations of such soils will have been hindered owing to their low permeability and will have been occurring over a long period of time. Correspondingly, bearing in mind low strength of such deposits, the principal deformation factor will be that of form change. When calculating settlements, this deformation should be regarded as requiring a particular scrutiny.

Unfortunately, the engineering methods of settlement calculations ignore this element, whereas the existing non-linear models of soil mechanics do not permit of an appropriate description of the indicated specificity of clay soils behaviour. Not wishing to dwell on the drawbacks of numerous existing models, one might feel content to say that it is the correct treatment of form change deformation that presents itself as their biggest challenge.

2. Statistical data on properties of St. Petersburg clay to define soil behaviour parameters in view of insufficient site investigations data.

To our deepest regret in the majority of cases the projects where monitoring data are available lack sufficient site investigations, particularly, results of triaxial tests. As a result, no data on soils behaviour under form change are available. It must be noted that in this respect it is not so much the strength of the samples which is of interest, but their deformation properties at relatively insignificant deformation values. That is to say, it is necessary to somehow restore the curve representing dependency of deformation on deviatoric stress in triaxial tests.

To bridge this gap we had collected a big number of triaxial tests results (around 300) obtained for various St. Petersburg clays based on which the following dependency between the unconsolidated-undrained triaxial tests and water content of the soil was derived (Figure 4):

$$c_u = 1.6373 e^{0.122 w\%} \ \text{[kPa]} \qquad (1)$$

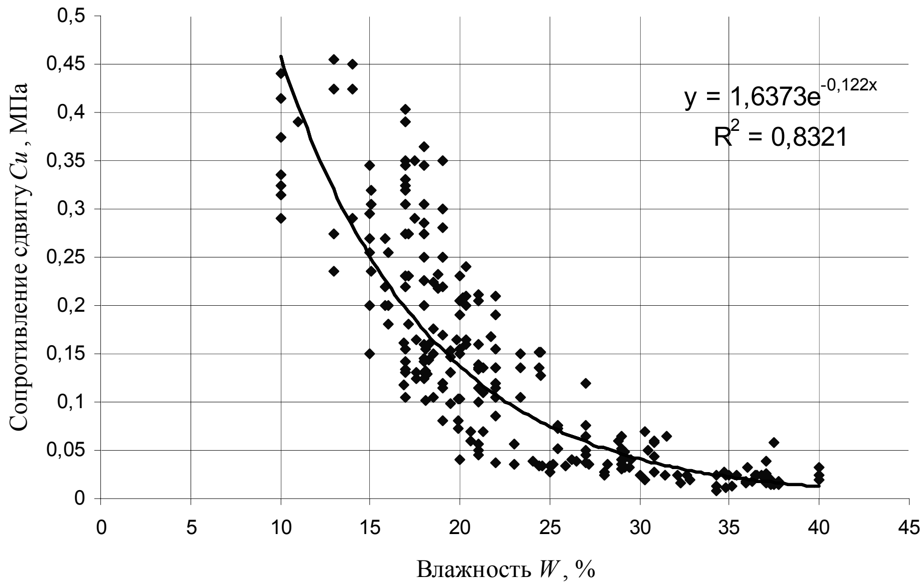


Figure 4. Exponential dependency of shear resistance C_u (MPa) on water content W

For the available tests the derived correlation ratio was relatively good (0.91), inasmuch as it allowed to rather accurately apply dependency (1) to define properties of a broad spectrum of St. Petersburg clays. As mentioned above, the majority of clays display plastic failure pattern. In this case the conditional failure criterion during testing was taken to be vertical deformation of 15%. Thus, dependency (1) not only gives strength values of soil, but to some extent restores deformability data in triaxial compression, assuming (with certain degree of conditionality) geometric similarity to curves resembling the one presented in Figures 1 and 2.

In cases where the data on SPT and triaxial tests were also available, the following dependency (Figure 5) between the strength in unconsolidated-undrained triaxial tests and cone tip resistance was derived:

$$c_u = \frac{q_c}{19} \qquad (2)$$

Correlation ratio (0.96) for the above dependency is higher than for the previous one, therefore, in cases when SPT data are available it is desirable to assume soil parameters at form change deformations according to formula (2).

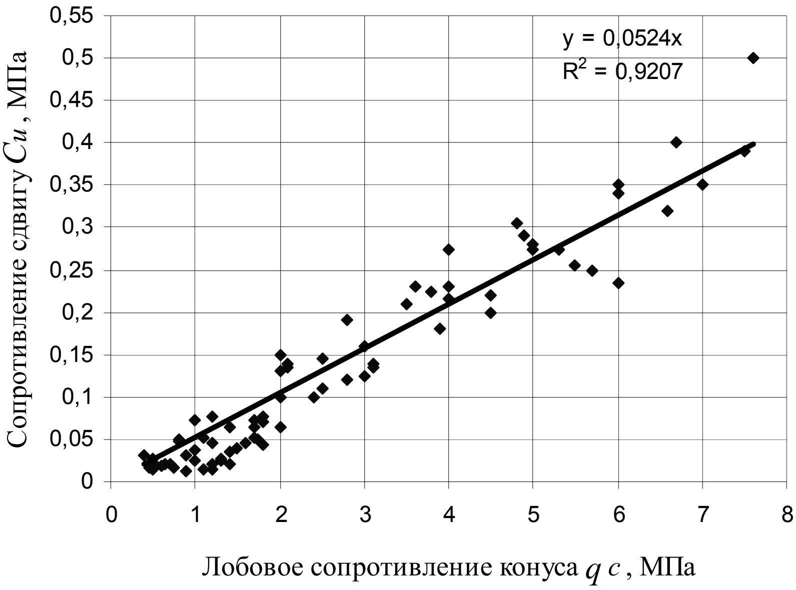


Figure 5. Linear dependency between shear resistance Cu (MPa) and cone tip resistance q_c (MPa) in SPT

It should be noted that quality of 'undisturbed' samples in the majority of cases was quite unsatisfactory. In this connection there is a possibility that the derived dependencies might have been those characteristic for partially disturbed samples.

Permeability ratios in clay soils are determined rather inaccurately; moreover, the conditions of tests (head gradients) essentially differ from the natural ones, which might lead to the higher water permeability parameters of the soil. Based on the analysis of numerous consolidation curves it is possible to adopt the following approximate average values of permeability ratios:

for soft lacustrine and glacial soils – K_f=7.1x10^{-5} (m/days)

for stronger glacial deposits (moraine deposits) – K_f=2.4x10^{-5} (m/days)

for hard clays – K_f=5.3x10^{-6} (m/days)

3. Long-term monitoring data.

To evaluate reliability of various methods of subsoil strain calculations, monitoring case histories of 15 St. Petersburg buildings were put together. Each case history is supplied with basic description of types of structures, foundations and structural loads. Also, the available data of subsoil parameters are given: tables of soils characteristics and SPT data (if available). The numbering of pictures listed below agrees with the number of case history in the report.

CASE HISTORY 1.

Building address: St. Petersburg, Krasnogvardeysky District, Bolsheohtinsky Prospekt 16.

Construction commenced in 2000. Overall building layout is shown in Fig. 1.1.

The building was constructed as:

A 17-storey structure 51.5 m high and plan dimensions of 70.5x20.0 m, featuring internal walls and intermediate floors constructed of solid reinforced concrete with thickness 160 mm to 200 mm and internal bearing walls with thickness 640 mm to 510 mm (Fig. 1.2.). Pressure throughout the site reached 255 kPa.

Fig.1.1 Front elevation of a 17-storey building.

The building has piled foundations on bored piles of equal length but varying diameters, 23.5 m and 450 and 350 mm respectively. Number of piles is 520. Static load tests have testified to pile load of 90 tonne for a 450 mm pile, such load being assumed in the design (for a 350 mm pile the load being 60 tonne). The design likewise assumes pilecaps to go under internal and external bearing walls, as being 1200 to 2400 mm wide and 900 mm thick.

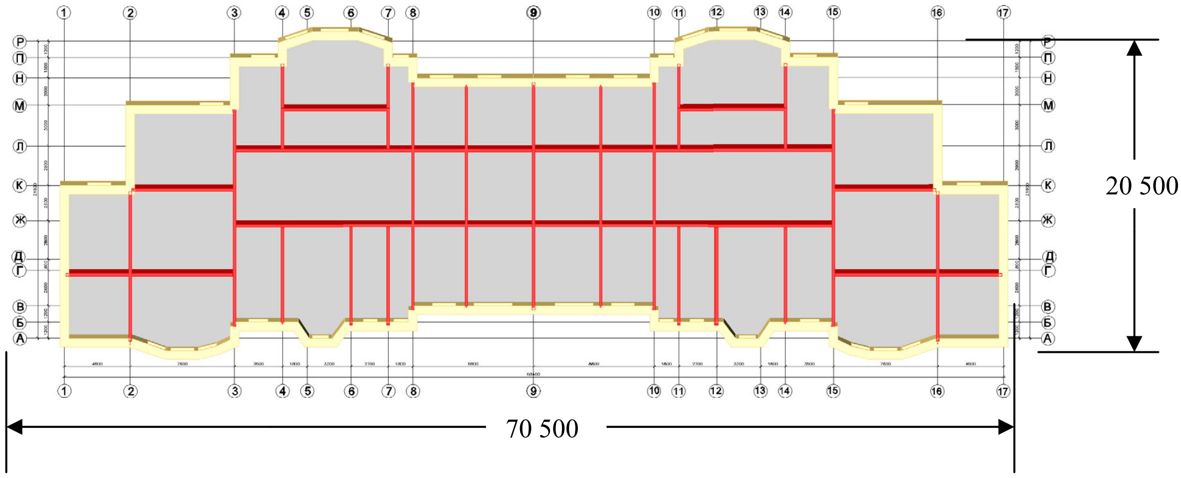


Fig.1.2. A typical floor plan (internal bearing reinforced concrete walls 200 mm thick indicated in red, external bearing brick walls 640 mm thick indicated in yellow)

Site Investigation Results

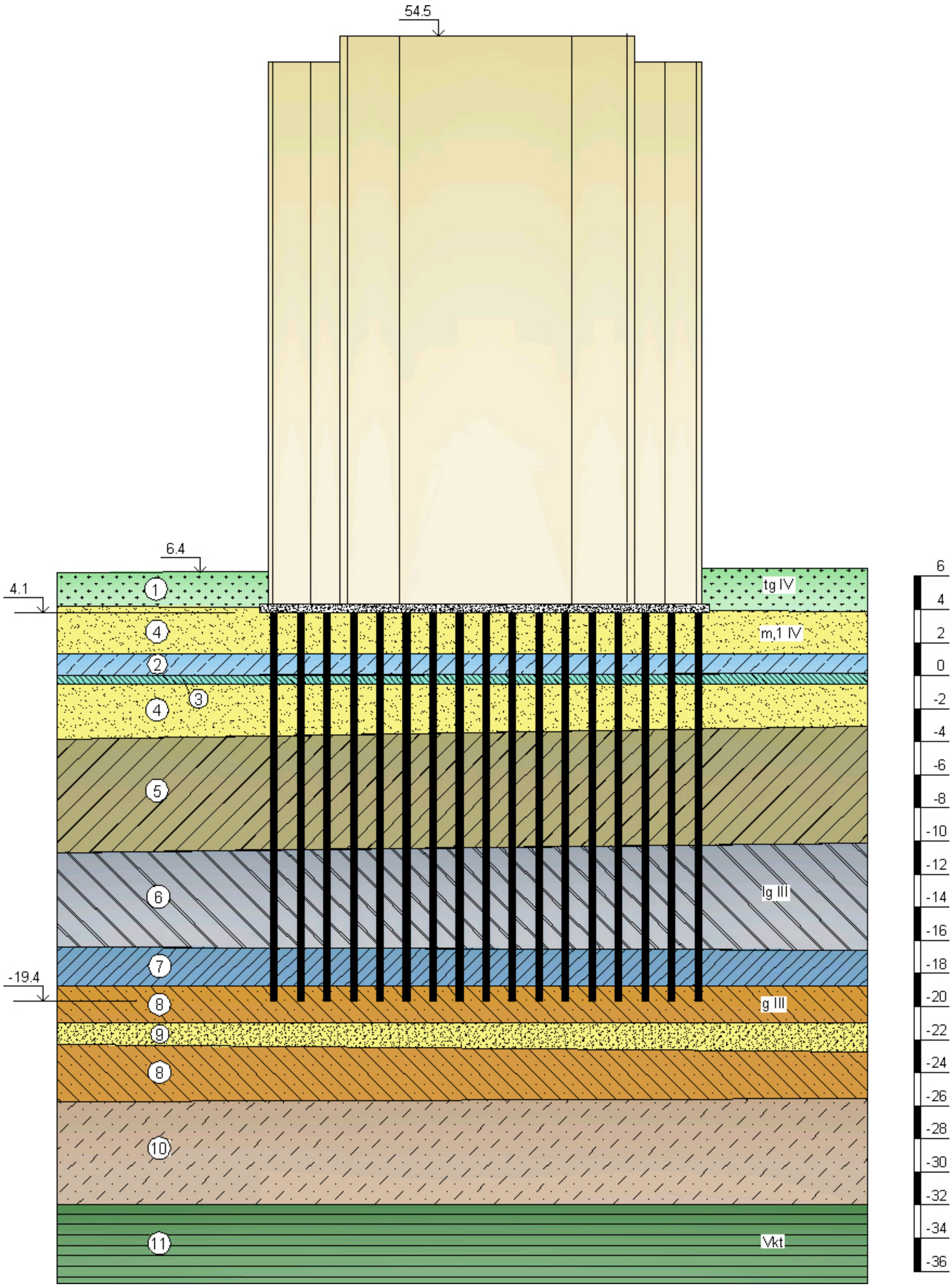


Fig.1.3.Site stratification for the construction of a 17 storey building

SOIL PARAMETERS

Table 1.1

Geolog. index	Soil identification	Ref. № of stratum	Characteristic	Plasticity index, I_p	In situ water content, W	Soil density $p, ò / m^3$	Porosity factor, e	Consistency parameters I_L	Consistency parameters C_B	Strength parameters φ, deg	Strength parameters $ñ, kg / cm^2$	Strain modulus $E, kg / ñm^2$
1	2	3	4	5	6	7	8	9	10	11	12	13
tg IV	Made ground	1	X	Working resistance $R_0 = 0.8$ kg/cm2								
			X_{II}									
m,1 IV	Light clay with sand sandstony with sand inclusions, soft and pliable	2	X	0.11	0.31	1.87	0.928	0.55	0.51	14	0.1	85
			X_I			1.87+0.03				12	0.07	
			X_{II}			1.87+0.02				14	0.1	
m,1 IV	Light silty clays with organic admixture, soft and pliable	3	X	0.20	0.57	1.63	1.556	1.10	0.35	7	0.09	30
			X_I			1.63+0.07				6	0.06	
			X_{II}			1.63+0.04				7	0.09	
m,1 IV	Medium density silty sand, saturated	4	X	-	-	2.04	0.600	-	-	29	0.03	150
			X_I			2.04+0.1				26	0.02	
			X_{II}			2.04				29	0.03	
m,1 IV	Banded silty sand with clay, weak	5	X	0.07	0.26	1.99	0.724	1.33	0.46	17	0.03	65
			X_I			1.99+0.04				15	0.02	
			X_{II}			1.99+0.02				17	0.03	
lg III	|Banded silty clay with sand light, weak	6	X	0.11	0.36	1.87	0.976	1.32	0.48	11	0.05	60
			X_I			1.87+0.02				10	0.03	
			X_{II}			1.87+0.01				11	0.05	
lg III	Banded silty clay with sand, light, weak	7	X	0.10	0.28	1.96	0.784	0.78	0.32	15	0.11	80
			X_I			1.96+0.05				13	0.07	
			X_{II}			1.96+0.03				15	0.11	
g III	Silty sand with clay, incl. gravel & pebble, weak	8	X	0.04	0.15	2.19	0.411	0.47	0.26	26	0.18	90
			X_I			2.19+0.01				22	0.12	
			X_{II}			2.19+0.01				26	0.18	
g III	Silty sands, stiff & saturated	9	X	-	-	1.87	0.550	-	-	30	0.04	180
			X_I			1.87+0.02				27	0.03	
			X_{II}			1.87+0.01				30	0.04	
g III	Silty sand with clay incl. gravel & pebble, strong	10	X	0.04	0.14	2.23	0.376	0.20	0.17	29	0.21	180
			X_I			2.04+0.1				35	0.01	
			X_{II}			2.04				39	0.01	
Vkt	Cambrian clays, hard	11	X	0.05	0.12	2.20	0.243	-0.8	-	26	0.35	400
			X_I			2.20+0.01				22	0.31	
			X_{II}			2.20+0.01				26	0.35	

X_H – *coded value;* X_I – *for bearing calculations;* X_{II} – *for deformation calculations*

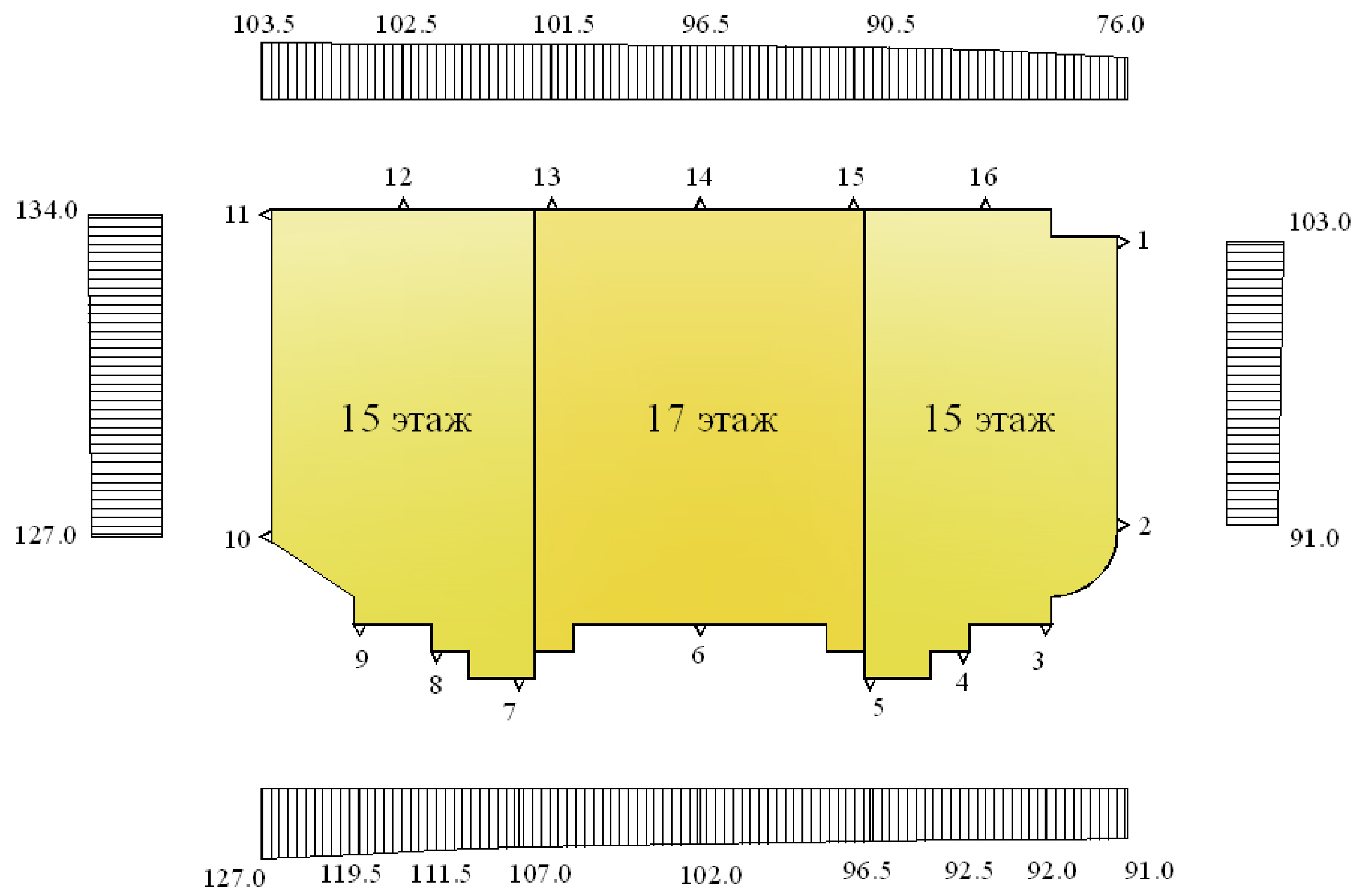


Fig.1.4. Settlement contours as per the latest measurement cycle

Settlement values per each settlement gauge (mm)

Table 1.2

Gauge/ time	18.03.2003	22.04.2003	11.06.2003	01.09.2003	12.11.2003	02.02.2004	07.04.2004	06.07.2004	01.11.2004	15.02.2005
	1 floor	2-3 floors	4-5 floors	8 floor	12 floor	13-14 floors				
1	0.0	0.0	-2.5	-18.0	-30.5	-61.5	-74.5	-86.5	-101.5	-103.0
2	0.0	0.0	-2.5	-17.5	-28.5	-56.0	-64.0	-76.0	-90.0	-91.0
3	0.0	-1.5	-2.5	-17.0	-29.5	-56.5	-64.5	-75.5	-90.0	-92.0
4	0.0	-0.5	-2.5	-17.0	-31.5	-58.5	-64.5	-76.5	-90.5	-92.5
5	0.0	-2.0	-3.0	-18.5	-32.5	-60.0	-67.5	-79.5	-94.0	-96.5
6	0.0	-1.0	-2.0	-19.0	-36.0	-66.5	-78.0	-84.0	-98.0	-106.0
7	0.0	-1.0	-3.0	-19.5	-35.0	-63.5	-81.0	-88.0	-101.0	-107.0
8	0.0	0.0	-3.0	-19.5	-37.0	-66.0	-84.5	-98.0	-109.5	-111.5
9	0.0	-0.5	-4.5	-20.0	-36.0	-67.0	-87.5	-101.5	-116.0	-119.5
10	0.0	-0.5	-2.0	-19.0	-36.0	-69.5	-91.0	-105.0	-118.5	-127.0
11	0.0	0.0	-2.5	-20.0	-36.0	-72.0	-95.0	-111.0	-125.5	-134.0
12	0.0	-1.0	-3.5	-22.0	-39.5	-73.5	-95.0	unrecoverable	-95.0	-102.5
13	0.0	0.0	-3.5	-25.0	-40.5	-73.5	-93.5	unrecoverable	-93.5	-101.5
14	0.0	0.0	-4.0	-21.5	-35.5	-64.5	-81.0	unrecoverable	unrecoverable	unrecoverable
15	0.0	-1.0	-5.5	-26.5	-41.5	-71.5	-89.5	unrecoverable	-89.5	-96.5
16	0.0	0.0	-4.5	-23.0	-40.5	-70.0	-86.0	unrecoverable	-86.0	-90.5

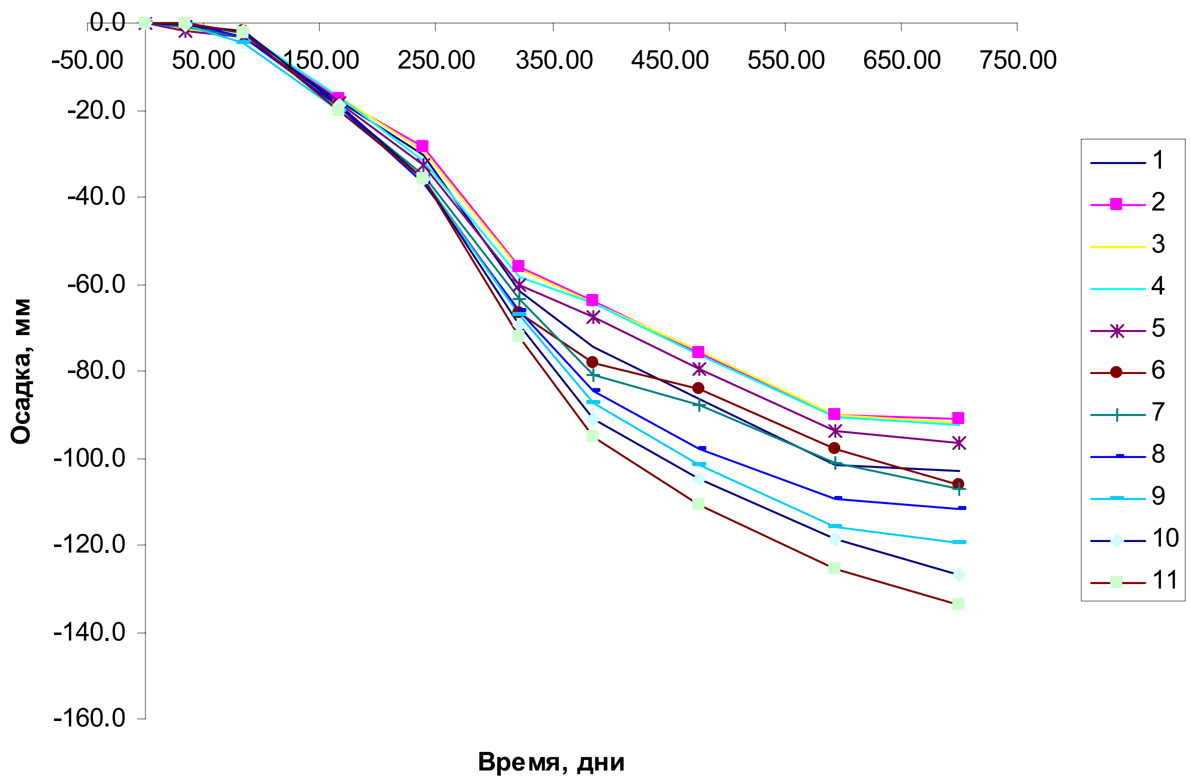


Fig.1.5. Settlement contours as measured on settlement gauges.