Reconstruction of Konstantinovsky Palace in a Suburb of Saint Petersburg

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ABSTRACT: *This article describes a complex approach to the condition survey and design executed as part of the remedial works on Konstantinovsky palace in the suburb of Strelna (Saint Petersburg)--one of the most beautiful historic monuments of 18th century Russia. The palace was almost completely destroyed in the 1940s. In 2000 the decision was made to reconstruct the palace, and the project for palace reconstruction was developed. The project was completed in 2003. The following important stages of the project are discussed in this paper: palace condition survey and site investigations, analyses of the palace retaining structures, design project of palace reconstruction, and phases of the remedial works.*

KEYWORDS: preservation and reconstruction of historic monuments, soil-structure interaction calculations

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

Reconstruction of Konstantinovsky Palace was one of the most important projects in Russia in the beginning of the new millennium. This work was stipulated by a ruling passed by the government of the Russian Federation, whereby the palace was to be converted into the Congress Palace. The plan for the Congress Palace project included the following:

1. Reconstruction and restoration of the Konstantinovsky Palace.
2. Reconstruction of the stable quarters, an edifice of dimensions commensurable with the main palace.
3. Reconstruction of two buildings built in the middle of the 20th century: The Engineers' Wing and the Hostelry
4. Construction of the Consuls' Village (comprising 11 cottages).
5. Restoration of the park.
6. Reconstruction of the talks chamber.

The project involved participation of 10 design firms and about 20 subcontractors. The total number of workers simultaneously present at the site reached up to 3,000 people. The entire project was completed in 1.5 years, commencing in November of 2001 and coming to a close in March of 2003.

This article describes the major works on conception and realization of the Konstantinovsky Palace reconstruction project. The authors of this paper participated in the planning and development of the project, as well as the actual reconstruction of the palace. For his involvement in the project, the first author was awarded the title Russian Civil Engineer of the Year in 2003.

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HISTORICAL BACKGROUND

Strelna Palace, more widely known as Konstantinovsky Palace, is a large palace located in the nearest suburb of St. Petersburg on the shore of the Gulf of Finland (Fig. 1-4). Peter the Great was fond of that location where he resolved to construct masonry chambers. Imperial residences in the suburbs of St. Petersburg would commonly begin as modest buildings and at a later date be expanded and reconstructed, graduating to more luxurious edifices. Strelna Palace, however, had from its inception been conceived as a stately and imposing structure. It involved leading European and Russian architects of the time, such as Jean Batiste Leblon, Nicolo Micetti, and Francesco-Bartolomeo Rastrelli.

Figure 1. Bird's-eye view of Konstantinovsky Palace and the Upper and Lower Parks in Strelna.

This palace was situated on the top of the 8-m high slope of the historical Baltic coast, the height being far from insignificant for the typically flat ground of St. Petersburg and the environs. The slope was reinforced with a retaining structure fashioned into a series of loggias (half-sphere niches with the open space towards the park). The front of the palace had been envisaged to provide a majestic fountain cascade, followed by a canal leading to the sea.

Subsequently, the great Russian reformer lost interest in Strelna and shifted his attention to the town of Peterhof as the place for establishing the official suburban residence town. The palace had lost its favour with the emperor. Its construction being very much delayed, the architect Micetti took offence and retired to his motherland to continue the creation of masterpieces there. The palace, having been constructed up to the roof level, remained uncompleted.

It was only following the accession of Empress Catherine that fortune smiled on Strelna Palace once more, and Rastrelli was commissioned to complete its construction. However, the court never moved into the new residence. The luxurious palace was again forgotten for 50 years and, as the case usually is with abandoned buildings, it was decaying quickly due to lack of maintenance and heating.

In 1802, the new owner, Emperor Paul presented the palace to his son Constantine, whereupon it became known as Konstantinovsky Palace. Refinishing of the palace was completed in 1 year. It was designed and supervised by A. Voronikhin. The sumptuous abode of the Grand Duke stood open to welcome its new owner. Fate, however, had no remorse as a large fire broke out on December 28, 1803, destroying the entire artistic decorum of the hapless building. Everything was to be renovated by architect L. Ruska. Architect A. Voronikhin designed a series of grottoes. The roof of

the grottoes served as a spacious terraced square facing the palace. The structure of the terrace at the same time functioned as a retainer for the palace, conditioning stability of the entire palatial complex.

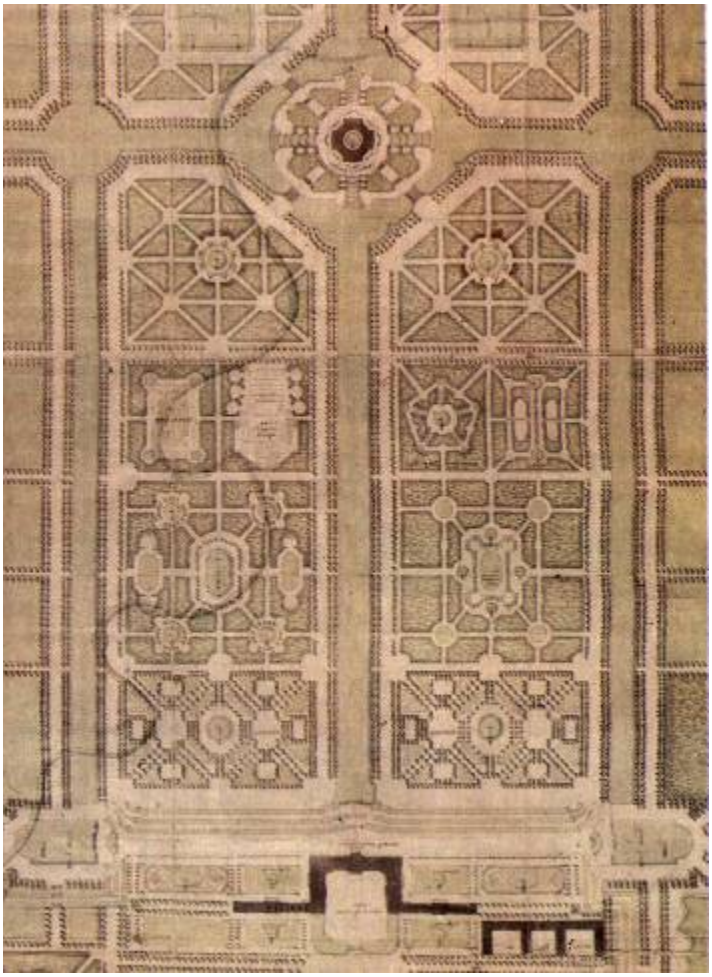

Figure 2. Plan of Konstantinovsky Palace in Strelna. Drawing by Jean Batiste Leblon. Beginning of the 18th century.

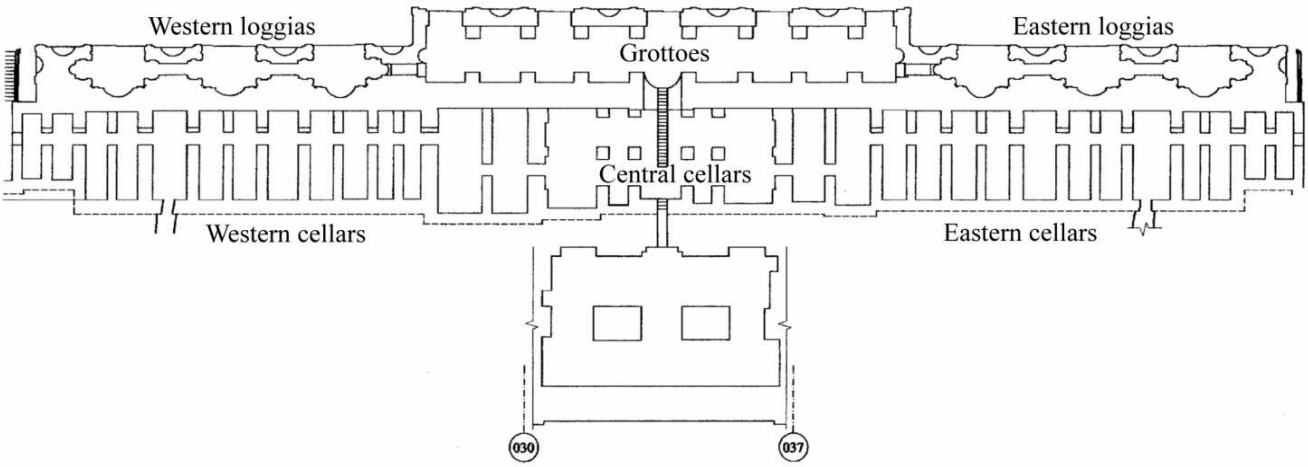

Figure 3. Plan of cellars, grottoes, and loggias of Konstantinovsky Palace.

The palace was destroyed during World War II, with just the walls remaining in place. During the post-war period the only parts restored to their original condition were the external walls, facades and the two reception halls. More recently, destructive tendencies have prevailed (Fig. 6). The most significant misfortune was the failure of the precipitation sewer. Water from the roof and the terrace found its way into retaining walls, bringing about dampening and partial collapse of the structure. The bearing timber piles were decomposing, and the stone vaults of the terrace suffered from significant masonry fallouts.

Figure 4. A photograph of Konstantinovsky Palace taken in 1910.

PALACE CONDITION SURVEY AND SITE INVESTIGATION

Strelna Palace is a 3-story building on a high basement floor (socle floor). It was constructed on linear rubblework foundations on the crest of a natural slope which descends into Lower Park by three tiers. The natural slope in front of the palace was fashioned into a horizontal area 23 m wide in the middle and 17.3 m wide on the edges. The absolute level of the terrace surface is at 12.7 m Baltic Datum (BD) (see Fig. 4, 5, 8). The vertical terrace ramp (8.0 m high) is retained by a complex system of masonry structures forming grottoes and lateral loggias on the front elevation (Fig. 3), as well as the suite of wine cellars between the grottoes and the palace. The grotto is divided throughout its length into 9 equal bays, each approximately 4.75 m in length.

Symmetrically on each part of the central grotto there are 3 loggias (see Fig. 3 and 4). The gable wall for both the grottoes and the loggias is the actual retaining wall. The loggias retaining wall contains half-sphere niches forming the volume of every loggia. In these locations the retaining wall is especially thin (around 1.5 m), but gradually increases up to 3.2 m elsewhere.

Behind the retaining wall there is a suite of basement premises (former wine cellars) with the absolute floor level of 8.7 m BD. These premises are vaulted with cylindrical brickwork arches supported by transverse walls. The transverse walls are located both in the middle and on the sides of each loggia.

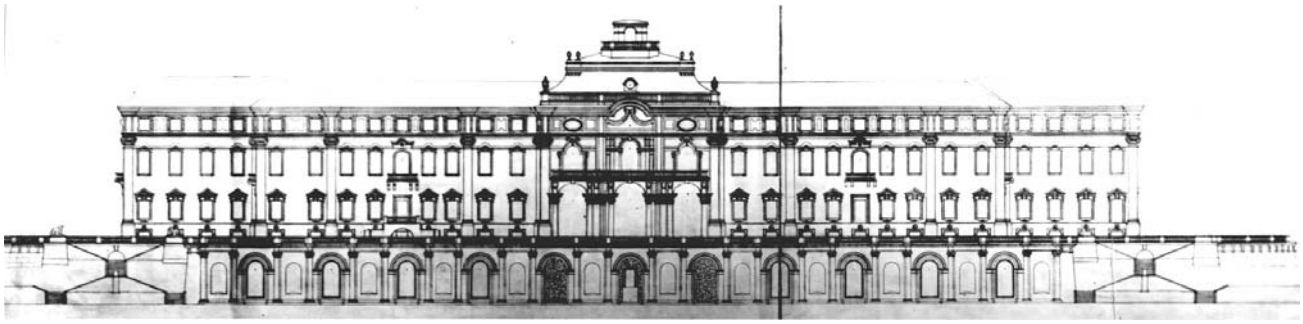

Figure 5. A drawing of the palace made in the first half of the 20th century.

The retaining structure layout of Konstantinovsky Palace in Strelna is a unique feat of engineering of the 18th and19th centuries. It serves not only as a podium for the palace on the Lower Park side, forming a spacious terrace in front of its north elevation, but also as a structure ensuring the building's stability on the brink of an 8-m slope at the historical coast of the Baltic Sea. Stability of the entire palace depends on the technical condition of its retaining structures.

The authors were commissioned by KGIOP (Governmental Monument Preservation Authority) to provide a pertinent condition survey of this monument or, more precisely, of the structure's areas of critical dilapidation (Fig. 6, 7). What the surveyors saw was an abandoned palace gracing a high slope, strengthened by a retaining structure fashioned into a series of grottoes and loggias. The principal bearing wall, withholding the ground on the slope and the palace on top of it, was considerably damaged in a number of locations. Water had found its way inside, penetrating through fall-outs over piles of brick rubble. Later, as cold weather set in, the water was transformed into ice. Ice stalactites hung on the precipitation drainage (which used to play an important role in dewatering the terrace).

Figure 6. A photograph of Konstantinovsky Palace taken in 2000 before reconstruction.

The condition survey had to assess the scale of structural dilapidation, identify its causes and mechanism, design ways to eliminate future deformations, and define the methods of bringing the retaining structure back to reasonable functionality.

Complexity of that salvage situation was increased due to the necessity to provide a celebrity vestibule in the basement underneath the central triple arch of the palace and in the grottoes underneath the terrace. For the reconstruction of the subterranean space to be effective, it was necessary to conduct extensive investigations, surveys, and analyses within strict time constraints. The structural layout and condition of all foundations, both of the palace and the retaining structure, were studied and described. To accomplish this, 28 trial pits were excavated, 35 boreholes were drilled through foundation

masonry courses, 2 large trenches were excavated on-site, the rigidity characteristics of brickwork were established, moisture conditions of the walls were studied, and the length of timber piles underneath rubblework foundations were defined.

Figure 7a. Dilapidated structures of the palace: grotto.

Figure 7b. Dilapidated structures of the palace: transverse walls.

The key issue in ensuring long-term palace and retaining structure preservation was solving the dewatering problem. The dewatering problem was the primary cause of the retaining structure's dilapidation in the first place. It was necessary to compile a detailed historical analysis of the dewatering system (both for groundwater and precipitation water), as well as to define the optimal configuration for the dewatering system to be reconstructed. Bearing in mind the location of the palatial complex on the crest of a natural slope, all contributing architects paid special attention to groundwater and precipitation dewatering. Stability and long life of the entire retaining structure wholly depended on an effective solution in that area.

The old dewatering system had been constructed in the form of a continuous collector made of brickwork courses "for the purposes of intercepting rain water collecting on the entire palace roof, and drying the basement areas" (City Commission on Construction, 1849). Apparently, the contour brickwork collector alongside the palace perimeter was responsible for

collecting the water discharged from 35 precipitation drainpipes, whereas the function of the intercepting collector in the north was to divert the water into the canal.

In the west and the east away from the palace in the bulk of the natural terrace ramp, a diverse network of masonry piping was found preserved from times when workers had tried to use it to arrange numerous gallery fountains in the area. The remnants of these pipes were exposed during a palace reconstruction attempt in 1950, and also in 1985 when heating mains was being laid adjacent to an outhouse near the palace.

Dewatering was originally realized in three cardinal points: north, west, and east. It was fashioned as a network of subterranean collectors supplemented with open gutters. In the north, the system was laid as three lines leading from the corners and the centre of the palace to the canal, where it had some vulnerable points in locations of height differential. The historical drainage system in these locations of the gallery collectors consisted of level differential or high-speed water flow installations, such as rough surface gutters on whose finishing courses were installed devices to prevent damage to the network. It is necessary to point out that the old drainage system was mainly precipitation oriented. In the upper terrace area which was compounded of clay soils and characterized by surface water flow, that network exclusively acted as precipitation dewatering (Kliorina, 2004).

To divert precipitation from the terrace surfaces (level 12.7 m BD) above the loggias and the grotto, there were 12 funnel-type water receptacles (6 above the grotto and 1 above each loggia). Water was diverted through rectangular ducts constructed of timber planks and embedded into grooves in the brickwork of the retaining walls by one brick-width.

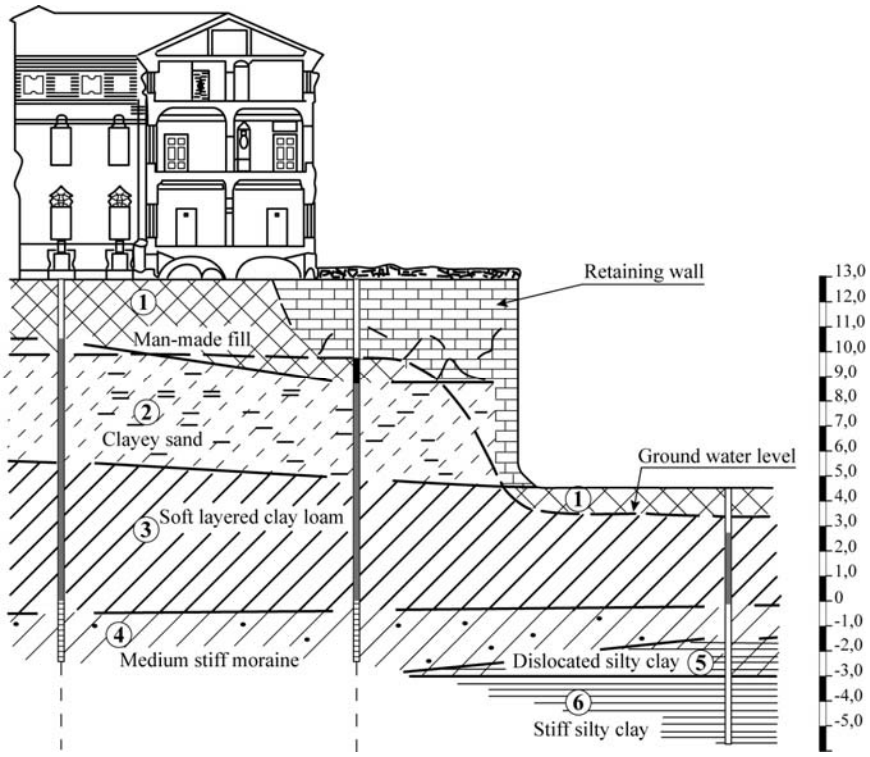

Figure 8. Cross-section of the palace and subsoil profile: 1. Made-up fill, 2. Clayey sand, 3. Soft varved clay loam, 4. Medium-stiff moraine clay loam, 5. Dislocated stiff Cambrian silty clay, 6 . Stiff silty Cambrian clay.

As attested by site investigation in the park, underneath 1 m of fill there is a 3-m layer of soft varved clay loam, underlain by medium stiff moraine clay loam (Fig. 8). Still deeper, at the level of around 14.0 m (absolute – 1.3 m), there are medium stiff and stiff deep Cambrian clays.

The slope incorporating the terrace is compounded by lacustrine-glacial clayey sands followed by silty lacustrine-glacial loams. Straight upon the terrace there is a stratum of man-made ground of sand with admixtures of lime cement, above which there is 2 m of brickwork (in the section between the palace and the cellars) serving as a base for the terrace. It could

not be entirely ruled out that the discovered brickwork is demonstrative of a bond between the courtyard foundations and the retaining structure. Above the clayey sand there is a layer of man-made ground about 3 m thick (Fig. 8, layer 1).

Hydro-geological conditions are characterized by the presence of groundwater associated with man-made strata and silty sand inclusions in lacustrine-glacial clay loams which together with clayey sands act as a confining bed. Groundwater is discharged into the Lower Park canal. The groundwater table generally follows surface geometry.

In short, the subsurface of the structures in question is mainly associated with silty clay-like soil that is 3.5 m and 10 m deep in the lower and the upper levels, respectively. The peculiarity of the hydro-geological conditions is mainly that there is considerably lower vertical seepage flow in comparison to horizontal, which means that when water moves towards the discharge point there is pressure on the retaining wall.

Soil properties are given in Table 1.

Table 1. Soil properties.

Layer No	Soil	Natural water content W	Void ratio e	Unit weight γ (kN/m^3)	Angle of internal friction φ	Cohesion C (MPa)	Young Modulus E (MPa)	Permeability ratio k_f (m/day)
2	Clayey sand	0.26	0.721	19.7	24	0.017	9.8	0.07–0.10
3	Soft varved clay loam	0.35	0.957	18.8	16	0.019	6.2	0.05
4	Medium stiff moraine clay loam	0.14	0.423	21.8	27	0.020	24.6	0.02-0.03
5	Dislocated stiff Cambrian silty clay	0.22	0.660	20.2	15	0.025	15.2	0.001
6	Stiff silty Cambrian clay	0.19	0.554	21.0	21	0.028	22.2	0.001

CONDITION SURVEY RESULTS

The condition survey confirmed the most negative expectations, postulating a threat of collapse for the historic palace. It was not so much the condition of the building itself but the retaining structure that instilled considerable reservations about the building's life expectancy. Although the condition of the building seemed adequate, the risk of failure appeared to be very significant due to the critical condition of the retaining structure.

The condition survey results were as follows (see Fig. 6, 7, 9, 10):

1. Foundations of the dilapidated retaining walls were constructed of bricks. The foundations were no longer capable of being classified as a structure. There was imminent danger of crushed brickwork movement with formation of local bulges.

2. The entire brickwork structure was soaked in water, causing dilapidation through cycles of freezing and thawing.

3. There were no foundations of the transverse walls of the cellars. Footing was level with the cellar's floor. Decomposed timber pile heads supported the transverse walls.

4. Dilapidated terrace gutters had caused weakening of some retaining wall sections adjacent to niches of the loggias and grottoes.

5. The precipitation sewer consisted of three straight courses underneath the retaining structures, designed to divert precipitation and groundwater from the palace. There was water flow through the ground underneath the cellar walls, and through dilapidated retaining wall sections.

6. Most structural damage (fallouts) was associated with the destroyed drainage sections underlying the retaining structures.

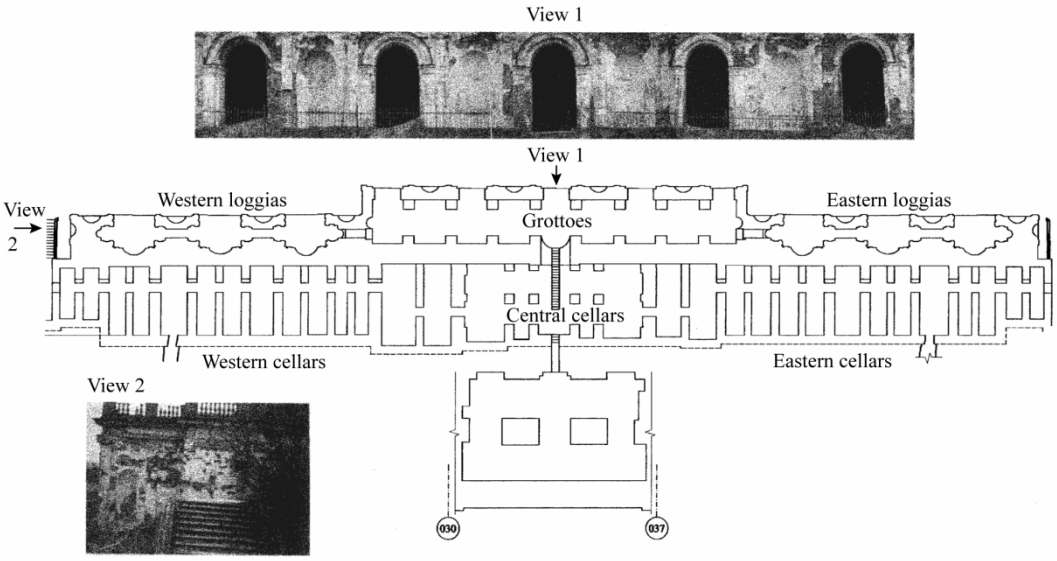

Figure 9. Dilapidated cellars, grottoes, and loggias of Konstantinovsky Palace.

The dilapidation mechanism can be presented as follows. The damaged precipitation sewer caused water discharge from the precipitation water collector 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 drainage pipe under the floor alongside the retaining wall stopped functioning, increasing hydraulic pressure to the retaining wall, and over-dampening the brickwork. Because of the damage to the horizontal waterproofing of the terrace floor, percolating water sufficiently increased, and all cellar structures were finally soaked.

Frost penetration brought about the process of brickwork corrosion accompanied by frost heaving pressure on the retaining wall. At the same time, the vertical terrace drains started malfunctioning, thus thinning and weakening the bearing section of the retaining wall. Cycles of frost penetration into damp brickwork caused destruction thereof by low temperatures. Countless traces of such destruction were observed throughout. Intermittent water discharge into the ground in front of the retaining wall resulted in degradation of the timber pile heads supporting the transverse cellar walls, bringing about their settlement.

Figure 10. Collapsing cellar wall behind east loggias.

Seepage of water through the retaining wall in the weakened section caused suffusion of mortar, formation of seepage passages, and crumbling of brickwork around such passages during frost penetration. Movement of water through the wall caused a flow velocity increase, as well as higher than critical pressure gradients in the subsoil of the transverse walls, washing out of fines from under the transverse cellar walls with corresponding piping, and formation of cavities underneath the walls and the floors.

Piping from under the walls led to their uneven settlement. It also resulted in the subsoil surcharge and generation of additional horizontal pressure on the retaining wall. Gradual dilapidation of the retaining wall brickwork finally resulted in the palace's partial collapse in locations where its bearing section was most weakened by the formation of gaping holes and fallouts (ground and dilapidated brickwork suffusion cones) in the loggia niches. The above description of the dilapidation mechanism was verified by means of detailed geotechnical analyses.

ANALYSES OF THE PALACE RETAINING STRUCTURES

The purpose of the analyses was to: (1) assess the influence of retaining wall dilapidation on groundwater seepage regime, and (2) estimate the stability of the retaining wall.

The Cambrian clays were considered a conventional confining layer. Water pressure corresponded to groundwater levels established by the site investigation. Wall dilapidation was modeled as a local intensification of cracks and a permeability increase from 0.01 k_f up to 100 k_f, where k_f is permeability ratio of clayey sand layer with (k_f = 0.07–0.10 m/day) (see Fig. 8 and Table 1, soil layer 2). Draining systems were assumed to be out of working order.

Analyses demonstrated that at low permeability of the wall, equal to 0.01 k_f, the wall is capable of functioning in the capacity of a confining layer. It this case, maximum decrease of groundwater level drop behind the wall was 30 cm. Seepage velocity values in the wall are practically zero. If there wasn't any waterproofing behind the wall, the brickwork would be constantly damp.

An increase in dilapidation and permeability leads to a corresponding increase in draining properties of the wall. Water seepage will go on both through subsoil stratum underlying the wall and through the lower part of the wall. This will lead to even greater damage to the material. Further permeability increases of up to 100k_f are conducive to an increase in seepage, and current velocity is realized through the underside of wall, causing its further dilapidation.

The following options of structural analyses were considered (in 3 dimensions) to properly identify the causes of brickwork damage in the loggias and cellars and to select a pertinent strengthening option:

1. with piled foundation underneath cellar walls;

2. without piled foundations underneath cellar walls to account for the timber pile's decomposition;

3. in conditions of soil wash-out from underneath cellar walls and decrease of foundation masonry strength.

To account for the above factors it was necessary to perform soil-structure interaction calculations of subsoil, foundation structures, and superstructures. The calculations were made with the help of FEM models, a software developed by the authors (Ulitsky et al 2003, Shashkin, 2006). This software allows for the calculation of joint stress-strain condition of the superstructure and subsoil by using the finite elements method in 3 dimensions. Numerical analyses are not presented in detail since this is beyond the scope of this paper.

An elasto-viscoplastic hardening model (Shashkin, 2006) was used to model the non-linear soil behavior, with the limit state surface described by Mohr-Coulomb criterion. Soil properties used in the analyses are provided in Table 1.

Analyses were conducted in two stages: In the first stage, the state of stress under the ground's self weight was calculated. In the second stage, deformations resulting from the weight of the superstructure on the subsoil and foundation were calculated. These calculations showed that:

1. Provided that the piled foundation is kept intact underneath the cellar walls (Fig. 11, 12) and the loggia structures remain in stable condition, with settlement of exterior loggia structures not exceeding 3.4 cm, and the settlement of the cellar walls not exceeding 5.8 cm, the building would be stable.

2. In the case of the decomposed timber piles, if the cellar wall settlement reached 7.7 cm it would cause the transverse wall's dilapidation. Due to decomposition, the piles were considered inadequate to support the required loads and thus were not included in the analyses.

3. In the case of ground piping and foundation brickwork loosening, if the settlement reached 8.3 cm and the deformation profile resembled that shown in Fig. 13, then the brickwork material would be dilapidated. Observed brickwork dilapidation suggested that this mechanism was actually occurring at the site.

Therefore, the conclusion was that the retaining structure was critically damaged and it would be necessary to carry out complex remedial works.

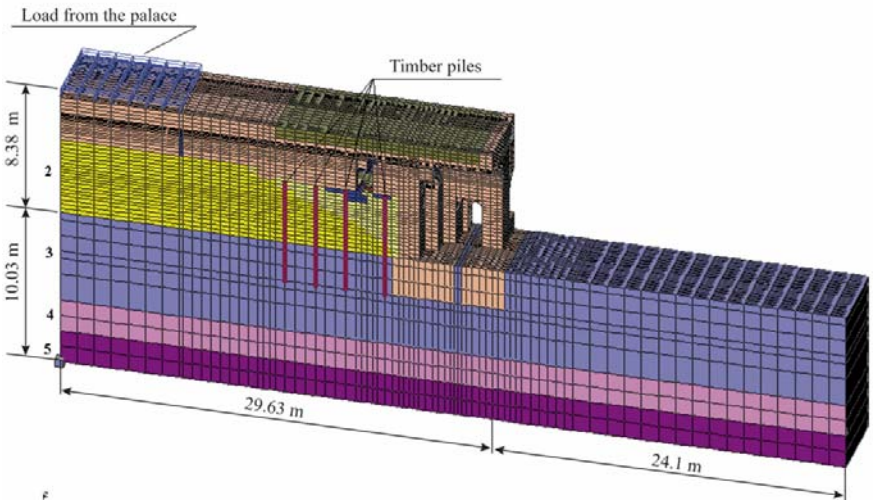

Figure 11. Calculation profile of the cellars' and loggias' deformation with account of intact timber piles.

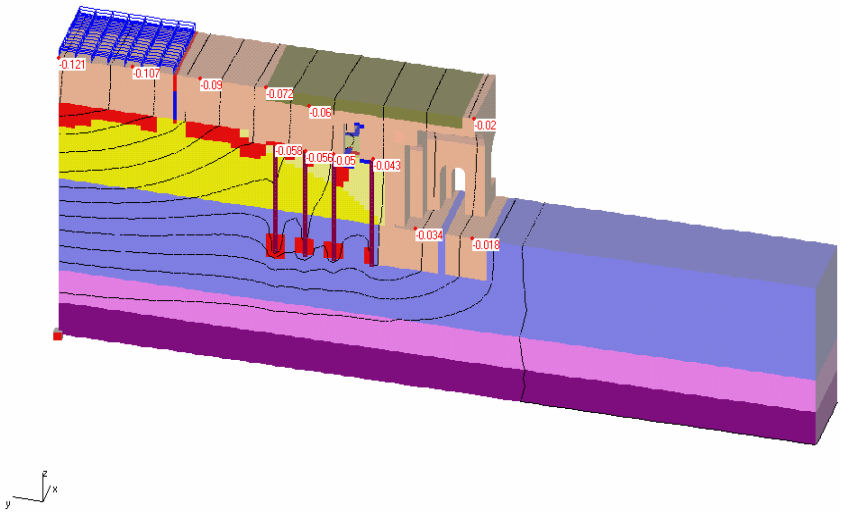

Figure 12. Contours of settlements of the cellars and loggias (m) with account of intact timber piles. Dark red color denotes soil regions reaching the limit state by Mohr-Coulomb criteria.

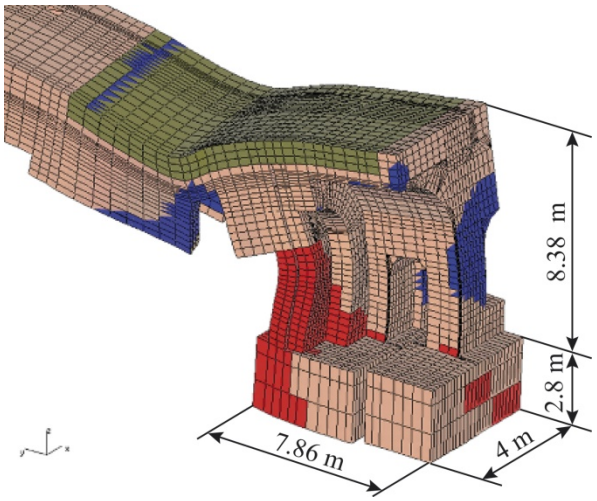

Figure 13. Deformation scheme (scale of deformations enlarged 500 times) and limit state areas in brickwork (defined by Mohr-Coulomb criteria) accounting for subsoil ground wash-out and decomposed timber piles. Shaded sections denote limit state areas generated by compressive and tensile stresses.

DESIGN PROJECT OF PALACE RECONSTRUCTION

The Palace condition demanded immediate rendering of complex strengthening works. Such strengthening had to ensure the following:

1. Reconstruction and subsequent preservation of the brickwork;
2. Ability of the retaining wall to sustain horizontal ground pressure;
3. Reliable load transfer onto incompressible subsoil strata to eliminate any subsequent settlement-related deformations which may have arisen owing to dilapidation of the retaining structures and foundations.

The following circumstances had to be taken into account when implementing the above:

1. Complete deterioration of brickwork foundations into a crumbly mass with clayey filling;
2. Most probable prevalence of the same condition on a considerable portion of the subterranean retaining wall;
3. A retaining wall thickness of 3.2 m;
4. The absence of foundations underneath transverse cellar walls.

In such circumstances, the possibility of any local patchwork or consecutive (bay-by-bay) progress of works was precluded by the unsatisfactory condition of the brickwork, danger of local collapse of structures, and complete unavailability of materials for local replacement of the brickwork of those retaining wall sections which were in immediate contact with the ground bank.

In light of the overall surface dilapidation and general weakening of the brickwork, the option of bandages, confining frames, or other structural reinforcement was considered impractical for providing safe retaining structures and would damage the appearance of the historical building. Therefore, the only option that would address all the issues listed previously appeared to be pressure grouting and strengthening of brickwork, with underpinning of all retaining structures with piles embedded into stiff stratum (Fig. 14, 15).

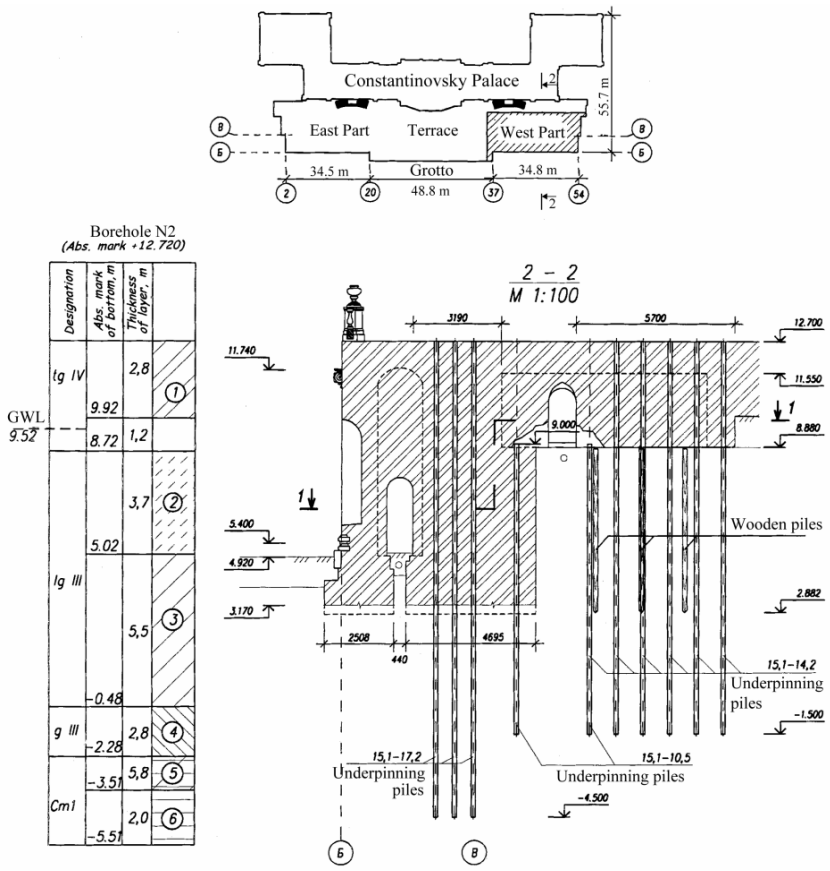

Figure 14. Palace layout in plan - Underpinning of the retaining structure. See Fig. 8 for soil stratification.

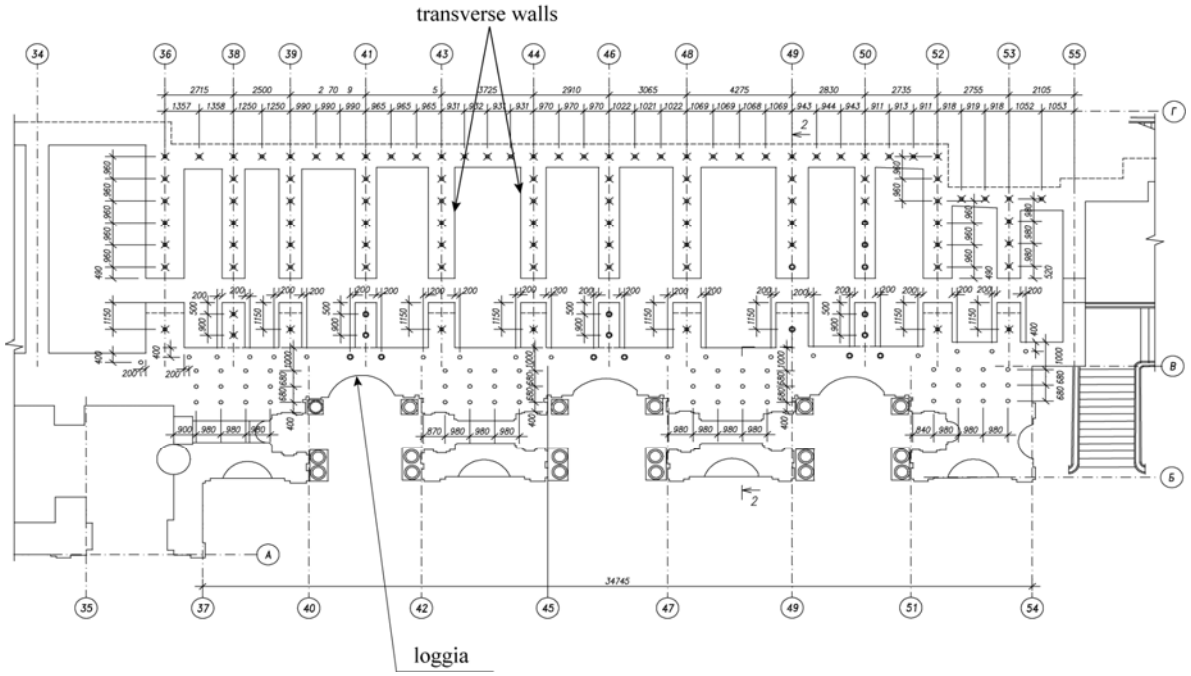

Figure 15. Location of underpinning piles in plan: o , x - underpinning piles.

Grouting of the brickwork was necessary in order to restore its strength and stiffness. Strengthening was required to properly allow the transfer of loads from the entire structure. Finally, underpinning piles had to be constructed in order to transfer the structure loads onto the incompressible subsoil stratum. It needs to be noted that conventional underpinning piles installed at an angle from the level of the lower terrace (around 4.0 m Baltic Datum) would prove ineffective as the dilapidated foundation brickwork was incapable of accommodating the heads of the underpinning piles, and the retaining wall itself was practically unavailable for underpinning.

Based on the above, the foundations underpinning of the retaining structures was carried out in the following sequence (see Fig. 14, 15).

Phase One

1. Strengthening of the critically dilapidated structures (3 left and 3 right loggias).

2. Provision of temporary propping scaffolding in cellar chambers installed on wedges in the cellar floors. Wedging of the scaffolding was regularly inspected. The unsupported spans in locations of the brickwork fallouts were likewise propped.

3. Drilling of 42 mm vertical bores above the partitions of the retaining wall from the terrace in front of the palace down to the brick-wall footing level. Subsequently, the brickwork was grouted by intervals with packing lime mortar until completely permeated.

4. Redrilling of the bores by 151-mm core bores down to the top of the firm Cambrian stratum following 70% setting of the mortar. Drilling below foundation footing was either carried out using thixotropic grout or was casing protected. Cement grout with added plasticizing and shrink-proofing agents was pumped into the subsoil and brickwork at 0.2 and 0.1 MPa respectively, followed by a stain-proof reinforcement casing tube being oscillated into the grout mix. The resulting pile was thus constructed with its toe against the stiff Cambrian stratum reinforcing and underpinning the entire retaining brickwork section. The tube was required to ensure both longer pile life and subsequent possible of deepening of the cellars. Toe levels and bearing of the piles had been previously confirmed based on the static loading tests.

5. Drilling of 42 mm vertical bores paced at approximately 1.0 m from the terrace in front of the palace along each transverse wall down to the brick-wall footing level (absolute level 8.9 m BD) in order to reinforce the transverse walls and rear longitudinal wall of the cellars. This was followed by interval grouting and subsequent redrilling of the bores by 151-mm augers, used in the bored piles construction, down to absolute level of 1.5 m BD. Those piles were likewise reinforced through their entire length.

6. Construction of pile heads in the dilapidated areas of the transverse walls at the level of the brickwork footing (absolute level 8.8 m BD), with subsequent construction of the pile caps and masonry courses within the original scope.

Phase Two

1. Completion of the retaining structure strengthening works.

2. Provision of works described in Stage 1 above for unreinforced sections of the walls.

3. Removal of terrace surface material and construction of a reinforced concrete wall connecting pile heads above the brick vaults.

Phase Three

1. Provision of the terrace surfacing incorporating drainage and snow melting systems, finished by tiling.

2. The provided strengthening should serve to ensure the reliability and long life of the retaining structures, preserving their appearance and historic materials almost completely unscathed by any patchwork or replacement of brickwork. Such approach proved most appropriate in relation to this important architectural monument.

3. The constructed strengthening option was successful even when faced with an unexpected challenge. The architects suddenly decided to provide front access to the palace from the Lower Park and furnish a vestibule underneath the terrace. To do this, all cellars had to be deepened by 1.0-1.5 m and the transverse brick walls were temporarily suspended on the thin underpinning piles (Fig. 16, 17, 18). Quality of the strengthening was attested by the fact that not one section of the brickwork was in any way displaced. It was therefore ascertained that the strengthening was successful.

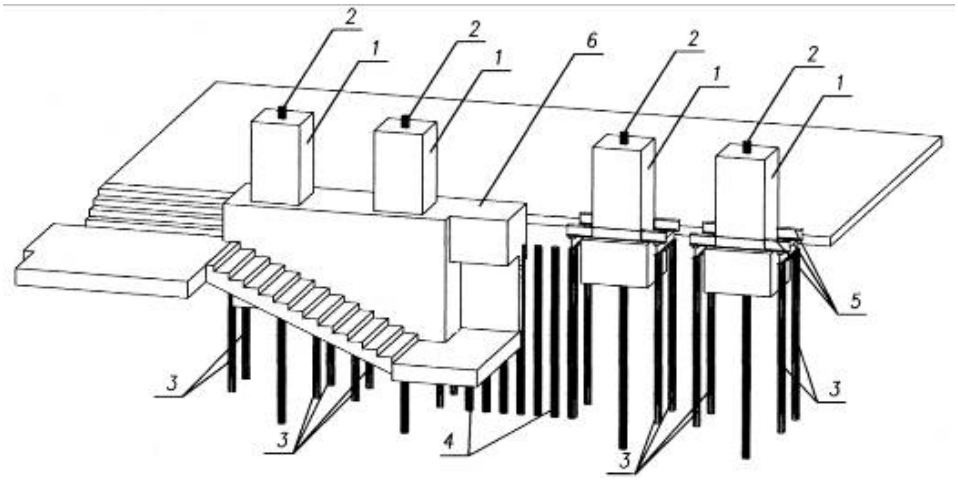

Figure 16. Provision of celebrity entrance overlooking the Lower Park (deepening of the basement down to 5.0 m): 1. existing brickwork pillar, 2. underpinning piles constructed from terrace surface, 3. pile supports for propping brickwork pillars, 4. pile wall, 5. steel waling transmitting loads form pillars onto piles, 6. retaining wall constructed of reinforced concrete.

Figure 17. Deepening of the basements by 1.0-1.5 m.

Figure 18. Bored pile with tube reinforcement viewed from underneath foundation.

Figure 19. Konstantinovsky Palace, south façade. View of a final reconstruction stage (January 2003).

Figure 20. Cellars of Konstantinovsky Palace, 2004 (at the location of reconstructed retaining cellar wall, see Fig. 10).

It took about 1.5 years to completely reconstruct Konstantinovsky Palace in Strelna (Fig. 19). The palace officially opened as the Congress Palace in 2003 during the tercentenary celebrations of Saint Petersburg (Fig. 20, 21).

Figure 21. Konstantinovsky Palace, south façade. View after a final reconstruction stage (July 2003).

CONCLUSIONS

Konstantinovsky Palace, an important historic monument of the early 18th century, underwent almost complete destruction in the first half of the 20th century. Later it was partly restored, but became seriously dilapidated during the past 15-20 years.

A thorough condition survey of the palace showed that due to the dilapidation of the drainage system, the retaining structures had been seriously weakened. Old timber piles had been almost completely decomposed. These factors compromised the structural integrity of the palace and endangered its stability.

Soil-structure interaction analyses of the retaining structures were conducted taking into account the subsoil piping, foundation brickwork loosening, and decomposed state of the timber piles. The calculations showed that the retaining structures of the palace were in critical condition and in need of immediate strengthening.

The complete reconstruction project of Konstantinovsky Palace was developed on the basis of a condition survey and soil-structure interaction analyses. This project envisaged strengthening of the critically dilapidated structures of the palace by pressure grouting of the brickwork and underpinning of the entire range of retaining structures with piles embedded into a stiffer Cambrian clay stratum. The geotechnical part of the project presented the highest challenge and ensured success of the whole project.

The project was successfully realized in 2000-2003, and Konstantinovsky Palace was opened in 2003 during the tercentenary celebrations of Saint Petersburg.

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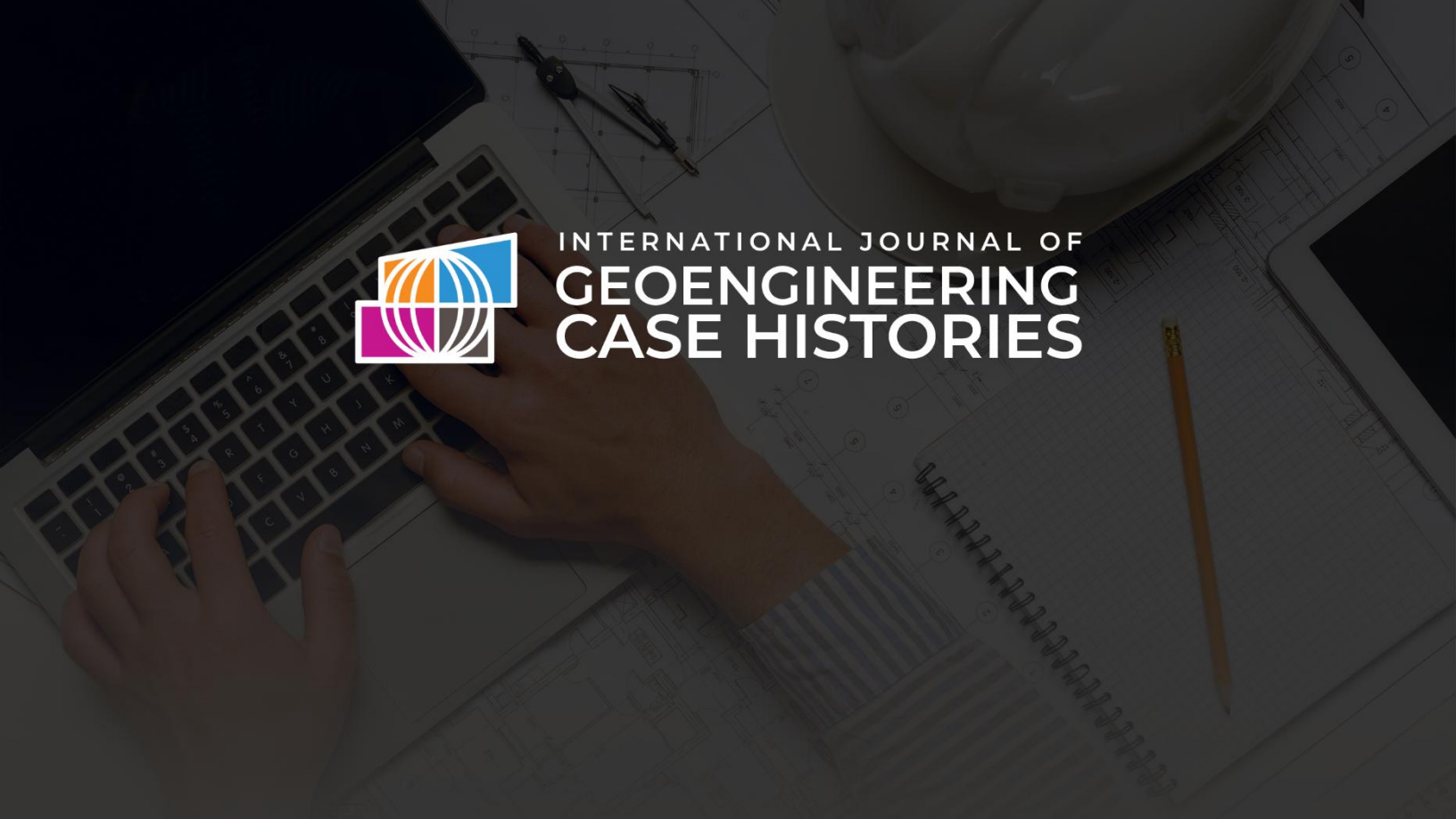

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