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THE ROLE OF GEOTECHNICAL MONITORING AT DESIGN OF FOUNDATION STRUCTURES AND THEIR VERIFICATION – PART 1

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Abstract

For a long time, design of the square foundations has not posed a problem in engineering practice. Foundations are designed on the basis of determining the bearing capacity of the subsoil, where irregularities in its determination oftentimes affect the efficiency (economy), while possible failures of bearing resistance of foundation soil are rare. More important factor is the resulting deformation of second limit state under consideration - settlement, relative settlement, tilting and excessive deformation. The current pressure on the cost reduction of design preparation and obtaining important data from geotechnical survey often results in many cases into adverse effects of settlement and differential settlement of foundations of the buildings. The question lies in a variety of analytical methods for assessing the service ability of limit states used in European countries as well as the underestimation of the proposal risks. Authors of the article want to document the fact that the most important influence on adequate and safe design is the most accurate determination of geotechnical parameters and the appropriate selection of the calculation method. For the purposes of explanation, Monte Carlo simulation technique was used to test a variety of geotechnical parameters, which will be presented in second part of article. If all construction processes are carried out successfully, rarely are the buildings evaluated once again. However, when the opportunity to participate in the stage of engineering survey and collection of geotechnical parameters as well as the control of the construction process by tools of geotechnical monitoring presents itself, it is valuable to perform the analysis of the entire process for pointing out hidden risks.

Keywords:

Geotechnical monitoring;
In situ geotechnical testing;
Settlement of foundation;
Soil replacement.

1. Introduction

Opinions that favour the usability of service ability limit state before the ultimate limit state (bearing capacity) are getting into the foreground. Admittedly, these views contain a rational core: enhancing the calculation results in lower occurrence of structural disorders as a result of exceeding the carrying capacity of less and less, while excessive or unequal settlement still remains and it is an actual problem also in other countries [1].

In this paper is presented calculated theoretical settlement of the building of commercial center in Žilina and it is compared with the actual settlement obtained by geodetic survey method, that used very precise levelling (VPL). The construction of the shopping center in Žilina city centre was founded on shallow foundations – square footings. This was allowed by favourable foundation conditions at the construction site, where the active subsoil was formed by sediments of gravel terrace. There has been discovered local geological abnormality, and gravel terrace in right side of building has very low thickness and gravels in foundation level are mostly with clayey fillings. The calculations of the settlement using analytical methods do not confirm the assumption of designers, that service ability limit state is less sensitive to input parameters. This error leads to an underestimation of the importance of the elasticity modulus, which sometimes happens especially in cases of soil formed by non-cohesive soils. The result is then usually unequal settlement, large settlement, and deformation of foundation structure, [2].

1.1. Methodology of geological survey

Present situation in construction business is characterized by the pressing of engineers to decrease time and cost of the design preparation and processing, which has influence also to the geological survey stage and land preparation. Geotechnical in-situ testing [3] plays an important role at surveying works. The tendency to use more in situ geotechnical testing comparing to classic way of geological survey, (rotary drillings and sampling, then laboratory analyses), has many advantages, [4]. Firstly, it is continuous evaluation of geological profile, cost effective and time saving process comparing with laboratory tests. From static penetration tests with piezocone (CPTu) can be classified soil type and many properties can be derived too, but we cannot exclude laboratory analyses from geological survey process, [2]. Depending on the future structure and geological environment, following surveying methods are suggested as suitable for the design of square foundations, Tab.1.

1.2 Geological conditions at area of interest

Geological conditions at area of interest are represented by Paleogene layers with alternation of sandstone and claystone, which are predominant and at the surface part are completely weathered to clay. Paleogene subgrade is covered with a massive accumulation of the deposits of the river Váh (upper terrace) of verified thickness from 20 to 21 meters. The terrace is made up of gravels class (G-F), or (GC), where there is quarterly covered by sandy clays (CS) or low plasticity clays (CI). Thickness of clays in the southern part of the site is low (which has been certified to be max. 1.5 meters, or did not occur at all – they have been replaced by an artificial backfill). In the northern part, however, has been verified to a depth of 4 to 10 meters. Sandy clays are firm to stiff consistency, low plasticity clays are stiff to firm consistency and also soft consistency. The top layer of the territory consists of anthropogenic sediments of various thicknesses from 0.3 to 1.9 meters, Fig. 1.

Table1: Suggested in situ testing method for the design of the spread foundation, [5].

Geotechnical design	**Soil Type**	**Suggested In-Situ Geotechnical Test**					
		*****most suitable, **appropriate, *least appropriate, N/A - not applicable**					
		SPT	**DP**	**CPT**	**CPTu**	**PMT**	**VST**
Bearing capacity	Fine soils	***	*	**	**	**	**
	Coarse grained	***	***	*	*	**	*
Consolidation settlement	Fine soils	N/A	N/A	*	***	N/A	N/A
	Coarse grained	N/A	*	*	*	N/A	N/A
Settlement	Fine soils	N/A	*	***	***	*	N/A
	Coarse grained	***	**	**	**	*	N/A

where: SPT – is a Standard penetration test, DP – dynamic penetration, CPT – cone penetration test, CPTu – penetration with pore pressure measurement / dissipation test, PMT – presiometer test, VST – vane shear test.

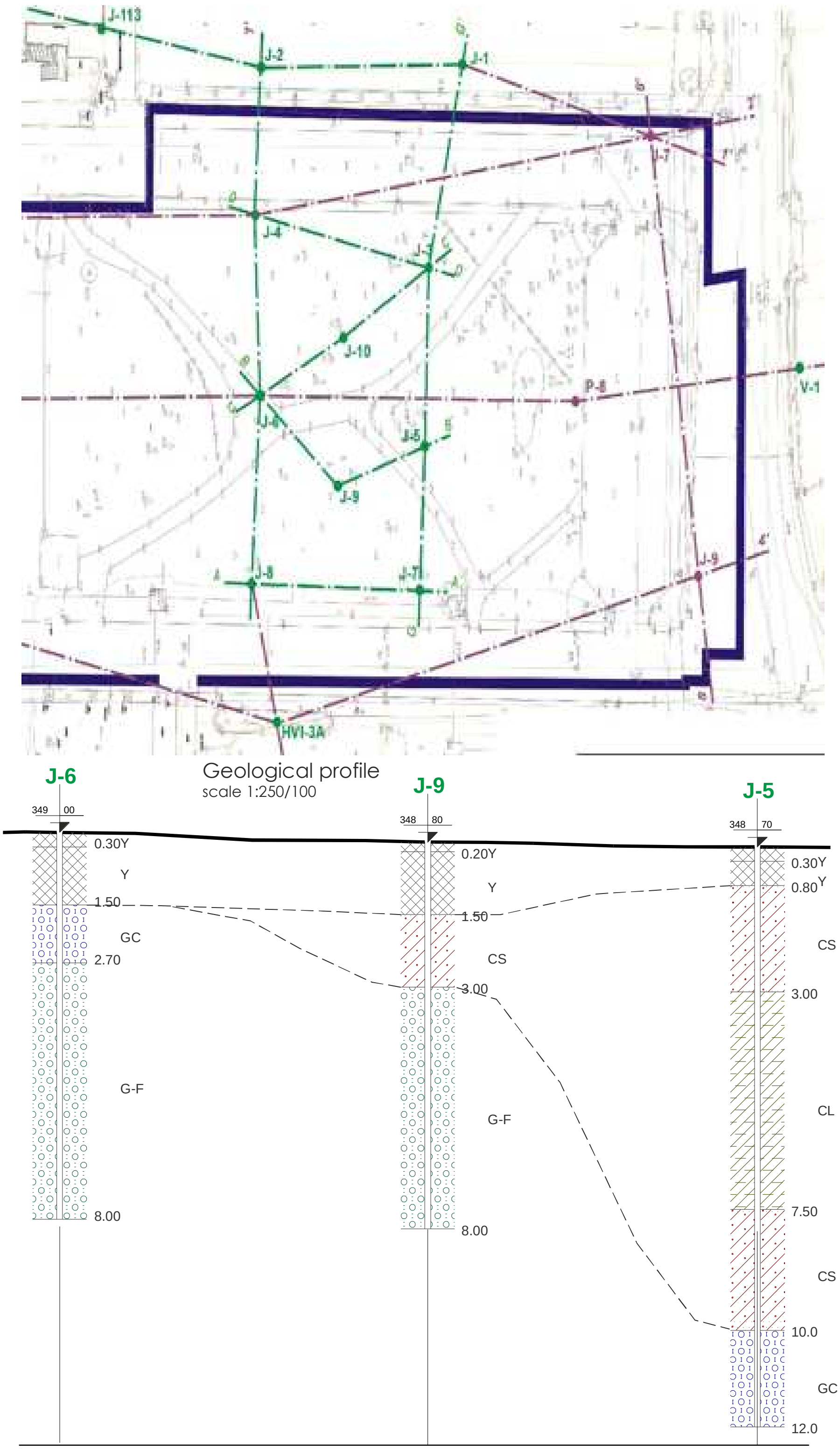


Fig. 1: left - Situation of boreholes of detailed geological survey in the part of low thickness of terrace, right - Geological profile through terrace deposit abnormality (borehole J-5), [6].

Hydrogeological conditions in the area reflect the geological structure and depend mainly on rainfall and climate conditions in the area. The level of groundwater at the site is located in a bottom layer of the gravel terrace and has been verified by deep wells at a level of 15.20 to 17.10 m below surface. Groundwater in the area is not aggressive to concrete and iron reinforcement.

2. Obtaining of geotechnical parameters

Supposed level of foundation was 8 to 10 m below surface. For speeding up the process of geological survey, firstly boreholes until the depth of 8 m were done and afterwards they were backfilled. At same position, dynamic penetration tests were carried out from level of bottom of boreholes till base of the terrace, from 8 to 14 resp. 16 m. Geotechnical parameters of gravel sediments were derived mostly from dynamic penetration tests [3, 5]. With knowledge of the soil profile, correlation equations (1, 2) were used for calculation of the deformation moduli through the dynamic penetration resistance q_{dyn}

$$E_{def} = n \cdot q_{dyn} \tag{1}$$

$$E_{def} = 9 \cdot q_{dyn}^{0.6} \tag{2}$$

The most appropriate results were derived from the empirical equation (1), with local parameter n = 5.3 valid for gravel with fine soil (G-F), and for clayey gravel (GC) was used parameter n = 3.8. Recommended Eq. (2) was not used, due to the fact that when results were compared with modulus obtained from static plate load test at bottom of foundation pit (Fig.2), there were higher differences than Eq. (1) offered.

For the calculation of consolidation settlement, the most important parameter is oedometric modulus E_{oed}, which can be calculated from (3), using conversion coefficient $ß$, as a function of Poisson's ratio v.

$$E_{oed} = \frac{E_{def}}{\beta}; \quad \beta = 1 - \frac{2\nu^2}{1-\nu} \tag{3}$$

Static plate load test (PLT) is the most used in situ method for evaluation of deformation properties of the tested layer, especially for earthwork controlling, [7, 8]. According to the strength of the soil and the diameter of plate d, effective depth is (1.5 ÷ 3) d. By recommended standard procedure, deformation modulus from first load cycle $E_{def,1}$ and second load cycle $E_{def,2}$ can be calculated. Deformation moduli from PLT and dynamic penetration tests derived from (1) were then compared at the same depth and good correlation was discovered with $E_{def,2}$ values. Because of higher contact stress below square footings, maximal level of stress at testing was 400 kPa.

3. Design of square footings

General designer of building with 2 underground floors and 3 upper floors decided to use square footings instead of the big abnormality at the place of NW of the foundation pit. The favourable position of ground water allows foundation soil replacement at depth of 1.5 m, and every footing was optimized according to the tested deformation parameters and the level of acting load to receive good values of equal settlements. This brave solution brings along significant cost savings compared to pile foundation alternative.

Fig. 2: Plate load test at replacement layer of square footing, Žilina, 2008.

Calculations with respect to service ability limit state must be used to demonstrate that the operational design load does not cause even or uneven deformation of the foundation, which would lead to unacceptable deformation of structures, [9, 10].

The final settlement *s* of the subsoil (total and partial) under investigated point of foundation was determined using the theory of one-dimensional consolidation settlement, where incremental stress $\sigma_{z,}$ was corrected by structural strength component.

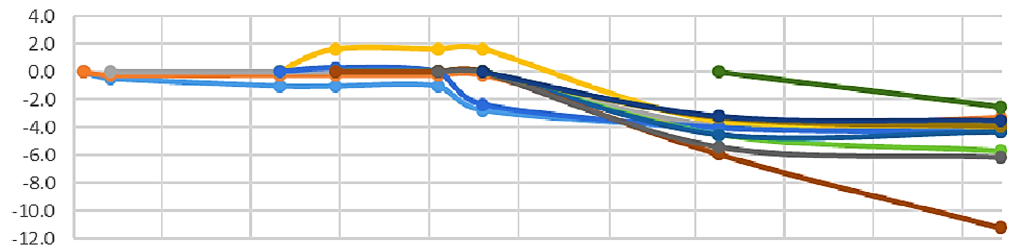
The final values of settlement were compared with limit values $s_m < s_{,lim}$ and also differential settlement was calculated in every place of foundation. This square footings alternative was successful and the contractor issued orders for precise levelling monitoring of the settlement at different parts of the large building.

4. Geodetically obtained vertical deformations

At the selected places of the building, the contractor allowed to install geodetic marks at columns of the second underground floor for purposes of measuring the settlement. For this set of points, 8 stages of precise levelling were done from the year 2008 until 2009 to obtain exact values of settlements through the time of construction stages, Fig. 3. Tree square footings closed to geodetic marks for calculation of the settlement has been selected. Calculation was done with the real values of foundation soil and level of loading stress, in addition to their comparison with the values of precise levelling measurement. Static plate load tests were carried out below footings E12, D1/D14, F15. These places were closed to the surveyed geodetic point VD9, VD2 and VD12, which allows comparing the calculated settlement and the real settlement, Tab. 2.

Table 2: Calculated and measured settlements of square footings, [9].

Footing No.	**Without replacement of foundation soil [mm]**	**With replacement layer and module $E_{def,1}$ [mm]**	**With replacement layer and module $E_{def,2}$ [mm]**	**Geodetic measurements of settlement by VPL [mm]**
E 12	30.50	12.30	5.70	6.12
D1/14	4.00	0.90	1.10	3.20
F 15	9.40	5.50	5.05	3.55



	7.2.2008	25.2.2008	20.6.2008	28.7.2008	6.10.2008	6.11.2008	16.4.2009	26.10.2009
VD1	0.0	-0.4	-1.0	-1.0	-1.0	-2.7	-4.0	-4.3
VD2	0.0	-0.2	-0.2	-0.2	-0.2	-0.2	-3.9	-3.2
VD3		0.0	0.0	0.0	0.0	0.0	-3.9	-3.6
VD4			0.0	1.6	1.6	1.6	-3.6	-4.0
VD5			0.0	0.3	0.0	-2.3	-4.0	-4.3
VD6				0.0	0.0	0.0	-4.5	-5.7
VD7				0.0	0.0	0.0	-4.5	-4.3
VD8				0.0	0.0	0.0	-5.9	-11.2
VD9					0.0	0.0	-5.4	-6.1
VD11						0.0	-3.3	-3.9
VD12						0.0	-3.2	-3.5
VD14							0.0	-2.5

Fig. 3: The results of the settlement measurements during construction process, [9].

5. Conclusions

The values of the settlement obtained by a precise geodetic measurement were found in good correlation with calculated values. The good correlation should be at the footing in axis E 12, because the position of geodetic mark is exactly fitted to this axis, (point VD-9). In the case of the other two observation points, they were placed in adjacent columns near footings. The biggest differences were obtained in the axis E12, according to the used value of deformation modulus E_{def}. The last geodetic measurement at point VD 9 was done on 10.2009, when the skeleton of the building was finished and the obtained deformation was 6.12 mm. It is expected that the final settlement will be higher, because the operational load will be acting on the foundation soil below the footing.

Another interesting fact is that the used values of deformation modulus $E_{def,2}$ instead of $E_{def,1}$ of soil replacement layer are better fitted with the measured values. More fitted must be modulus from the first loading cycle $E_{def,1}$, but the reason behind the inappropriate values can be a time factor and the gradual increase of overburden stress prior to construction of the columns on footing, which means that the partial load has already taken place and has not been captured by geodetic measurements.

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