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Analyses of historical buildings condition with respect to soil-structure interaction

L'analyse d'etat des bâtiments historiques en tenant compte de leur interaction avec sous-sol

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ABSTRACT
Analyses of geotechnical case histories are shown to have been conducted with use of numerical modelling using *FEM models* software which features various realized patterns of ground behaviour and of superstructure's response thereto taking into account the associated reciprocal effects. The possibilities of solving physical and geometrical non-linear problems in spatial setting, as well as of realizing finite element schemes featuring several million of freedom degrees makes the software an effective tool in studies of geotechnical case histories.

RÉSUMÉ
On offre d'effectuer une analyse d'etat des bâtiments historiques en utlisant des modèles numériques avec la programmerie des calculs combinés du bâtiment et son sou-sol "*FEM models*", qui travaille avec des modèles différents d'une conduite de sol and bâtiment en tenant comte de lear interaction. La possibilité de trancher des problèmes physiquement et géométriquement non-linéaires dans l'espace et de réaliser des schémes par éléments finals permet d'utiliser cette programmerie pour l'analyse précise de situations géotechniques.

1 INTRODUCTION

Retrospective study of geotechnical case histories is one of the most challenging types of geotechnical problems as a whole. As has become evident from practical expertise buildings develop deformations on account of a multitude of causative factors whose raison d'être lies in both technical and natural domains. In a number of cases the deformations are brought about by a combination of the two, in others only one of them prevails. It is not always that a particular factor or a confounded combination of the two factors permit of indubitable expert evaluation. For an appropriate 'diagnosis' to be followed by an efficacious 'prescription' one requires not only reliable site investigation data clearly delineating the ground or structure condition, but also a calculation of the adverse input on the part of the negative factors brought to light by the site investigation. When trying to establish the factors which have resulted in deformations it is critically important to furnish substantial assessment of soil-structure interaction. *FEM models* software developed by Saint-Petersburg geotechnical engineers can accommodate modelling of 'structure-foundation-subsoil' system in 3-D setting utilizing various physical and geometrical non-linear models of ground and superstructure.

Usually retrospective analyses of geotechnical case histories are conducted in the following sequence:

A – preliminary studies of the available technical documents (the design, site investigation reports, as-built drawings, etc);
B – identification of possible risk factors and draft of investigation programme for a particular site;
C – implementation of the investigation programme (site investigation, geological and geodetic investigation, condition surveys, etc) with more thorough definition of risk factors;
D – building a numerical model; numerical assessment of individual risk factors and combinations thereof, as well as of their influence on 'structure-foundation-subsoil' system.

Here we shall analyze two buildings in Saint Petersburg. They both comprise historical heritage of our city and are the most magnificent monuments.

2 ANALYSIS OF HISTORICAL BUILDINGS IN SAINT PETERSBURG

2.1 *Stock Exchange on Vasilievsky Island spit*

First we shall review the building of the Stock Exchange on Vasilievsky Island Spit in Saint Petersburg. This building which has become one of the symbols of Saint-Petersburg was constructed in 1805 according to the design of Thomas de Thomon (Fig. 1) who raised a rectangular ancient temple-style building on a granite rock stylobate formed by a system of massive pillars and walls covered with cross-vaulting. The central hall of the Stock Exchange is capped with a caissonned cylindrical canopy.

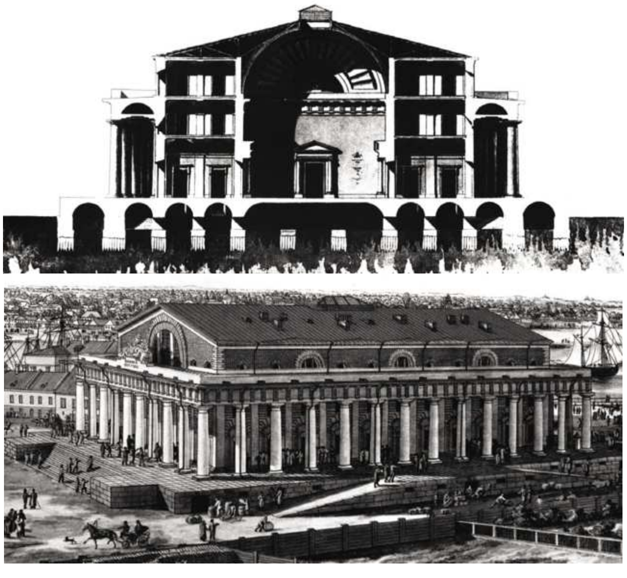


Figure 1. Former Stock Exchange on Vasilievsky island spit:
a) cross section; b) the building by the construction completion

In 2002 a large scope of works on the building's elevations was carried out. The cracks developed over two centuries of the building's life were revealed. The restorers expressed their concern regarding the renewed cracks appearing in the new superficial finishes on the gable ends and splitting the building along its longitudinal axis.

To identify the reason for the development of these deformations we studied the available historical accounts of the Stock Exchange construction, explored the site conditions, investigated the actual layout of the foundations, measured the accrued settlement differentials, analysed the principal concept behind the superstructure build-up, established the actual subsoil conditions, identified the dynamic background rendered by the nearby traffic, and, finally, conducted a series of soil-structure calculations with the account of all risk factors gleaned through all the above assessment procedures (Ulitsky, 2003).

Unfortunately no geodetic monitoring of the settlement had been previously conducted. Based on the setting-out carried out on all levels of column bases throughout the building perimeter it was possible to establish the settlement differential present between the north and the south elevations amounting to as much as 13-14 cm, whereat the settlement differential along the elevations proved negligible.

The principal feature of the ground conditions was identified as considerable heterogeneity of the soil strata (Fig. 2). The made-up ground is underlain by intermittent sand and soft clayey sand strata with some presence of loam, including a stratum of peaty clay sand whose thickness increases from 0.0 m to 2.0 m directed from north to south elevations.

Another possible factor conducive to the actual risk was a possible heterogeneity of the buildings foundations. Thomas de Thomon constructed his building on site of the pulled down Stock Exchange which had been previously built by G. Quarenghi. In construction practice of the time it was a common approach to incorporate old foundations into new structures (which method was implemented by G. Quarenghi himself).

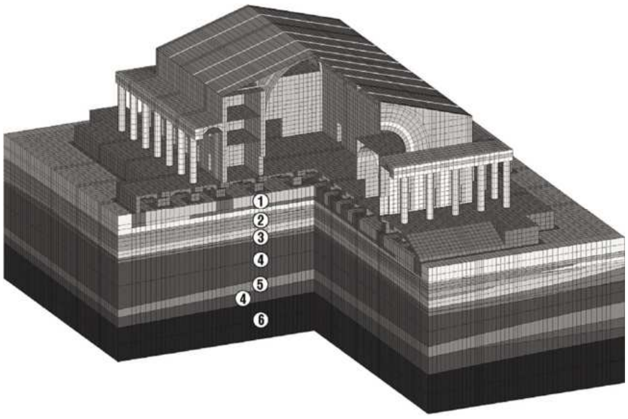


Figure 2. Stock Exchange building and its subsoil. Computer generated outlook. Soil layers: 1 – silty sand, 2 – clayey sand with inclusion of peat, 3 – soft sandy clay, 4 – semi hard clayey sand, 5 – sand with gravel, 6 – dislocated clay

To investigate the matter further we carried out a condition survey on the foundations.

The survey displayed the layout where the strip foundations of the exterior stylobate walls (compounded of granite and limestone elements) served as a strengthening embankment for the excavation pit, inside which (over a layer of timber beams) a solid limestone foundation wall was constructed supporting pillar-type rubblework foundations (Fig. 3).

The hypothesis as to discontinuity of the foundation layout was disproved. No rotting of timber beams was observed; fine sand underneath was found to be of mostly firm composition. Therefore it was possible to establish that development of the deformations in the given case was unrelated to either of the two most common causes of foundation failure in Saint-Petersburg, these being, firstly, decomposition of timber elements within foundations and, secondly, washing out of sand fines from subsoils.

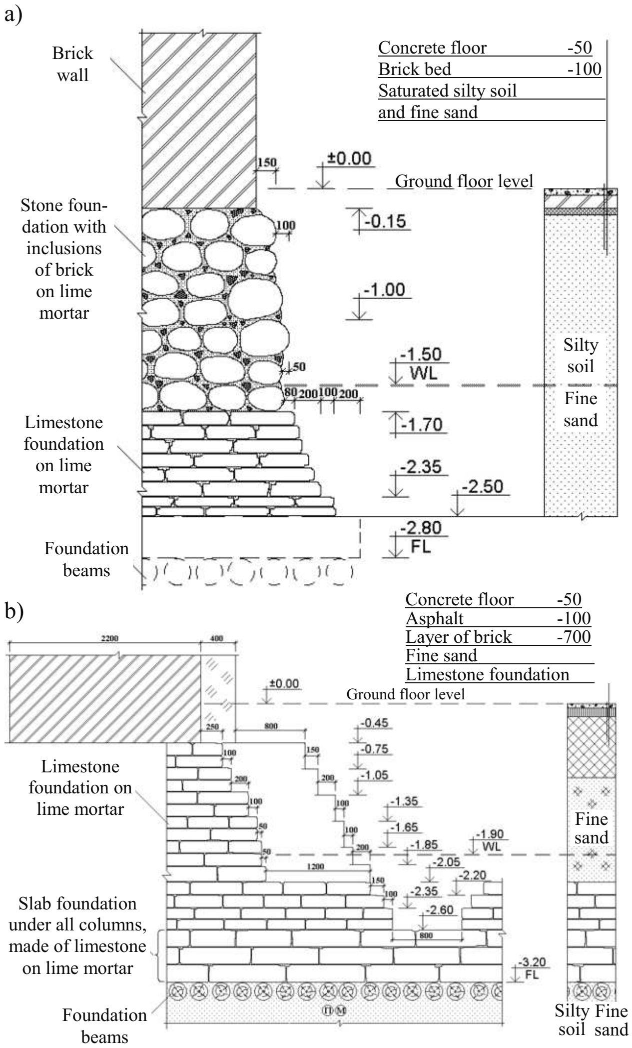


Figure 3. a) Exterior wall foundation of Stock Exchange stylobate;
b) Foundations of Stock Exchange pillars

The supposition as to unbalanced arc action in the central vault of the Stock Exchange being a contributing factor to the deformations was also discarded. Condition survey showed that the vault was a 'false' one, suspended from the consoles of the reinforced concrete trusses installed in 1914 during reconstruction of the building as per the design submitted by architect Theodore Lidval.

Finally, the list of provisional contributing factors was reduced to the only one remaining possibility i.e. non-homogeneity of the underlying ground composition. We set out to conduct a series of geophysical tests (seismotomography) which confirmed some weaker strata underneath the south part of the building.

To furnish nuerical modelling of the building's behaviour within the scope of our *FEM models* software we used high order elastic rectangular shelled elements to approximate functions of unknown displacement and angles of rotation. We also used volumetric elastic elements, as well as elastic rod elements.

Elasto-plastic soil model for numerical simulation has been chosen. This model presupposes a linear connection between stress and strain within the surface of Coulomb-Mohr criterion, as well as dilatancy-free flow of ground on the limiting surface. To create a subsoil model, spatial character of strata sequence was taken into account (see Fig. 2).

Calculations were carried out in two steps (Fig. 4). The first step featured modelling of the natural stressed subsoil conditions, the second – construction of the building. As was shown by the calculations, the overall settlement of the building throughout the entire period of its life should have been in the order of 26-44 cm. Thereat settlement differential between the corners of the building reached 9-14 cm increasing towards the south wall, which was validated by monitoring. The bulk of compressible strata taken into account was 13-15 m. The most substantial contribution to settlement differential was rendered by the stratum of peaty loam. Considering that the absolute levels of the underside of that stratum vary from – 0.8 m to – 5.3 m, the foundations of the building fail to reach any reliable support and are embraced by the peaty loam area in the south-east part of the buildings.

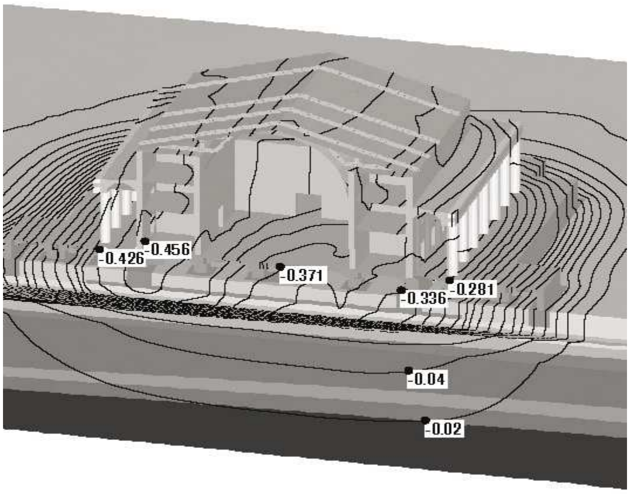


Figure 4. Contours of subsoil settlements (m)

As per the calculation results the reason for opening of the cracks lies in development of tensile stress in the upper part of the walls owing to settlement differentials (Fig. 5a). The calculation results fully agree with the actual conditions of cracking (Fig. 5b).

The calculation identified the most adverse settlement on the foundations located under the double contour of the building walls. A somewhat smaller settlement of the other sections of the foundation is responsible for development of tensile areas in brickwork vaults of the central and edge zones of the stylobate, also validated by the actually observed situation. The conclusion therefore suggested itself as to the discontinuity of the subsoil strata being responsible for the detrimental settlement differential (Fig. 5).

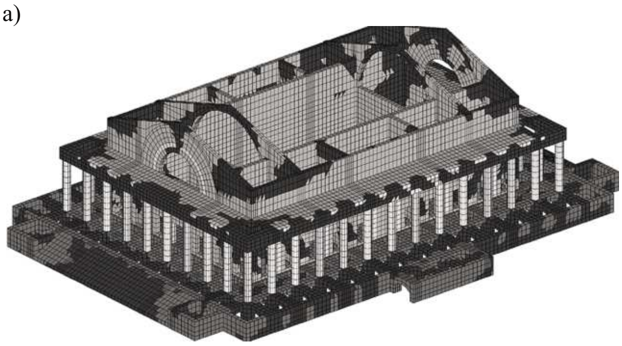
Geodetic monitoring which has been conducted by ourselves since 2002 shows the current settlement rate on the Stock Exchange approaches 3 mm a year. Such settlement rate is typical for the post-glacial strata of Saint-Petersburg, capable of long-term (so called 'secular') creep under a constant dynamic load.

Measurements of vibration generated by the passing traffic in the structures of the Stock Exchange showed vibration acceleration of 0,035 m/s^2 this value being typical for a city with a heavy traffic load. Normally, at secular creep the form of deformed subsoil is inherited (i.e. settlement profiles retain affinal similitude).

Thus, there are no grounds to expect any progressive character of deformations and no strengthening or underpinning of the building is required, provided, of course, that the present situation remains unchanged.

The building of the Stock Exchange is currently under constant monitoring. No intensifying deformations have been recently observed.

a)



b)

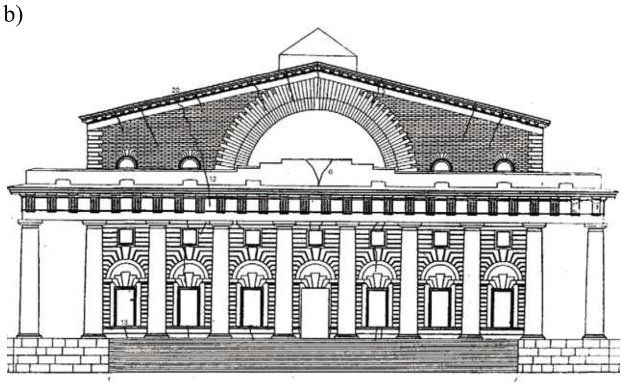


Figure 5. a) Possible locations of cracks development. Shaded areas denote major tensile stresses development; b) Cracks layout according to condition survey results

2.2 *Constantinovsky Palace in the suburb of Strelna near Saint Petersburg*

This architectural monument of 1710 enjoyed participation of the leading European architects of the time, such as J.B.Leblon, N.Micetti, F.B.Rastrelli, A.Voronikhin (fig. 6).

Figure 6. Constantinovsky Palace in the suburb of Strelna. Photo taken in 1910

Its dilapidation over past few decades was brought about by a group of contributing factors. The condition survey identified the following features. Foundations of the exterior elevation wall of the loggias as well as foundations of the retaining walls were constructed of brickwork. The brickwork was dilapidated and therefore could suffer squeezing with formation of local bulges, quite similar to the subsoil material bulging under pressure from the superstructure.

The entire brickwork structures were soaked in water conducive to dilapidation thereof through repeated frost penetration. Rotten timber pile crowns supported the transverse walls. Dilapidated terrace gutters had brought about weakening of some retaining wall sections adjacent to niches of the loggias and the grottoes. Dilapidated drainage incorporating precipitation sewer was formed by three straight courses underneath the retaining structures designed to divert precipitation and ground water from the palace. Most structural damage (fallouts) was associated with the destroyed drainage sections underlying the retaining structures (Fig. 7).

Figure 7. Dilapidated transverse cellar wall behind east section loggias (photographed in 2000)

The most considerable dilapidation was observed in locations with the initially thinnest sections of the wall (in the cross-section weakened by the vertical gutter of precipitation drainage and by local crumbling of brickwork around that gutter) in the area overcrossing the dewatering mains of the palace.

Damaged precipitation sewer caused random water discharge from the cessbox adjacent to the gable end of the cellars chamber through the ground underneath the floor and the transverse cellar walls towards the retaining wall. The retaining wall took on hydraulic pressure, which process was accompanied by excess brickwork overdampening. Owing to the damage of the horizontal waterproofing of the terrace flooring, unorganised water discharge sufficiently increased with all cellar structures being finally soaked.

Frost penetration brought about the process of superficial brickwork corrosion accompanied by frost heave pressure on the retaining wall. At the same time the vertical terrace drains started malfunctioning thus decreasing and weakening the bearing section of the retaining wall.

Seepage of water through the retaining wall in the weakened section thereof conditioned suffusion of mortar, formation of seepage passages and crumbling of brickwork around such passages during frost penetration. Movement of water through the wall occasioned a flow speed increase, as well as exceeding of critical pressure gradient in subsoil of the transverse walls and washing out of fines from under the transverse cellar walls with corresponding formation of washing out passages and cavities underneath the walls and the floors.

Washing out from under the walls led to their uneven settlement (with the highest values thereof adjacent to the retaining wall). It also resulted in subsoil surcharge and generation of additional horizontal pressure on the retaining wall.

To assess the nature of ground water movement within the slope a series of numerical experiments was conducted. The main feature of the given problem solution is incomplete definition of boundary conditions, as the surface of free current is initially unknown.

In calculations we set out to assess the influence of retaining wall dilapidation on ground water seepage regime. The rigid roof of Cambrian clays was taken as conventional confining layer. Wall dilapidation was modelled as cracks intensification and, respectively, permeability increase from 0.01 k_f up to 100 k_f, where k_f is permeability coefficient of clay sand layer. Draining systems were taken as being out of working order.

Dilapidation and permeability increase leads to corresponding increase of draining properties of the wall. At equal permeability values of the wall and clay sand the ground water level by the wall falls by 2.3 m. Water seepage will be effected both through subsoil stratum underlying the wall and through the footing of the wall itself, leading to an even stronger damage to material thereof.

The following options of structural calculations in spatial setting with the help of our software were considered to properly identify the causes of brickwork damage on the loggias and cellars and to select pertinent strengthening option:

- with piled foundation underneath cellar walls;
- in conditions of piled foundations excepted from action underneath cellar walls owing to timber piles decomposition;
- in conditions of soil wash-out from underneath cellar walls and decrease of foundation masonry strength.

Calculations were to the following effect.

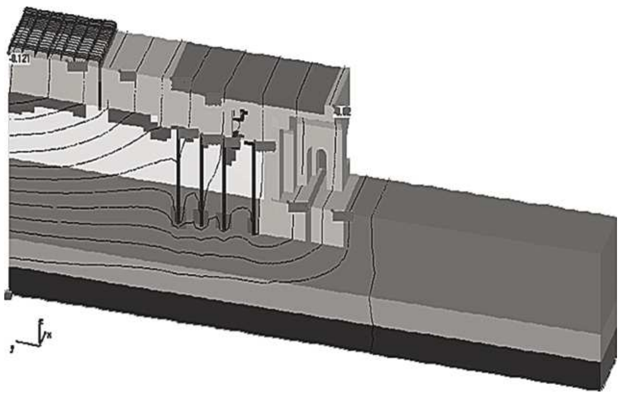


Figure 8. Calculation profile of cellars and loggias with account of timber piles

1. Provided that the piled foundation is kept intact underneath cellar walls (Fig. 8) the loggia structures remain in stable condition, with settlement of exterior loggia structures not exceeding 3.4 cm, and settlement of cellar walls not exceeding 5.8 cm.
2. In case of piles exception the cellar walls settlement reaches 7.7 cm which will serve as cause for transverse walls dilapidation.
3. In case of ground wash-out and foundation brickwork loosening settlement reaches 8.3 cm and deformation acquires appearance as shown in Fig. 8 (deformations are enlarged 500 times for the sake of impression). In that case brickwork material is dilapidated.

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