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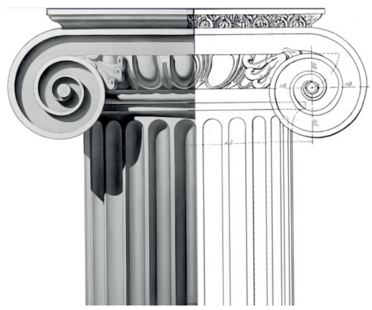

CONTENTS

EDITORIAL NOTE

Through the centuries of human civilization organized societies have advanced by constructing various works, combining ingenuity with aesthetics, in order to fulfil specific needs. Apart from defence works, that were engineering projects of the utmost importance, other specific needs refer to housing and transportation (buildings / bridges / embankments / tunnels / harbours of various forms and complexity etc.), water management (aqueducts, dams etc.), places of worship (temples and churches), energy related projects and so on. In all these works, especially the more advanced and refined, architecture was combined with engineering in order to ensure in the best possible way the functionality requirements together with aesthetics in terms of colour, space and compatibility with the surroundings. Thus, this new journal focusing on research issues in architecture and engineering has a far reaching focus.

An early example of such a project is the Minoan palace at Knossos in Crete-Greece that was built in 1900 BC and was destroyed, probably by an earthquake, about 1700 BC. A second larger palace was built with its remains depicted in the figures below after the decline of the Minoan civilization that dominated the East Mediterranean region for some time in antiquity.

The remains of the palace of Knossos in Crete-Greece

As can be seen in this example there are important aspects that all projects must comply with; that is structural stability for service as well as extreme loads, durability of all materials and parts and effective maintenance. More recently, new experimental facilities and computer simulations allow us to analyse complex past or contemporary architectural formations in such a way as to better safeguard their structural stability for service as well as extreme loads. Technological advances can allow us to efficiently monitor the deterioration of various parts of a project. Furthermore, new materials and techniques enable us to design, construct and effectively maintain such projects and prolong their life span. This is obviously very important for projects we would like for many reasons to preserve. Despite recent advances in all these issues there are many relevant important aspects that are still in need of research. Moreover, as our capabilities are enlarged, the undertaking of constructing more daring projects becomes feasible; this subsequently generates a new set of problems that need to be solved regarding structural stability for service as well as extreme loads, durability of all materials and parts and effective maintenance. The exchange of information through scientific publications on all the above issues is of extreme importance. Thus, I believe that the publication of relevant research manuscripts in this online journal in English with the title “Architecture and Engineering”, under the auspices of Saint Petersburg State University of Architecture and Civil Engineering of the Russian Federation, conforms with such a necessity and will serve such a purpose.

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BENDING OF SAWN WOOD PRODUCTS OBTAINED FROM CONVENTIONAL SAWING AND PARALLEL TO GENERATRIX SAWING

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Abstract

This paper considers bending of sawn wood products obtained from logs by conventional sawing and sawing parallel to generatrix. Issues related to strength and stiffness of sawn wood products sawn parallel to generatrix have not being studied yet, thus such issues are of practical and scientific interest. Earlier our investigation dealt with changes in tilting angle of wood fiber of sawn wood materials obtained from logs by conventional sawing and sawing parallel to log generatrix.

This concept is used for generating of mathematical models describing changes in elasticity modulus across the wood plank width which enables to foresee the wood plank bending strength and stiffness.

The proposed procedure allows for the evaluation of wood planks bending stresses occurred when various log sawing schemes were used. Wood planks bending value is calculated to cover the difference between stiffness of wood planks obtained at the implementation of both sawing schemes.

The investigation results are not only of scientific interest, but are also of practical interest in terms of construction and fabrication of laminated wood structures, wood structural panels etc.

Keywords

Bending, stiffness, elasticity modulus, stresses, strength.

1. Introduction

Wooden house construction becomes the object of great interest under the current economic conditions. Wood is considered as anisotropic material which properties could ten times differ one from another in terms of different structural directions (Ashkenazi, 1978; Glukhikh, 2007). Generation of wood structure as a center of equal strength facilitated improvement of mechanical and physical characteristics longitudinal to the fibers ten times greater than those transverse to the fibers. Ideally, the wood fibers in sawn wood products are to be parallel to the edges and sawn faces of sawn wood products and stocks. In this case the fibers are not cut off and bending, tensile and compaction strength values are at their maximum.

In reality the tree trunk shape is not cylindrical, so the fibers are turned to be cut off which adversely affects wood products and stocks strength and stiffness.

Researchers and practitioners are continuously searching and developing schemes for log sawing aimed at the increase of volume output of sawn wood products at sawing, reduction of sawn wood products loss during further drying, increase of strength of structural sawn wood products used for building structures fabrication.

One of the up-to-date schemes for sawlog material cut-up is sawing parallel to generatrix. Experienced sawyers acknowledge obtaining of more wide sawn wood products when sawing parallel to generatrix (Fig.1). However, degradation in drying quality of such materials is not taken into account, as the core of the materials protrude to both sawn faces and at some areas along the length of wood plank the fibers are cut off under the angle twice greater than that of conventional cutting material.

It is obvious that when wood planks are located on the one side relative to the log axis, transverse and longitudinal distortion do not change their signs along the plank. This statement is proven by numerous investigations (Akishenkov,1980; Kolenchuk, 1950; Lyulenko, 1967; Petrukhin, 1970, Skripal'shchikova, 1975; Sokolov, Glukhikh, 1971; I. A. Strikha, 1974; Sukhova ,1958, etc.).

Our investigations (Glukhikh, Chernykh, 2013; Glukhikh, Zaripov, 2008) prove out that when drying, distortion of wood plank with end face located on various sides relative the log axis changes in terms of sign along the plank. The paper (Glukhikh, 2004) confirms that shrinkage stresses in the core of wood plank is more than 4 times higher than the same stresses in wood planks produced out of periphery zone. Considering this fact, one can consider double-sided log sawing parallel to generatrix more reasonable (Fig.2).

Issues related to strength and stiffness of such materials are not investigated, but they are of scientific and practical interest. Each new log sawing scheme as well as the procedure, effective output and other issues are to be explained in terms of strength and stiffness. This relates to sorting out of structural sawn wood products, fabrication of laminated wood structures, wooden structural panels, wooden and wooden-concrete bridges, etc.

We evaluated changes in tilting angle of wood fiber of sawn wood materials obtained from logs by conventional sawing and sawing parallel to log generatrix (Glukhikh, Khrabrova, Akopyan, 2013). This concept can be used to foresee the nature of changes in wood plank and their bending stiffness.

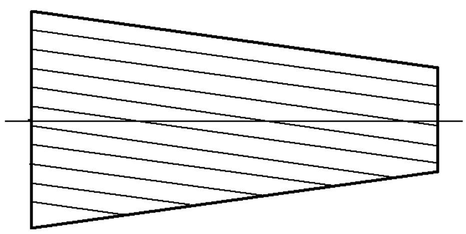

Fig.1 Single sided sawing of a log parallel to tapering

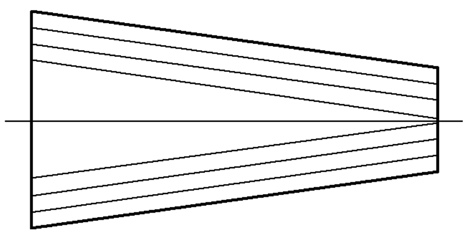

Fig.2 Double sided sawing of a log parallel to generatrix

2. Investigation of bending stresses in wood planks obtained by various log sawing schemes

Taking into account changes of wood fiber angles of tilting to three mutually perpendicular axis passing through sawn faces, end face, plank edge, the solution can be considered for a beam with changing modulus of elasticity.

The solution shall be found out for one of the simplest bar schemes (Fig.3).

The task is to be solved using the methods of theory for anisotropic body elasticity.

Hook's general law shall be used under the condition of small deformations:

$$\varepsilon_z = \frac{\partial \upsilon}{\partial z} = \frac{\sigma_z}{E_z} - \mu_y \frac{\sigma_y}{E_y}$$

$$\varepsilon_y = \frac{\partial \vartheta}{\partial y} = -\mu_z \frac{\sigma_z}{E_z} + \frac{\sigma_y}{E_y} \qquad (1)$$

$$Y_{zy} = \frac{\partial \upsilon}{\partial y} + \frac{\partial \vartheta}{\partial z} = \frac{1}{G}\tau_{zy}$$

where E_z, E_y, μ_z, μ_y, G are arbitrary functions of constant elasticity of wood as an anisotropy object,

E_z, E_y – elasticity modulus;

μ_z, μ_y – transverse deformation coefficients (Poisson's coefficient);

G – elasticity modulus of displacement.

Equilibrium equations are as follows:

$$\frac{\partial \sigma_z}{\partial z} + \frac{\partial \tau_{zy}}{\partial y} = 0 \qquad (2)$$

$$\frac{\partial \tau_{zy}}{\partial z} + \frac{\partial \sigma_y}{\partial y} = 0$$

Limiting condition values for this example:

a) $y = 0;\ \sigma_y = 0$
b) $y = 0;\ \tau_{zy} = 0$
c) $y = b;\ \sigma_y = 0$ (3)
d) $y = b;\ \tau_{zy} = 0$

Besides those four limiting conditions, the following equilibrium conditions are to be met

a) $\int_0^b \sigma_z dy = 0$

b) $\int_0^b \sigma_z y dy = -P\frac{z}{h}$ (4)

c) $\int_0^b \tau_{zy} dy = -\frac{P}{h}$

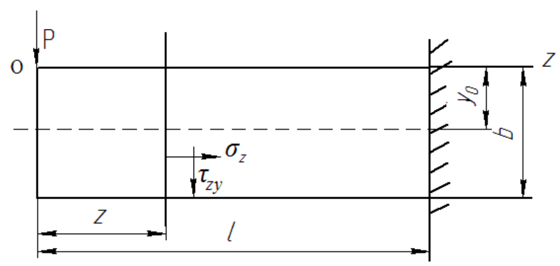

Fig.3. Scheme of an anisotropic beam

where h is width of transverse section (in our case this is the plank thickness).

When solving the similar problem, S.G. Lekhnitskiy (Lekhnitskiy, 1957) suggests that the nature of stresses distribution within such anisotropic beam is the same as that occurred at any other uniform beam, i.e.

$$\sigma_z = -\frac{Pz}{h} f'(y),\ \tau_{zy} = \frac{P}{h} f(y) \tag{5}$$

Suggesting validity of the second hypothesis on pure bending, on the basis of which the beam fibers do not apply pressure on each other at transverse bending thus subjecting to longitudinal tensile (compaction) deformation, i.e. $\sigma_y = 0$.

Applying interrelations (5), Hook's general law (1), equilibrium equations (2), conditions (3) and (4), S.G. Lekhnitskiy found the following function of f(y) (Lekhnitskiy, 1957):

$$f(y) = C\int E_z y dy + d\int E_z dy + e, \tag{6}$$

where C, d, e – integration constants determined on the basis of conditions (3) and (4).

As a result of our problem solving, formulas for stresses calculation in the beam are as follows:

$$\sigma_z = -\frac{6Pz}{hS} E_z(y)(2S_1 y - S_2) \tag{7}$$

$$\tau_{zy} = \frac{6P}{hS}\int_0^y E_z(y)(2S_1 y - S_2)dy \tag{8}$$

where $S_1 = \int_0^b E_z dy;\ S_2 = 2\int_0^b E_z y dy;$

$$S = 12[\int_0^b E_z dy \cdot \int_0^b E_z y^2 dy - (\int_0^b E_z y dy)^2] \tag{9}$$

The equation for the beam deflective axis is the same as for the isotropic beam with stiffness D equaling to

$$D = \frac{hS}{12S_1} \tag{10}$$

$$y = \frac{P}{6D}\left(z^3 - 3l^2 z + 2l^3\right) \tag{11}$$

Thus, considering this condition it is possible to calculate deflection of wood planks obtained by conventional sawing, y_1, and sawing parallel to generatrix, y_2, and use the relation, i.e.

$$\frac{y_1}{y_2} = \frac{P\left(z^3 - 3l^2 z + 2l^3\right)6D_2}{6D_1 \cdot P\left(z^3 - 3l^2 z + 2l^3\right)} = \frac{D_2}{D_1} \tag{12}$$

This relation will demonstrate what fold differs the stiffness of wood planks obtained as per two log cut-up schemes.

3. Conventional sawing method

In this case the function of elasticity modulus E_z may be represented by the following algebraic function:

$$E_z = E_z^0 + E_{z1}(2y - b)^2 + E_{z2}(2y - b)^4 \tag{13}$$

where E_{z1}, E_{z2} – coefficients of algebraic equation (13) determined by calculation;

E_z^0 – absolute term of the equation (13) representing the value of elasticity modulus in the beam layer for y = $\frac{b}{2}$.

The example of determination of the equation coefficients and the absolute term (13):

Following the calculation of change in elasticity modulus E_z throughout the section height based on the previously derived complicated formula:

$$\frac{1}{E_z} = \frac{n_3^4}{E_t} + \frac{l_3^4}{E_r} + \frac{m_3^4}{E_a} + \left(\frac{1}{G_{ar}} - \frac{2\mu_{ar}}{E_a}\right)\cdot m_3^2 l_3^2 +$$
$$+\left(\frac{1}{G_{rt}} - \frac{2\mu_{rt}}{E_t}\right)\cdot l_3^2 n_3^2 + \left(\frac{1}{G_{at}} - \frac{2\mu_{at}}{E_a}\right)\cdot m_3^2 n_3^2$$

(Glukhikh, Khrabrova, Akopyan, 2013), E_z^0, $E_{z\kappa}$, E_{zc} are known.

Using these values, it is possible to approximate elasticity modulus E_z with algebraic function of the fourth order. For this purpose it is sufficient to find coefficients for variable $E_{z\kappa}$, E_{zc} using the following conditions:

- the edge: for $y = 0$, elasticity modulus $E_{z\kappa} = E^P_{z\kappa}$, i.e. is equal to the calculated elasticity modulus $E^P_{z\kappa}$;
- passing point: for $y = \frac{b}{4}$, elasticity modulus E_{zc} is equal to the calculated elasticity modulus E^P_{zc};
- the beam middle: for $y = \frac{b}{2}$, elasticity modulus E_z^0 is equal to the calculated elasticity modulus E^0_{zp}.

4. Sawing parallel to generatrix

The function of elasticity modulus E_z may be represented by the following algebraic function:

$$E_z = E_a - E_{z1}(2y - b)^2 - E_{z2}(2y - b)^4 \tag{14}$$

where E_a is absolute term of the equation (14) representing the value of elasticity modulus in the beam layer for $y = \frac{b}{2}$;

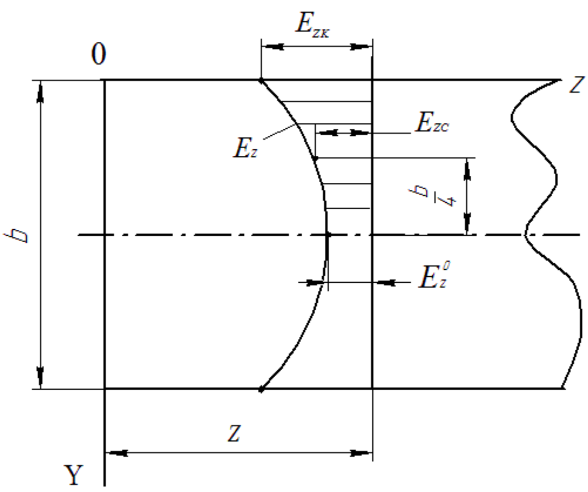

Fig.4 Change in elasticity modulus Ez across the wood plank width at conventional sawing

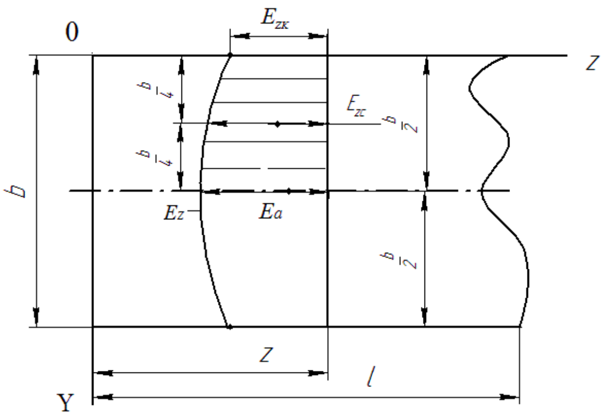

Fig.5 Change in elasticity modulus Ez across the wood plank width at sawing parallel to generatrix

E_{z1}, E_{z2} – coefficients of algebraic equation (14) determined by calculation.

Thereby the following conditions are to be met: elasticity modulus in the neutral layer is E_a, and at the edges – E_{zk}. It is considered that for $y = \frac{b}{4}$, elasticity modulus is also known (the same as for the previous problem).

- for $y = 0$ $E_z = E_{zk}$, i.e. $E_{zk} = E_a - E_{z1}b^2 - E_{z2}b^4$

- for $y = \frac{b}{2}$ $E_z = E_a$

- for $y = \frac{b}{4}$; $E_z = E_{zc}$,

$$E_{zc} = E_a - E_{z1}\left(2\frac{b}{4} - b\right)^2 - E_{z2}\left(2\frac{b}{4} - b\right)^4 =$$

$$= E_a - E_{z1}\frac{b^2}{4} - E_{z2}\frac{b^4}{16}$$

5. Results. Distribution of the elasticity modulus across the wood plank width

The diagrams above (Fig. 6) show the changes in elasticity modulus across the wood plank width when different sawing methods are used. For the first time it is determined that in case of sawing in parallel to generatrix maximum E_z value is reached in the wood plank center and reduced when getting closer to the edges. In case of conventional sawing, reverse behavior may be seen.

When, in case of conventional sawing, a wood plank moves away from the log longitudinal axis, the elasticity modulus decreases. Vice versa, in case of sawing parallel to generatrix, the elasticity modulus increases.

When a wood plank is sawn out at a distance of 80 mm from the log longitudinal axis, the average elasticity modulus, in case of conventional sawing,

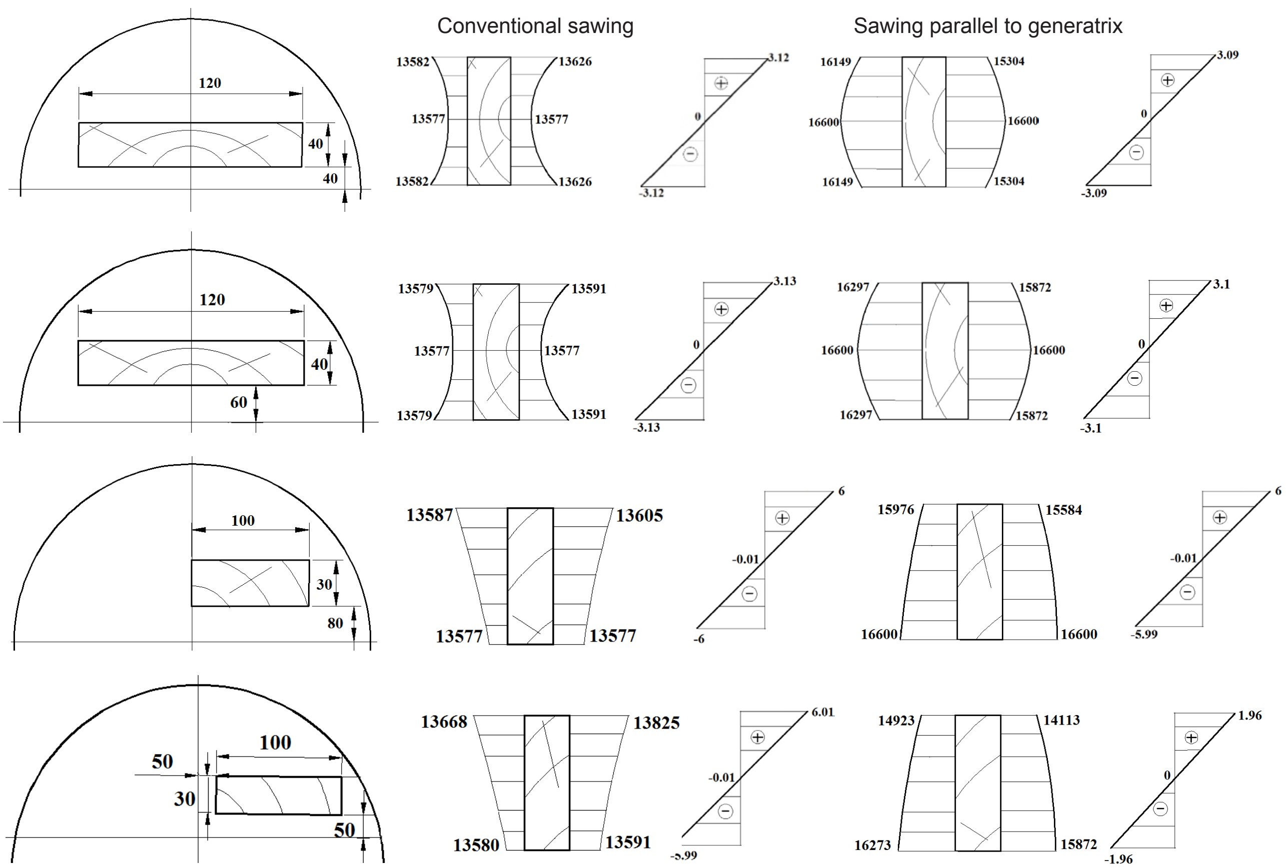

Fig. 6 Comparison of the elasticity modulus and stress sketches across the wood plank width. Conventional sawing and sawing parallel to generatrix.

Table 1
Wood plank stiffness, various coordinates ($\gamma = 3^\circ$)

Coordinate of wood plank sawn face width, B mm		Distance from the log axis to the sawn face under investigation, b, mm	Stiffness		Wood bending, D_2/D_1
B_1	B_2		Conventional sawing, D_1	Sawing parallel to generatrix, D_2	
-60	60	40	7,833,468	9,046,821	1.15
-60	60	60	7,823,873	9,291,571	1.19
-60	60	80	7,821,558	9,399,283	1.2
-60	60	100	7,820,875	9,454,118	1.21
50	150	50	3,423,186	3,727,653	1.08
50	150	80	3,403,472	3,898,885	1.14

equals to E_{avg} = 13580; in case of sawing in parallel to tapering. The average elasticity modulus equals to E_{avg} = 16487. It turns out that the elasticity modulus is 19% higher when sawing parallel to tapering. For calculation purposes, the angle of generatrix tilting γ to the log axis was taken as equal to 3°. If the angle equals 5°, the elasticity modulus is 46% higher when sawing parallel to generatrix.

Table 1 shows variation in stiffness of sawn wood products obtained from different sections of a log. In case of sawing parallel to generatrix, stiffness of a wood plank increases with increase in the distance from the log axis to the sawn face under investigation. In case of conventional sawing, stiffness, in reverse, decreases with increase in the distance. In terms of percentage ratio, stiffness of sawn wood products in case of sawing parallel to tapering ($\gamma = 3^\circ$) is 14–19% higher than that of sawn wood products obtained by conventional sawing. The obtained sawn wood products exhibit less deflection and higher strength properties.

6. Summary

The elasticity modulus describes material properties in terms of higher or lower deformation under load. Stiffness of materials limited by elasticity may be compared using the value of elasticity modulus.

Mathematical models describing changes in elasticity modulus of a wood plank width sawn parallel to generatrix enable to foresee the wood plank bending strength and stiffness.

The suggested procedure allows for the evaluation of bending stresses of wood planks obtained by various log cut-up schemes.

Sawn wood products obtained by sawing parallel to generatrix exhibit higher strength and stiffness properties which can be used in fabrication of construction part and structures with improved strength properties.

The investigation results are not only of scientific interest, but also of practical one in terms of construction and fabrication of laminated wood structures, wood structural panels, etc.

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DISMANTLING AND RECONSTITUTION OF PRASAT SUOR PRAT, ANGKOR THOM, CAMBODIA

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Abstract

This paper presents a geotechnical aspects of dismantling process of one of masonry towers named Prasat Suor Prat that had been constructed in the end of 12th century during the Angkor Period in Cambodia. A series of 12 towers had been constructed from south to north along the east side of Royal Plaza in Angkor Thom.

One of the towers, named as N1 Tower, was found badly displaced with inclination of about 5 degrees to the north-west and horizontal spreading at the foundation level. The Tower was dismantled before restoration work by JSA (Japanese Government Team for Safeguarding Angkor) to study possible mechanism that had caused inclination of the Tower and horizontal spreading of stair stones. Dismantling upper structures as well as foundation mound were performed by archaeological trench. The trench had revealed the mechanism of deformation of the structure as well as foundation. Before the dismantling, was the inclination caused by tilting of the foundation mound caused by general sliding failure of the foundation towards adjacent pond. However, it was revealed that the mound was not tilted but kept horizontal under yielded state causing only horizontal spreading. It was found the inclination was caused by slip down of the sidestep cut stones that had covered the side slopes of the mound

Keywords:

Prasat Suor Prat, Angkor, dismantling of heritage structure, inclination, foundation spreading

1. Introduction

Angkor locates at a wide plain that expands between the Kulen Mountain in the north and the Ton le Sap Lake in the south as shown in Fig.1.The width of the plain is about 50km with gentle inclination of 1/1000 from the north to south.

Angkor monuments distribute within the vast area of the plain. The famous monuments of Angkor Wat as well as Angkor Thom locate in the middle of the plain. Water is provided in the plain not only from rain fall but also from Kulen Mountain by Siem Reap River. In the middle of the plain, great man made reservoirs called as “Barai” were constructed to supply water in dry season. At present the West Barai is still in use with a dimension of 8km by 2.5km as shown in Fig.1.

JSA (Japanese Government Team for Safeguarding Angkor) had performed the first deep geological boring in the area that revealed underground condition in 1994.

Fig.3 shows a north-south section of the boring logs at the Royal Plaza, Bayon in Angkor Thom, Angkor Wat, and Siem Reap city. The upper 40m of the ground surface is Quaternary deposits of silty fine sand with several clayey rich layers. In general SPT (standard penetration Test) N-value increases with depth except at the top surface. N-value at the top surface is 20 and decreases to 10 at the depth of 5m. The boring at this point was carried out in March 1995 of dry season. The cone penetration test that was performed in rainy season at the same site revealed that the cone resistance decreased to

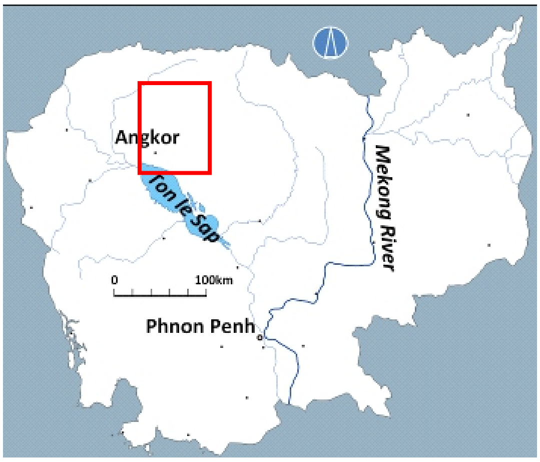


Fig.1. Cambodia and Angkor

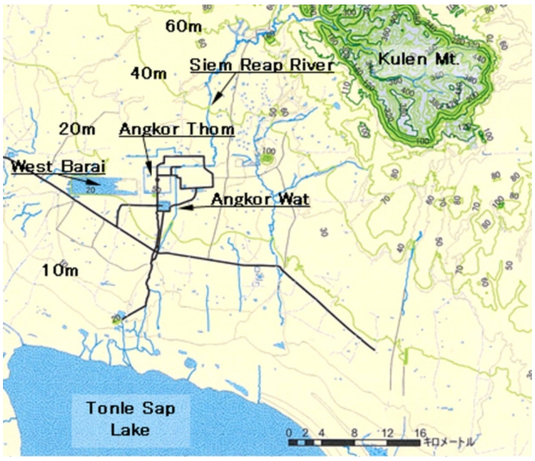


Fig.2. Angkor area (see the quadrangle area in Fig.1)

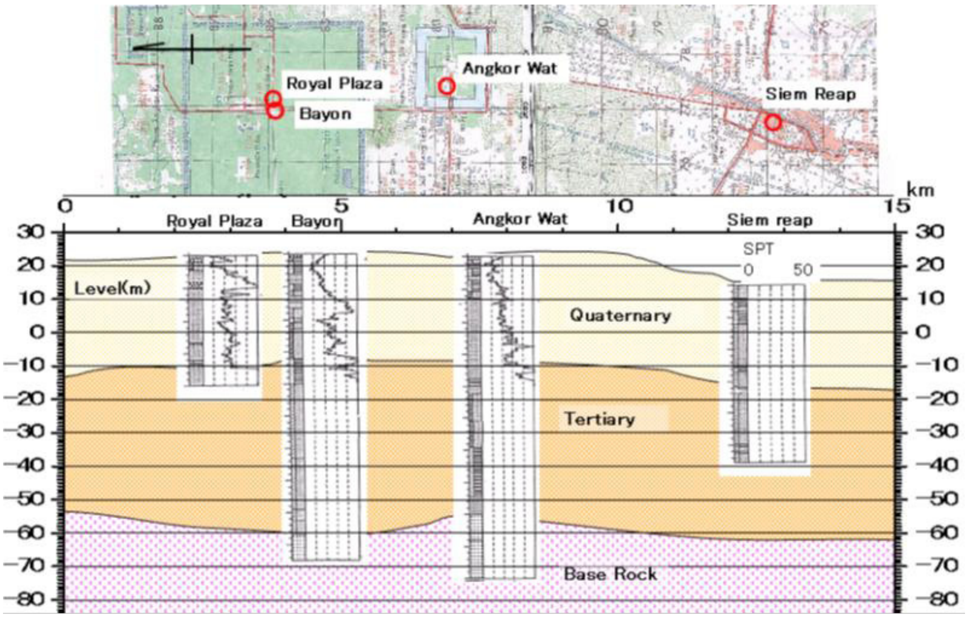


Fig.3. Geotechnical Section along N-S section from Bayon, Angkor Wat, to SiemReap

zero at the surface and increased with depth down to about 5m. The top surface of 5m is affected by the seasonal change of water level.

Water condition in the ground gives crucial effects upon mechanical characteristics of surface ground in Angkor and forms one of the characteristic elements of cultural heritage.

2. Prasat Suor Prat

Prasat Suor Prat (Tower of Rope Dancer) is a group of 12 independent towers along the west side in the Royal Plaza as shown in Photo-1 and in Fig.4 of plan view. These towers are divided into two groups of North and South. The inclination of side walls for all the towers at Prasat Suor Prat were measured and listed in Table1 and shown in Fig.5. Inclinations of the towers are less than 2-4 degrees exept N1 and S1 and with direction of NW to NE. N1 and S1 show the largest inclination among each group. Each Tower stands near the north and the south ponds respectively (Akazawa, 2005).

The largest inclination was found for N1Tower. The following section was devoted to the state of deformation before conservation of 2000-2005.

3. Deformation characteristics of N1 Tower

One of the 12 towers named as N1 that had been inclined about 5degrees northwards was selected by JSA for safeguarding work as shown in Photo 2. The tower consists from a main tower with a front room called antechamber. The main tower is three story masonry structure with laterite brocks with 10m in width at foundation and about 20m in height. The inside of the tower is hollow structure with four open windows at every wall. Fig.6 shows vertical section and plan view of the N1 tower nearby the north pond.

The differential settlement of the foundation was measured about 40cm at the north and west corner

Photo 1. Prasat Suor

Photo 2. N1 Tower, Prasat Suor Prat

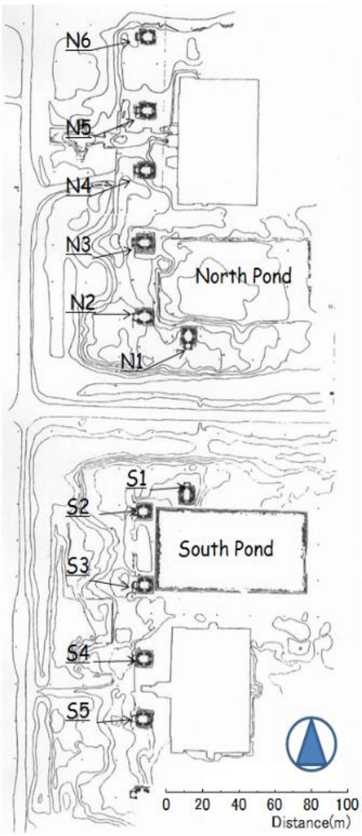


Fig.4. 12 towers of Prasat Suor Prat

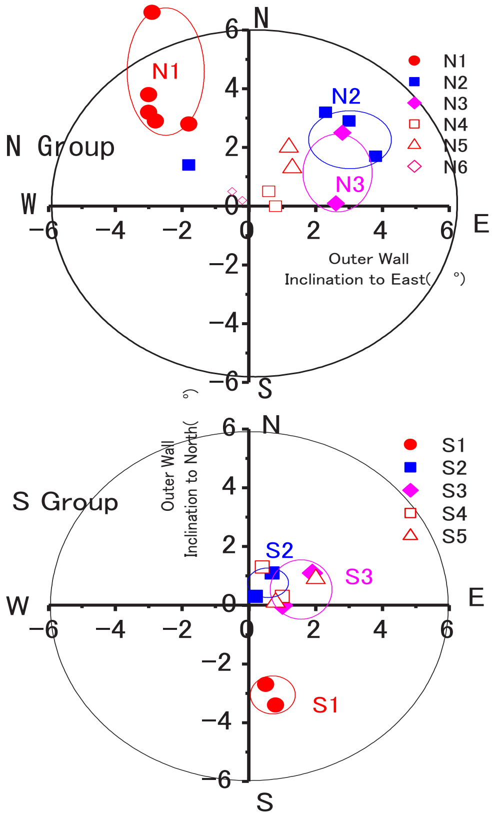


Fig.5. Inclination of Towers

that was lower than that of the south and east corner (Akazawa, 2005 and Fukuda, 2005).

Fig.7 also shows contour lines of equi-settlement relative to the south-east corner of the foundation.

The open window of the west wall is deformed as shown in Photo 3.

The widths of the bottom of the window are wider than those of the top for all four walls and the measured results are shown in Table-1. The laterite blocks of the side face of the foundation were found widened about 6-8%. One of the characteristics of the deformation of the foundation of N1 Tower is horizontal spreading. Fig.8 shows two major trends of deformation in the foundation system of inclination towards the North pond and horizontal spreading.

Table 1
Inclination of side walls at Prasat Suor Prat

Group	No.1	No.2	No.3	No.4	No.5	No.6
North	3-7 NNW	3-4 NE	2-4 NE	0-1 NE	0-1 NW	0-1 NW
South	3-4 SSE	0-2 NE	1-3 NE	1-2 NE	2-3 NE	

Table 2
Width of open window at top and bottom

Side	Width of opening		Difference	Expanded rate
	top	bottom		
	Lt	Lb	ΔL=Lb-Lt	ΔL/Lt
	cm	cm	cm	(%)
east	170.3	182.0	12.0	7.1
west	171.3	185.0	14.0	8.2
north	168.0	178.0	10.0	6.0
south	124.0	131.7	7.7	6.2

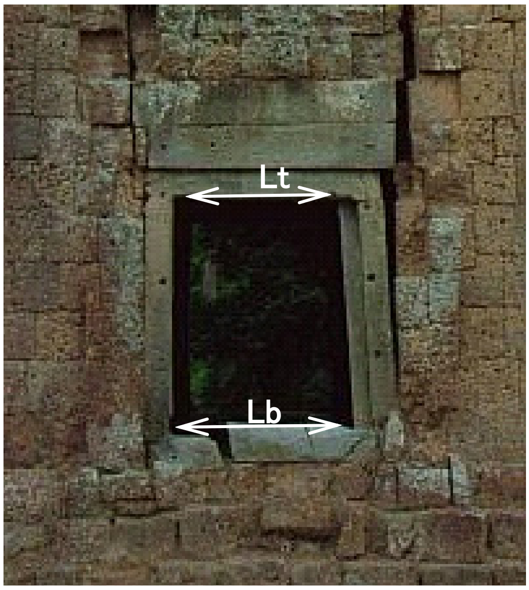


Photo 3. Deformed frame of open window at west side

The horizontal line of side face stones of foundation mound was inclined to northwards. The levels of upper surface of the top (L1), the second (L2), and the third (L3) lines in the upper part of Fig.7 were measured at several points and plotted in the lower part of Fig.7. The inclinations of these side stones of the foundation mound were found about 0.4m per 10m of horizontal distance. The inclination of the foundation as well as upper structure suggested that the inclination of the Tower might have been caused by such ground deformation as sliding failure.

4. Dismantling of Heritage Structure

It is generally accepted that dismantling should be avoided if other method is available for restoration work. The ICOMOS Charter – Principles for the Analysis, Conservation, and Structural Restoration states 3.17 Dismantling and reassembly should be undertaken as an optional measure required by the very nature of the material and structure when conservation by other means impossible or harmful.

Restoration work of N3 Tower in 1960 and Dismantling N1 Tower

In 1960,' N3 Tower of Prasat Suor Prat had been found badly inclined. French team had performed restoration work for the N3 Tower. Stones of the upper structure had been dismantled without treating the foundation system. After the restoration work, the N3 Tower had been found inclined again within a few years. When JSA began the restoration work of N1 Tower of Prasat Suor Prat, the question of insufficient result of restoration work of N3 was discussed.

As shown in the previous section, the foundation of the N1 Tower had been inclined to NNW direction

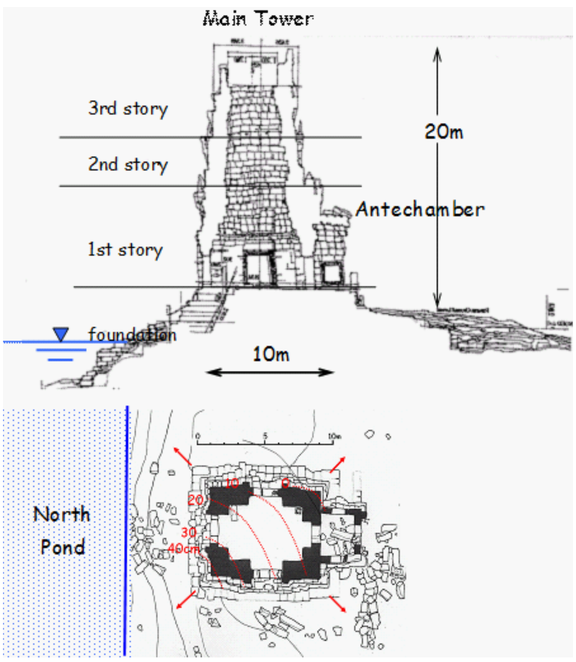


Fig.6 Vertical section and plan for N1 Tower

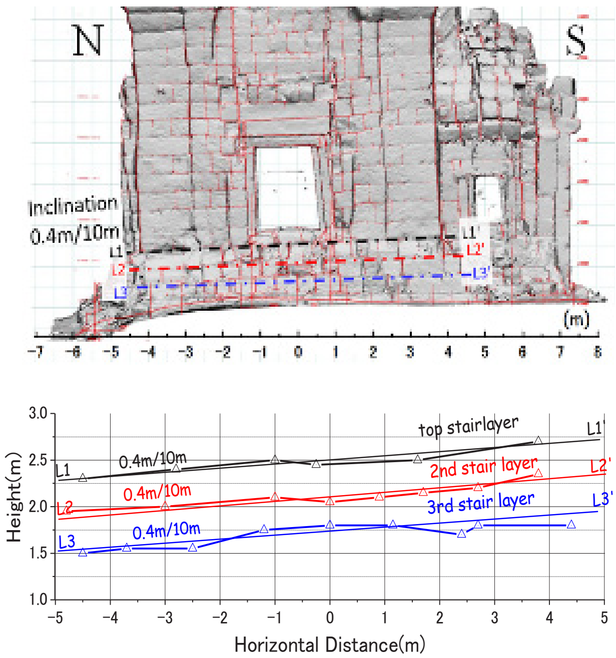


Fig.7 Inclination of step stones at the west side, N1 Tower

and spread horizontally in all directions. It was clear that the deformation of upper structure had been caused by insufficient performance of the foundation system. It was decided to study foundation system by dismantling and solve mechanical instability that had caused inclination and spreading of the upper structures.

5. Dismantling of N1 Tower

Dismantling of N1 Tower of Prasat Suor Prat was carried out before reconstruction of the Tower.

After the laterite blocks of the upper structure were dismantled, the foundation structures were further dismantled. The soil mound was excavated down to reach natural ground layer (Nagatomo et al., 2005).

Photo 4 shows the trenched site of N1 Tower of Prasat Suor Prat, where vertical section was exposed.

Fig.8 shows a trenched section of N1 Tower along N-S direction. The upper structures consist from Main Tower and Antechamber. The bottom level of the upper structures was the same level of top level of side step stones. As already stated, the top level of step stones had inclined to northwards with settlement of 0.4m for horizontal length of 10m as shown in Fig.7 and also indicated in Fig.8 as well.

When the trench proceeded to the level of +0.00m, the manmade compacted sand layers were exposed. We had expected the same inclination of the compacted soil layers in the foundation mound. However, amazingly, we found the manmade layers of soil mound formed almost horizontal layers as shown in Fig.8. It was mysterious for us to understand until the excavation reached to the much lower level.

6. Deformation mode of foundation failure

When the trench was further deepened, the section in the north side of the foundation mound showed a small local bearing failure at horizontal point Y11 in Fig.8, which had resulted in sliding of retaining slope of north side face. The bottom element of side stones had settled by about

Photo 4. Archaeological trench at N1

Photo 5. Trenched EW section (note the horizontal layers)

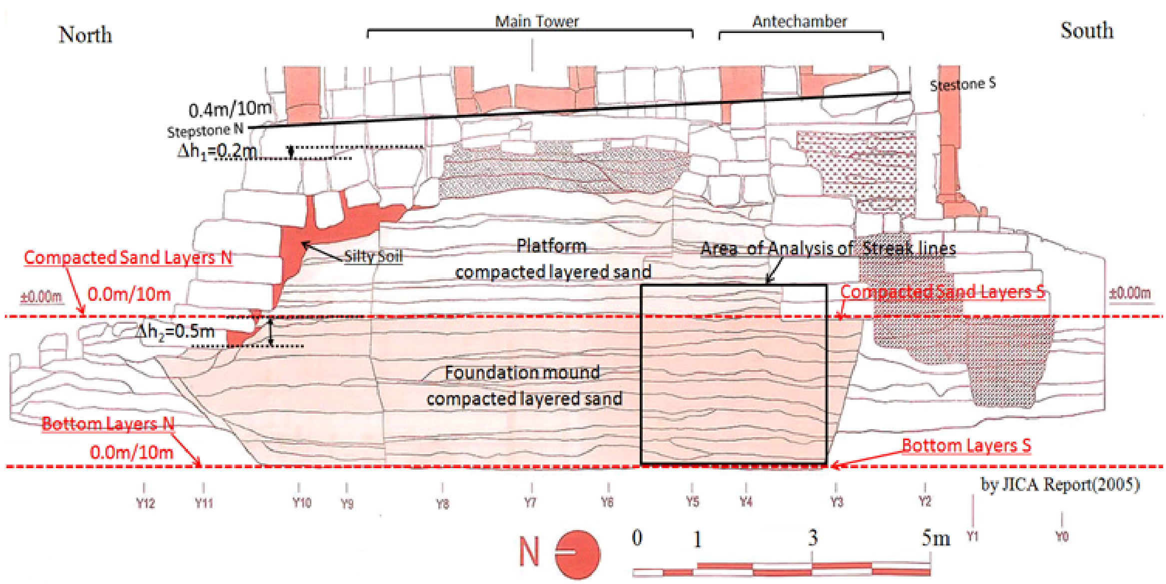


Fig.8. Trenched section along N-S direction

Photo 6. Side stone at north face

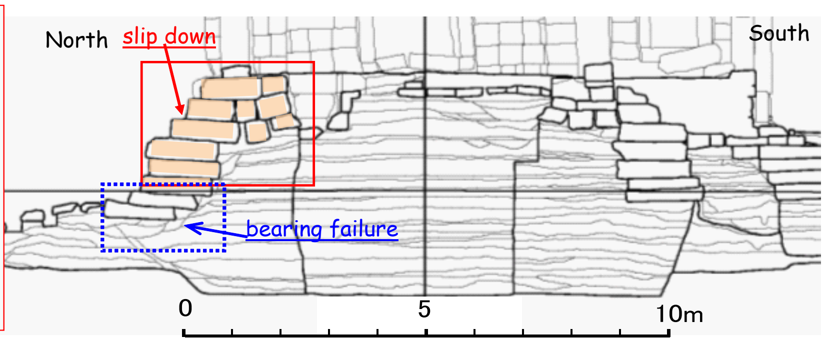


Fig.9. Mode of failure of step stones at the north side

ΔH_2=0.5m from the original level. The bearing failure took place for the bottom of the step stones. It should be noticed that the bottom of the step stones settled about 0.5m, however, the highest block of the step stones was found settled only ΔH_1=0.2m as shown in Fig.8.

It is estimated that the bearing failure at the base had been the initial step of the failure that caused to drag the upper step stones to slip down as shown in Fig.9.

Photo 6 shows the deformation of step stones at north side. The outer most stone settled and displaced outwards. Accordingly, the side stones behind the front stones were also displaced outwards but the horizontal displacement was less than the front stones.

7. Systematic streak line on the trenched surface

As shown in Photo 7 and in Fig.10, many streak lines were found on the face of trenched section of platform and foundation mound. Based upon the photos of the sections, the streaks were analyzed in the part of squired section of "Area of Analysis of Streak lines" shown in Fig.8.

The area shown in Fig.8 was further divided into two sections of upper and lower part as shown in Fig.11 and the distribution of the angle of streak lines were analyzed and the distribution patterns of the angles of streak line from the vertical line are plotted in Fig.12. The strong tendency of the angle of 30degrees from the vertical direction was obtained, which suggests the streak lines correspond to shear surface under failure.

The angle between the direction of failure surface and principal stress direction under the Mohr-Coulomb in

$$\theta = 45 \pm \phi/2 \qquad \text{(equation-1)}$$

where

θ: angle between shear plane and principal major stress axis

ϕ : internal angle of friction of material.

Since the internal frictional angle of sand obtained by laboratory test was 30 degrees, θ become 30 degrees when the vertical overburden stress is the major principal stress.The appearance of these streak lines suggests that the foundation mound might have experienced plastic yielding under vertical load.

Photo 7. Streak lines on the trenched surface

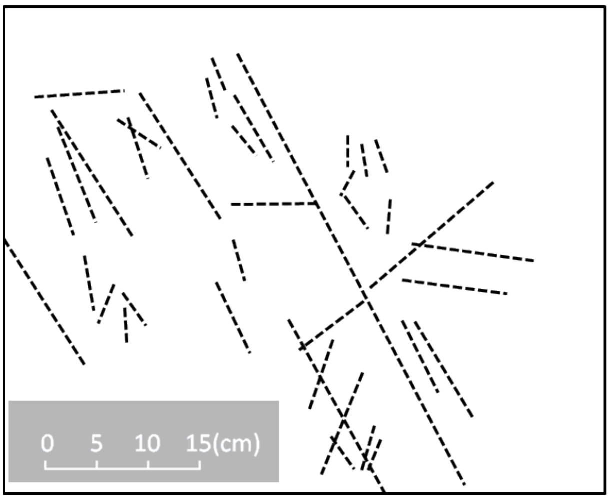
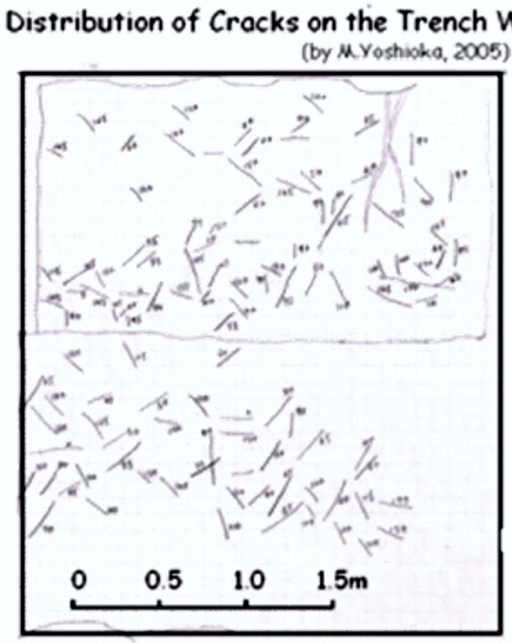


Fig. 10. Sketch of streak line on trench surface of Photo 7



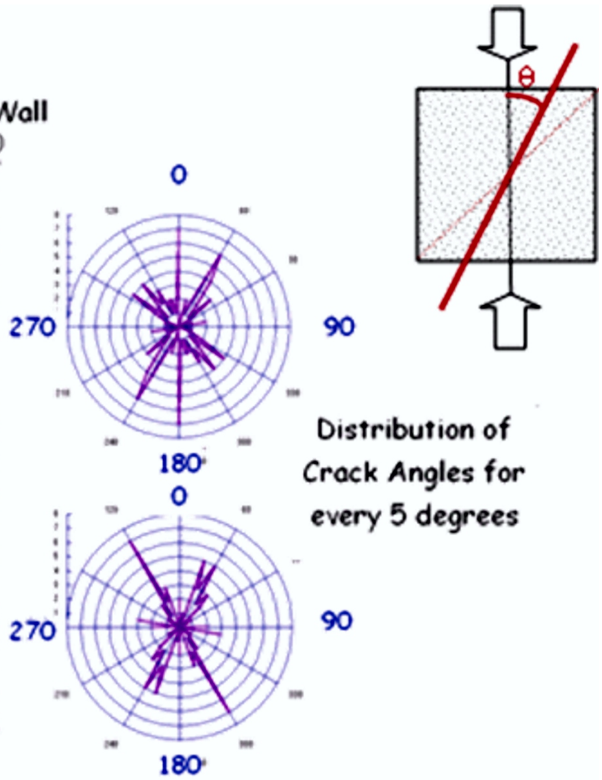


Fig.11. Sketch of streak lines in the section shown in Fig.8

Fig.12. Distribution of angles of streak lines

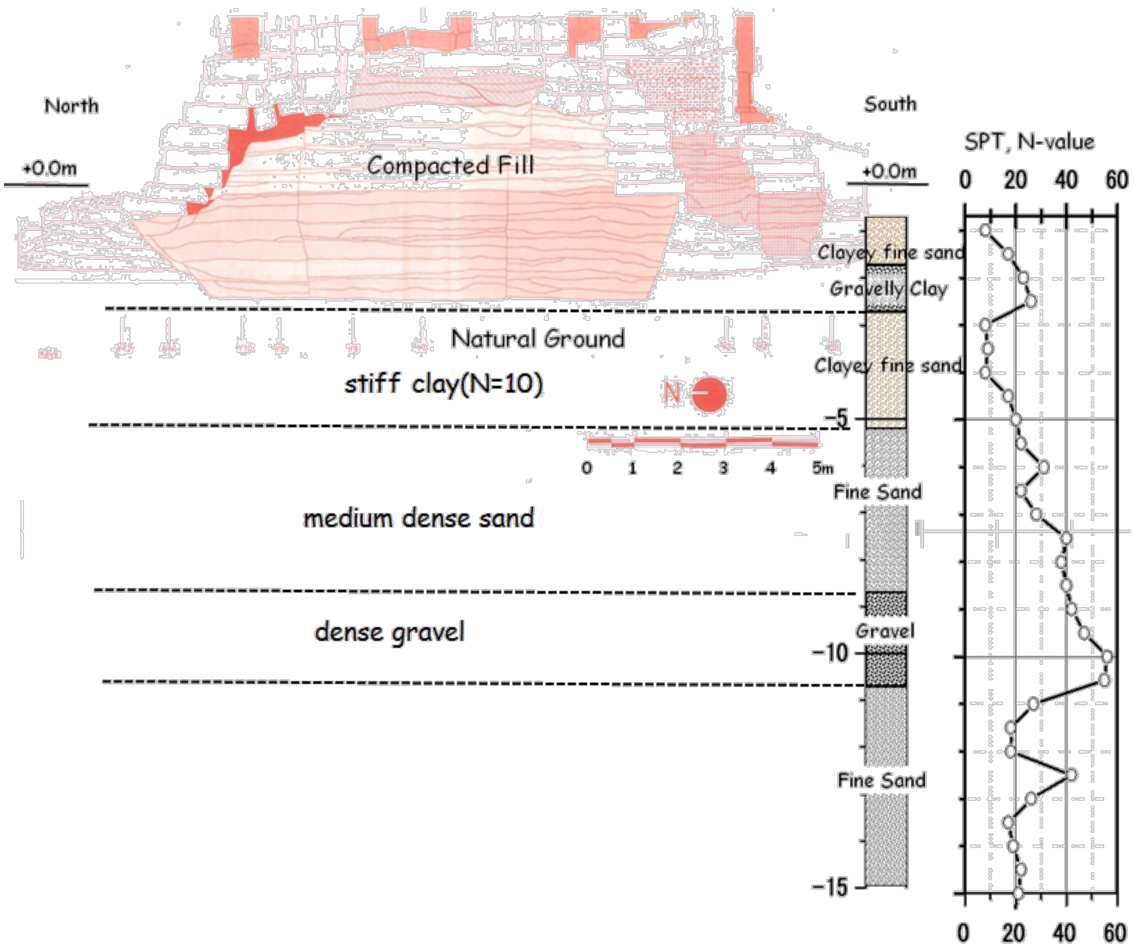


Fig.13. Geotechnical Condition near N1 Tower, Prasat Suor Prat

8. FEM Simulation of yielding of foundation soil of N1 Tower, Prasat Suor Prat

The response of the compacted soil mound by loading of the upper structure is simulated by a FEM and the result of the distribution of yielding points shall be shown.

Geotechnical condition at the site is shown in Fig.13. Beneath the compacted sand mound, stiff clay (N=10) of natural formation follows some 3m. There are medium to dense sand and gravel layers below the stiff clay.

Material Parameters are assigned based upon triaxial test results and field plate loading tests for compacted sand mounds and estimated parameters for laterite and natural soils as yielding condition of Mohr-Coulomb criteria.

The estimated total weight of the upper masonry stone structure was 8.97MN which resulted in the vertical load of 421kPa at the top of the foundation. The maximum vertical settlement of 18.4cm was obtained at the surface level of the mound. Figs.15 and 16 shows the distribution of plastic zones and shear strain that were caused by the load from the upper masonry structure of 400kPa. Plastic zones were shown in Photo-5, many streak lines were found on the trenched section of platform and foundation mound. Based upon the photos of the sections, the streaks were analyzed in the part of squired section of “Area of Analysis of Streak lines” shown in Fig.8.

The area shown in Fig.8 was further divided into two sections of upper and lower part as shown in Fig.11 and the distribution of the angle of streak lines were analyzed and the distribution patterns of the angles of streak line from the vertical line are plotted in Fig.12. The strong tendency of the angle of 30degrees from the vertical direction was obtained, which suggests the streak lines correspond to shear surface under failure.

The soil parameters are based upon some typical number estimated by the authors. However, the FEM results suggest the essential features of the characteristics of deformation of the Tower.

The seasonal change of level of underground water might have affected the strength of the soil mound and repeated nature of weakening soil mound has caused yielding of the mound and the

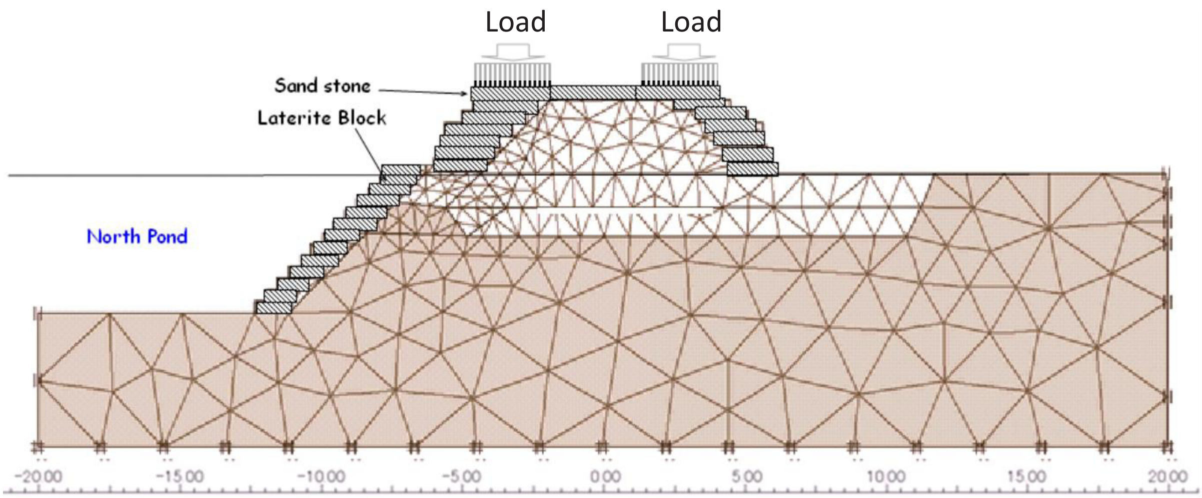


Fig.14. FEM mesh by PLAXIS 2D

Table 3
Soil parameters for FEM simulation

Material	Young's Modulus	Poisson's ratio	cohesion	Internal friction	Unit weight
	kPa		kPa		kN/m3
Compacted Soil	25,000	0.3	25	30	18
Natural Soil	10,000	0.3	20	30	18
Laterite/Sand stone	1,000,000	0.2			30

Table 4
Load from upper structure

Load from upper masonry	
Total weight	8.97MN(879tf)
Load stress upon the foundation stone	421kPa(43t/m2)
The maxium settlement obtained at top of mound	18.4cm

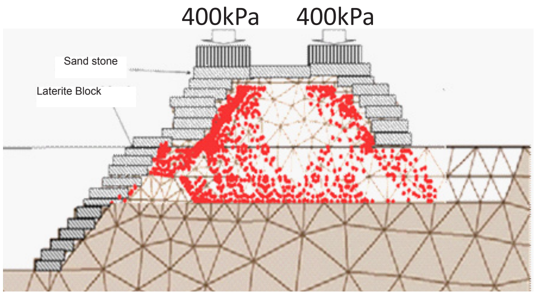


Fig.15. Plastic zone for loading of 400kP

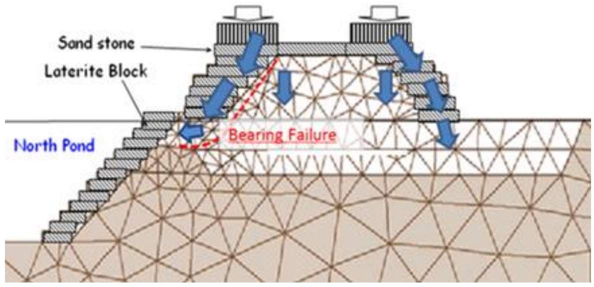


Fig.17. Stress flow in the foundation

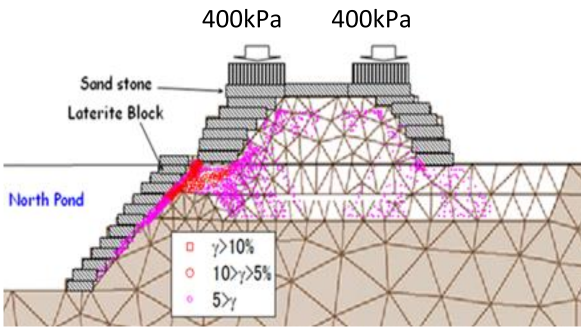


Fig.16. Shear strain for loading of 400kPa

local base failure. The reconstitution process was reported by Akazawa (2005,b).

9. Conclusions

The conclusions of this paper are listed as follows,

1. N1 Tower of Prasat Suor Prat, Angkor Thom, was dismantled to study deformation mechanism that had caused lateral spreading of the foundation and inclination of the Tower.
2. The N1 Tower shows inclination of about 5degrees of the side walls of the tower and 0.4m settlement along the base step line of 10m. The horizontal spreading 6-8% at the base level. Before trenching work, the N1 Tower had inclined because of the general sliding of the foundation mound towards a nearby pond.
3. Archaeological trench was performed during dismantling and showed streak lines on the trenched surface that suggest yielding state of the soil mound under the heavy load of the upper structure.
4. The trench also showed horizontal layer of the soil mound that had nothing to do with the inclination of the upper N1 Tower, except a local failure at the base of side. The load of upper Masonry structure was supported by foundation stones that were further transmitted to side stones. Rather high intensity of loading stress concentrated beneath the bottom level of sidestone had failed and triggered slip down of the side step stones along the sideslope of the mound.
5. The dismantling was successful to disclose the geotechnical mechanism of the deformation of the foundation structure of N1 Tower, Prasat Suor Prat.

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LESSONS OF HISTORY AND SEARCH FOR “NEW GLOBAL STYLE” IN ARCHITECTURE

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Abstract

The diversity of theoretical concepts in the modern architecture has led to the situation that in early 21st century one of the trends of the modern architectural theory is the search for the ways of creating *a New Global Style*. A similar situation was observed in the middle of the 19th century, when classicism was replaced by numerous *neo-styles*; and the main problem of the architectural thought was a search for the style of the epoch. The paper deals with the analysis of “lessons of history”.

Keywords

Architectural theory, style of the epoch, global style

1. Introduction

The newest theoretical concepts are reflecting the attempts to support individual trends and directions of modern architecture with some philosophical justification; or represent the ideas of architects-innovators. The diversity of these concepts has led to the fact that at the turn of the 20th-21st centuries, we see increasing attempts to formulate the concept of the “New Global Style”. A similar situation was observed in the middle of the 19th century, when the “search for the style of the epoch” was the main focus of the architectural theory. These analogies confirm the vitality of the aphorism of the German researchers M. Brix and M. Steinhäuser that “the history only is contemporary” and is able to give answers to many pressing questions (Brix M., 1978). The paper deals with the analysis of the history of the “search for a new style”.

2.1. Discussion about “new style of the epoch” in 1830–1850s

In late 1840s, Classicism turned into one of *neo-styles*. It was not only squeezed out by mediaevalist and “national” directions, but by that period, the collapse of its artistic system was over. In 1842, the English architect T. L. Donaldson declared, by summing up “the style development of the romantic architecture,” the then established “equivalence” of styles, and stressed that “there is no longer any single dominant style; we are now straying within a maze of experiments” (Kazhar, 2000, p. 217).

By the middle of the 19th century, a lot of currents evolved in architecture, which may be roughly placed between two poles. At one of them, architects were searching for the *architecture of constructive truth* and of *truthful use of material* (Fig.1). The opposite pole hosted the theory of neo-styles, dominated by a search for some symbolic meaning of individual forms. By applying *styles of the past*, architects also restored the ideas related thereto (Fig.2).

“They say that everything has been invented; and the times of openings are over. The only thing remaining for the art is to choose and imitate,” J. Savage wrote in “Overview of Styles in Architecture” in 1836. “Who said that architects of our time are deprived of new opportunities? Who said that they won’t be able to create an authentic architectural style out of a thousand different types of ... Egyptian, classical, Gothic, or any other motives...” A. Bartholomew objected in his article “Specification of Practical Architecture” (1846) (Kazhar, 2000, p. 217). The decade that stretches between these two statements marked a transition to a new problem of the architectural theory – the search for the ways of creating a *new style of the epoch*. After studying historical styles, theorists, set the task to define some general criterion of architecture, which could create some *integrated style*.

The discussion about the new style was launched by the work of the German theorist H. Hübsch “In What Style Should we build?” (1828) (Fig.3.). The very emergence of the *discussion about a new style*

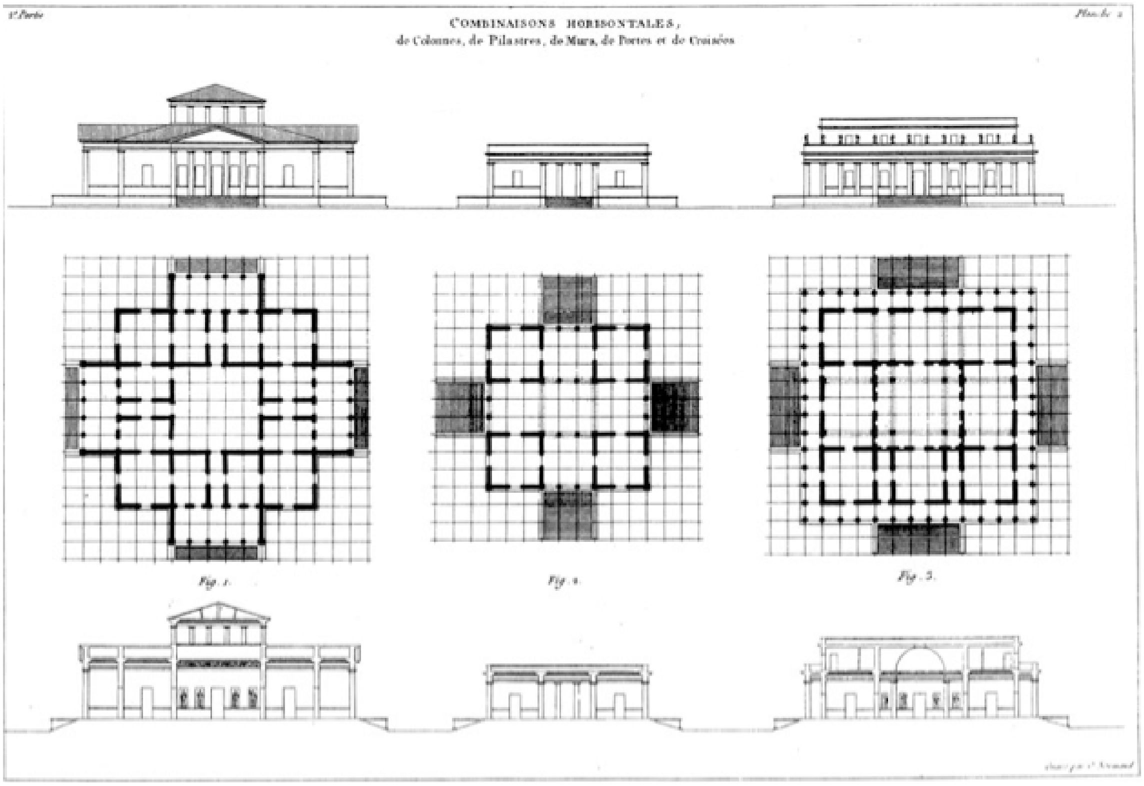

Fig.1. Grid system as the basis of "sustainable architecture", J. N. L. Durand, Plate *from et parallèle des édifices de tout genre, anciens et modernes : remarquables par leur beauté, par leur grandeur, ou par leur singularité, et dessinés sur une même échelle* by J.N.L. Durand. pub. D. Avanzo; (1830?)

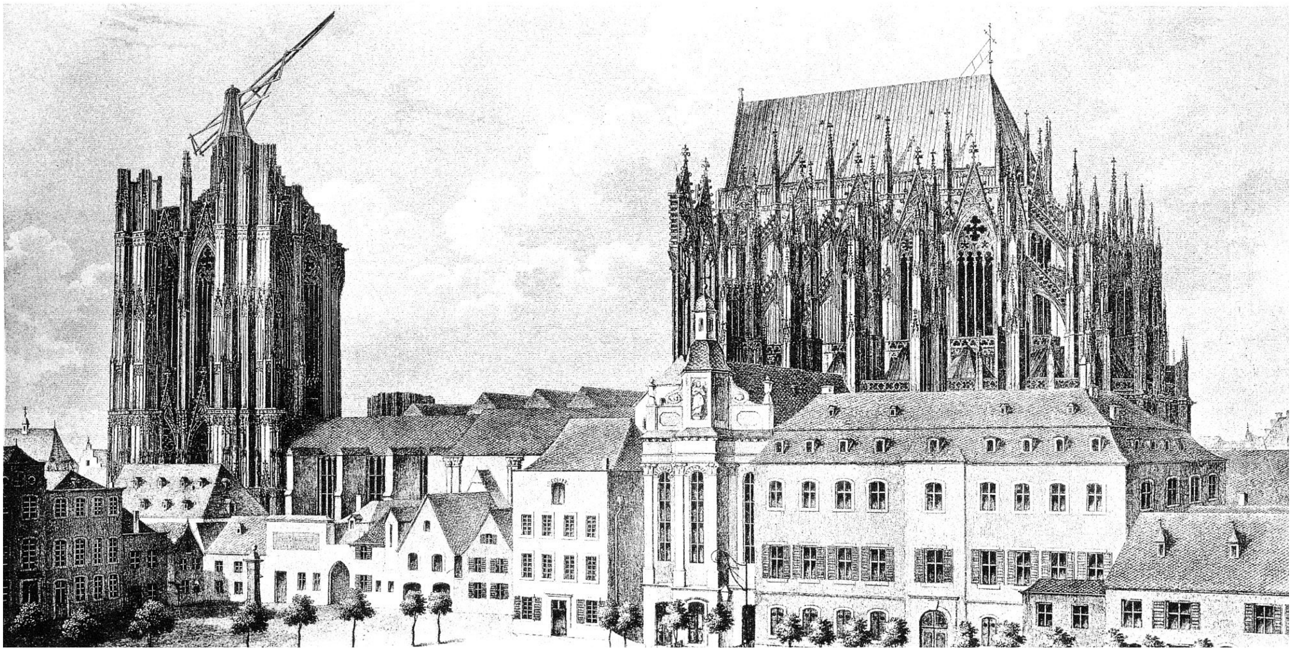

Fig.2. Completion of the Cologne Cathedral - an expression of the idea of building of the "Fatherland altar" (1248, 1842–1880)

was an indicator of the absence of a clear definition thereof. Most European researchers realized the *multiplicity* of the then concept of style. According to French theorists, the range of definitions spread between the poles of the *artist's style* and the *style of the epoch*. E. E. Viollet-le-Duc, for example, distinguished among *the style of the epoch* and *the style of arts*. He also highlighted the notions: *the relative style* (dependent on the nature of the subject) and *the absolute style* (defined by the dominant aesthetical concept) (P. Krakowski, 1978, p. 41–42). *The dual nature* of the style of the 19th century (its scientific and artistic aspects), or, rather, the constructive basis and the method of aesthetic

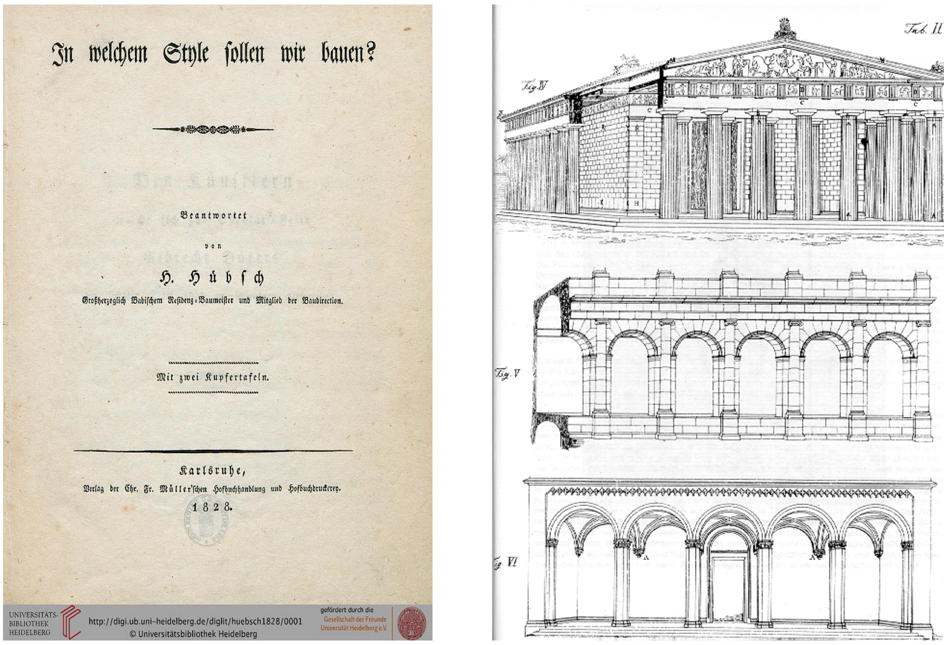

Fig.3. H. Hübsch “In what style should we build?”

impact, was emphasized by S. Dali (P. Krakowski, 1978, p. 37).

The German theorist G. Semper defined the style as “the highest level of artistic embodiment of the main idea of a piece of art, with account of all the internal and external factors that affect it.” H. Hübsch wrote that “the style should be understood as something universal that fits to all the buildings of a certain nation” (Kazhar, 2000, p. 221). The *new style of the epoch* had to have the *dignity and greatness*, *to be economical*, and have *the nature of a monument of arts*. The French master E. E. Viollet-le-Duc claimed that “a style is an expression of the ideal, which rests on certain rules” (Kazhar, 2000, p. 223).

All the architects, who proclaimed the idea of synthesis of historical and contemporary forms, in their practice used the *method of eclecticism*. It was based on a free use of the whole architectural heritage, and was considered to be the main instrument for creating a new style.

The masters, who used the method of eclecticism, were opposed by a group of theorists-rationalists, who believed *the proper use* of material and design, and the account of utilitarian purpose of the building to be the main aspects in creating a new style (Fig.4.).

The formation of the rationalistic trend reflected the important contradictions of the architectural theory of the 19th century. On the one hand, we saw architects’ desire to preserve the traditional forms; on the other hand, they wanted to use modern technical achievements, which inevitably led to a departure outside traditional frameworks. This situation allowed the Swiss researcher W. Hermann to state in 1932 that “never in the 19th century, architecture in its buildings and aspirations was closer to the

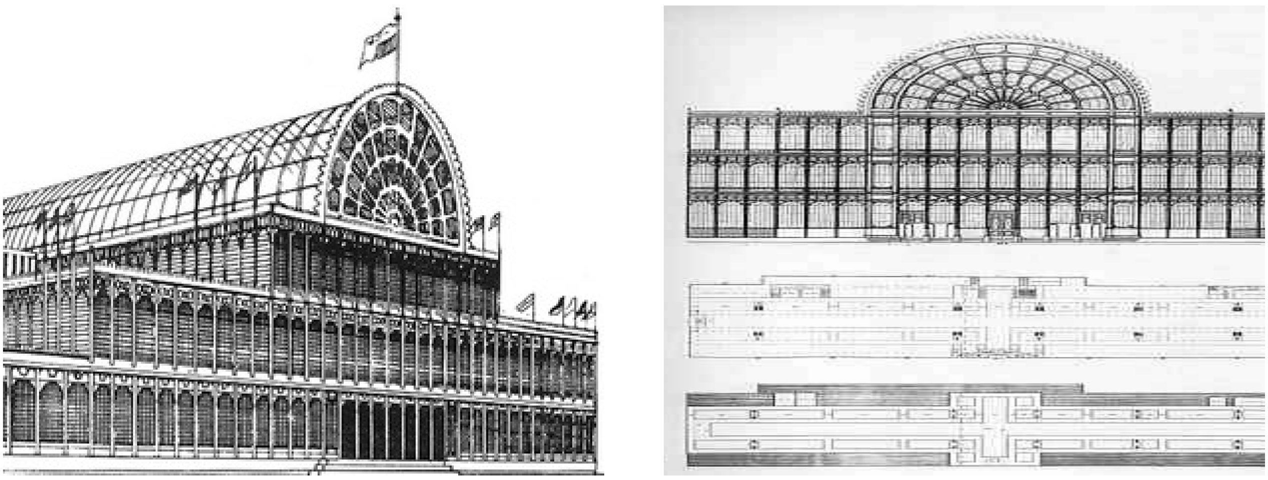

Fig.4. Joseph Paxton, The Crystal Palace in London, England, to house the Great Exhibition of 1851

modernity than in the decade from 1830 to 1840" (W. Hermann, 1977).

Theorists of the rationalist direction were opposed by the architects, for whom the *spiritual* and ideological-aesthetical aspects of style were the main points. They tried to find *some ideal period* in history and solve their contemporary problems through the use of the creative experience of that period (Fig.5.).

The main dispute was about the choice of the epoch to imitate. Most theorists thought that the right choice of the historical prototype will solve the problem. For example, G. Palm in his article "Style Distribution Among Individual Types of Buildings" (1845) tried to select *an appropriate style* for each type of buildings (Kazhar, 2000, p. 225).

The architectural theory of the 19th century was influenced by the progress of sciences, history and studies of the nature. The leading German theorist G. Semper defined the results of the style search from the standpoint of the "empirical theory of style." He categorized the debaters into three "schools" ("purists", "materialists" and "historians"). Similar to Semper, in 1863, S. Dali also identified three schools – the "historical", "eclectic" and "organic" ones.

The climax of the search for a new style was the contest announced by the Bavarian King Maximilian II in 1850. The programme of the contest closely tied architectural issues with public and social problems. The point was in the "architectural mission of the time" and in the role of architecture in "combining all the life interrelations and vital forces in the interests of the nation." The terms of the contest stated that the time was characterized by the development of the scientific thought and the ongoing long discussion about a possibility of creating a new architectural style. There was a need to change the situation of the first half of the century, when architecture "fluctuated between classicism and romanticism," while new forms evolved on the basis of turning to the past. Therefore, "there was an internal need to create new not only on the basis of borrowing the forms or fragments of the past, but to create something really new" (Kazhar, 2000, p. 229).

The programme oriented to creating something *new* was eclectic by itself: the style intended "to express the nature of the time" was to appear on the basis of historical prototypes. Each participant of the contest was allowed to "enjoy complete freedom with different architectural styles and their ornamentation for the expedient solution of the assigned tasks" (H. W. Kruft, p. 354). The search for the "new style", combining the "appropriate dignity and greatness", "practical expediency" and "economic efficiency" was to be completed with the creation of "a characteristic monument of arts and education." Although they hoped to get an "original, beautiful and organic whole", all the presented projects were pronounced examples of eclecticism (Kazhar, 2000, p. 229).

The contest outcomes were summed up by G. Semper: "Thus, at the highest royal will and order, in Munich, the Maximilian style appeared, which is based on the following idea: our culture is composed of elements of all previous cultures. Therefore, our modern architectural style should be a mixture of styles of all times and nations" (M. Brix, 1978, p. 197) (Fig.6.).

Semper saw a reason for the failure to create a uniform style of the epoch in the development of sciences and industry, which provided new materials at the disposal of architects. As to these materials, architects "firstly do not know how to use them; and,

Fig.5. Heinrich Hubsch. Search for a national style, Theatre of Karlsrue

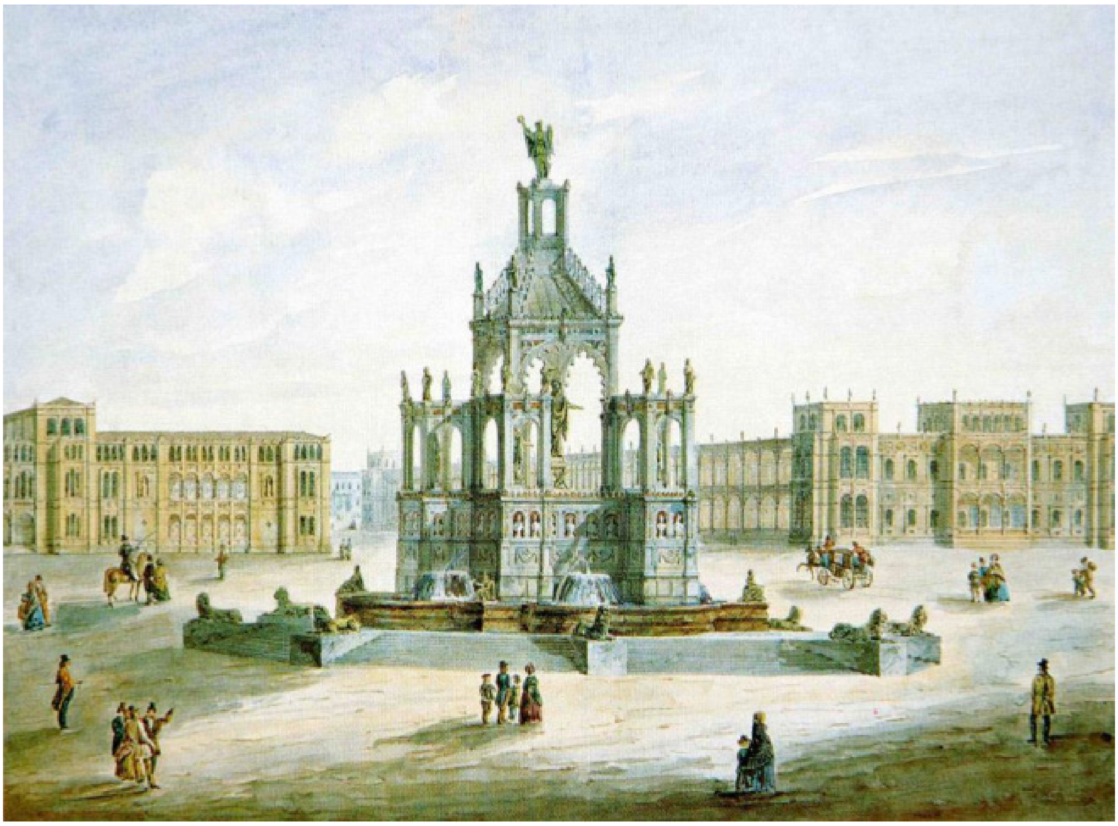

Fig.6. August Voit, Project of a Square in Munich, example of “Maximilianstil”, 1850

secondly, there are no social conditions,” which could contribute to rapid development of “new skills”. He saw another reason in architect’s dependence on the taste of the customer. He was dissatisfied with the situation, where a master had to deal primarily not with artistic tasks, but think about the way of expressing the “customer’s property status and social position.” The free artistic expression was also restricted by the new market relations, under which the “high art, too, entered the market” (Kazhar, 2000, p. 226). In the opinion of the theorist, an important reason for the decline of the art was also in “the absence of essential social ideas.” G. Semper defined the situation in the architecture of his time as “a state between destruction of the old and creation of the new” (Kazhar, 2000, p. 230).

The problems raised during the discussion on the new style of the epoch were not solved in the 19th century. The problem of style retained its relevance for the architectural theory in the decades that followed. In late 19th-the first third of the 20th centuries, the idea of the revival of “grand style” based on “eternal laws of creativity” was analyzed in the works by H. Wölfflin, K. Fidler, H. von Marées, A. Hildebrand and others.

2.2. Search for “New Global Style” of the 21st century

Today, we see a revival of the interest in the style of architecture. The topic is covered in the works by A. I. Dobritsina, A. V. Ryabushin, A. G. Rappoport and others. S. O. Chan-Magomedov suggested considering the history of architecture as a dialogue of international “super-styles”: classicism (based on the order architecture) and modernism (S. Chan-Magomedov, 2007). Nowadays, at the Ural State Academy of Architecture and Arts, L. P. Cholodova and her pupils attempt to identify the “fundamentals for the formation of the third global style, after classicism and modernism” and formulate the concept of the *super-style* of the new millennium, “which could consolidate and explain the latest global trends in architecture” (L. P. Cholodova, 2010).

Among foreign studies of recent decades, we can note the publications by U. O. Attoe, H. P. Bont, M. Vance, P. Eisenman, M. Tafuri and others.

In 2008, in London, Patrick Schumacher, published an article “Parametricism – A New Global Style for Architecture and Urban Design”[1]. Given the today’s absence of succession and a break of the links of modern architecture with its historical heritage, Schumacher suggested understanding the architectural style as a scientific-research programme and a paradigm.

[1] At the 11th Arcitectural Bienalle in Venice (2008), the article was called the “Manifesto of Parametricism”. The author is engaged – for the second decade already – in developing his theory of “parametric architecture”, based on several sciences: mathematics, biology, computer-based simulation and architectural programming.

Having highlighted *epochal* and *transient styles* in the history of architecture, Schumacher has introduced a concept of *auxiliary styles*, both historical and contemporary. For example, in modernism, we can distinguish functionalism, rationalism, structuralism, brutalism, metabolism and high-tech. All these intermediate styles of modernism have clearly followed the principles of functional designing: from the general to the particular. The postmodernism and deconstructivism have addressed historical styles in a new form by means of irony and collage (Fig.7).

The modern style of parametricism, on the one hand, is based on scientific methods and digital technologies; but, on the other hand, it creates new aesthetic criteria for the development of the newest system of form-shaping. Within this style, a number of *auxiliary styles* are developing: digital Baroque; digital morphogenesis; parametric urbanism, morpho-ecological designing and parametric ornamenting. Each of these auxiliary styles is developing its own architectural aesthetics, but they are all focused on the creation of new compositions from dynamically changeable geometric objects (Fig.8)

Fig.7. M2 building in Tokyo (architect K. Kuma, 1991)

Patrick Schumacher has noted that “although parametricism is rooted in digital animation techniques of the mid-1990s, it has fully manifested itself only in recent years with the development of advanced parametric designing systems. Now, parametricism has become a dominant and the *only style in the avant-garde practice*” (*highlighted by NK*). The author has emphasized that the new style “succeeds modernism as a new long wave of systemic innovations” and “terminates ... the transitional period of uncertainty, born by the crisis of modernism and notable by some brief episodes, including postmodernism, deconstructivism and minimalism” (P. Schumacher, 2008). According to Schumacher, parametricism “demands immensity in all areas – from architecture and interior design to large-scale urban planning” (Fig.9). This style precondition defines its “programmatic complexity” and its ability to adapt to the architecture and urbanism of the new “socio-economic era of post-Fordism” and “mass society” (P. Schumacher, 2008).

For Schumacher, a change of style means the achievement of a new level of development, a progress of architecture, and a process, in which the evolutionary development within the style is followed by a revolutionary leap and advent of a new style. For example, the crisis and decline of modernism led to the *current eclecticism*, which should be replaced by the *New Style*. The time will show whether it will be created, unlike the efforts of masters of the 19th century.

Fig.8. Galaxy Soho, Beijing, China. Zaha Hadid Architects, 2012

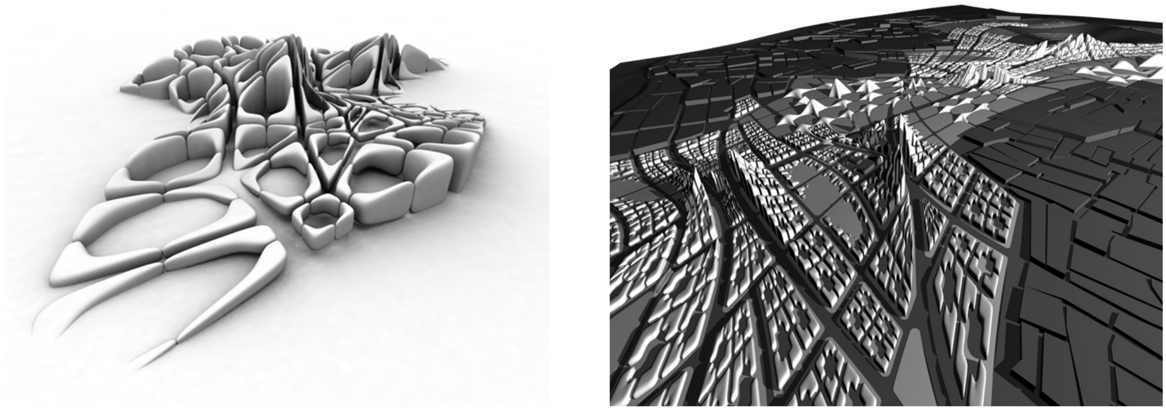

Fig.9. Zaha Hadid Archiects, Kartal-Pendik Masterplan, Istanbul, Turkey, 2006

3. Conclusions

As a result of a critical rethinking of the fundamental concepts of the architectural knowledge of the second half of the 20th-start of the 21st century, we can conclude that the formation of the modern creative thought is accompanied by a revision of the history of architecture and a change in evaluating its current status through the prism of today’s ideas about the spatial environment. The strategy and tactics for solving the problems of modern urbanism depend on the depth of understanding the relationships of architecture with the human world in all their complexities and diversities. Modern theorists have suggested that the *New Global Style* can be created through the synthesis of historical traditions with modern achievements of science and technology. A significant help in addressing the problem may be provided by “lessons of history”, in particular, the history of searching for the style of the epoch, which defined the content of the architectural thought of the 19th century.

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DEFORMABILITY AND STABILITY OF RECTANGULAR SANDWICH PANELS WITH CUTS UNDER IN-PLANE LOADING

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Abstract

In this work the method of defining the critical load for rectangular sandwich panels is described having within themselves definite length cuts in a direction parallel with one of their sides. The coefficients of stability are determined by equating to zero the bending moments and the shear forces at the borders of the cut. This condition together with the rest of the boundary conditions for a panel forms the system of homogeneous equations. By equating to zero the determinant the critical load is obtained

Keywords

cut; panel; hole; stability; generalized function; in-plane loading

1. Introduction

Thin-walled shell structures are applied in the various scopes of engineering: machine building, aircraft and shipbuilding. In the construction they are widespread as rational solutions of industrial, agricultural, trade, sport buildings coverings.

The most serious violations of regularities are cuts, apertures, holes and cracks. The various rigid inclinations as reinforcement bars also violate the regularity.

The stress concentration zones in the places of violation of regularity (end of rib, discrete constraints) make significant impact on load bearing ability and stability of thin-walled load bearing structures. At this, the known traditional analytical and numerical methods for study of mode of deformation of ribbed thin-walled structures are less effective.

In this regard originates the necessity of development of new effective methods of solution of mentioned class of tasks. Currently intensively developing theory of generalized, in particular, discontinuous impulse, functions drastically extent the possibilities of analysis of thin-walled structure with various violations of regularity.

The three-layered panel with light-weight filler and two outer load bearing layers would be widely applied in construction of residential and public buildings as prefabricated typical element, in that in accordance with destination of building would be various structural features and additions as additional constraints, apertures for arrangement of openings such as doors or windows.

The methodology of calculation of three-layered planar elements with cuts and holes on strength and stability is not very much developed. Currently, a simple and convenient methodology there is not available for obtaining engineering design formulae for the definition of critical compressive in-plane loading taking into account various structural features, including values and arrangement of the weak regions (cuts, holes etc.) mentioned above.

The significance of the proposed method of calculation of the stability of three-layered plates with cuts is underlined by its potential of being applied in the design of shell structures of up-to-date materials with increased requirements for their reliability as well as from the absence of reliable effective methods that could be efficiently applied in engineering calculations at design.

The modern advances of mathematics and mechanics provides the possibility to initiate studies of most complex problems related to research of the behavior of thin-walled structures having one or several cuts of arbitrary shape. This is possible due to the research efforts of famous scientists, with results of that are included in a number of published works, including monographs of A.N. Guz (1971), E.I. Grigoliuk (1973), G.N. Savin (1968), I.N. Preobrazhenski (1981), B.K. Mikhailov (1980) and others. However, the level of solution regarding the stability of such structural elements rather lags behind the requirements of practical applications.

This stated problem was earlier considered in a number of reviews; for example, of N.A. Allumae (1972), O.D. Oniashvili (1957, 1969), G.I. Janelidze (1948), N.A. Nash (1957), P.M. Naghdi (1972), L.M. Kurshin (1962), B.K. Mikhailov and G.O. Kipiani (1987), G.O. Kipiani (2013).

The construction of large-span buildings and implementation in practice of modern low-modulus materials with high strength characteristics leads to necessity of considering of large, in comparison with thickness, deflections at analysis of thin-walled structures.

The development of various fields of industry and construction is related with improvement of existing and creation of new thin-walled structures that includes shells, plates, rods, having reinforcements, breaks, apertures, cuts, point supports. Group of such singularities are called as discontinuous parameters (Mikhailov,1980; Vol'mir, 1967; Mikhaylov and Kipiani, 1996, 1988(1,2)).

Most natural method to increase the stiffness of shells – it is the arrangement of ribs. On the borders of holes, at increasing of local loadings is advisable to perform the reinforcement of spatial structures by definite length ribs. The methods of ribs fastening to skin have an impact on the deformation behavior of such structures (Mikhailov, Kipiani et al., 1989, 1991; Kipiani et al., 1995, 1992, 2008, 2012, 2013, 2014).

The singularities of geometrical and physical parameters in thin-walled structures cause essential concentrations of stresses and result in areas of crack origination or plastic deformation. The load bearing capacity of such panels in most cases is determined by the strength or buckling capacity of the stress concentration areas. The other kinds of violation of regularity are presented by breaks that occur in folded and multi-wave coverings. By their impact on stress state they are similar to ribs.

2. Basic mathematical formulation

In order to be able to consider a cut discontinuous functions are introduced in the geometrical relations of theory of thin three-layered bending plates with lightweight filler. This is achieved by assignment of displacement components as ratios (Fig. 1) (Savin, 1968; Mikhailov 1980):

$$u_1^* = u_1 - \Delta u_1 H_x H_{yy}; \quad v_1^* = v_1 - \Delta v_1 H_x H_{yy};$$
$$u_2^* = u_2 - \Delta u_2 H_x H_{yy}; \quad v_2^* = v_2 - \Delta v_2 H_x H_{yy};$$
$$w^* = w - \Delta w H_x H_{yy}; \quad \gamma_1^* = \gamma_1 - \Delta w \delta_x, \qquad (1)$$

where u_1, v_1 – are the in-plane displacements of points of median surface of upper load bearing layer within the plane of the panel; u_2, v_2 – are the same for lower layer; Δu_1, Δu_2, Δv_1, Δv_2 – are the divergence of edges on line of cut of median surfaces accordingly of upper and lower layers; Δw – is the out-of-plane displacement of edge on line of cut perpendicular to the plane of the panel**;** $\Delta\gamma_1$ – is the angle of break of median surface on line of cut; H_x=H(x-a_1), H(y-b_1), H(y-b_2) – are the Heaviside functions; H_{yy}=H(y-b_1)-(y-b_2) – are the special function consisting from unit functions; δ=δ(x-a_1) – is the delta-function of Dirac.

For obtaining the governing equations it is necessary to substitute the expressions of equation 1 by known ratios of elasticity for three-layered plates included in the equations of equilibrium of an infinite small element (Mikhailov, 1991). Towards this it is necessary to take into account that the values Δu_1, Δu_2, Δv_1, Δv_2, Δw and $\Delta\gamma_1$ are the regular functions of coordinate y within the cut and are equal at $y=y_1$ and $y=y_2$ as at the ends of the cut it is not possible to have stepwise changes of the displacement or of the angle of rotation because such changes will introduce breaks and discontinuities along the line of the cut. Next, proceeding with the necessary transformations it is possible to exclude the components of in-plane displacements, in a similar way as this is done in the theory of continuous plates, and thus reduce the system of equations to a single equation related to the function of the deflection (the out-of-plane displacement ***w***). This is stated by the following expression**.**

$$\left[2B\left(h+\frac{t}{2}\right)^2+2D\right]\nabla^4 w-\frac{Bh}{G_3}\nabla^6 w=$$
$$=\left(1-\frac{Bh}{G_1}\right)P+2\left[B\left(h+\frac{t}{2}\right)^2+2D\right]$$
$$\left[\Delta w\delta_x^{III}+2\Delta w_y^{II}\delta_x^{IV}+\Delta w_y^{IV}H_x\right)H_{yy}+$$
$$+\left(\Delta\gamma_1\delta_x^{II}+\Delta\gamma_{1y}^{II}\delta_x\right)H_{yy}+$$
$$+2\Delta w_y^{III}H_x\delta_{yy}+\Delta w_y^{II}H_x\delta_{yy}^{I}+\Delta\gamma_{1y}^{I}\delta_x\delta_{yy}-\frac{Bh}{G_3}\times$$
$$\times\left[\Delta w_y\delta_x^{VI}+2\Delta w_y^{II}\delta_x^{III}+\Delta w_y\delta_x^{I}+\Delta\gamma_1\delta_x^{IV}+\Delta\gamma_{1y}^{II}\delta_x^{II}\right]H_{yy}+$$

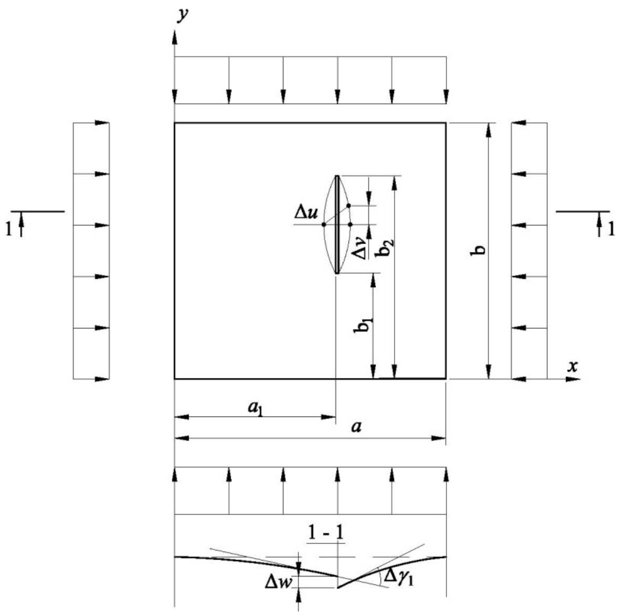

Fig. 1. Displacements Δu, Δ*v*, Δw and Δγ in upper and lower layers

$$+\left(\Delta w_y^{II}\delta_x^{III}+2\Delta w_y^{IV}\delta_x+\Delta w_y^{VI}H_x+\Delta\gamma_{1y}^{II}\delta_x^{II}+\Delta\gamma_{1y}^{IV}\delta_x\right)H_{yy}+$$
$$+\left(4\Delta w_y^{IV}\delta_x^{I}+2\Delta w_y^{V}H_x+2\Delta\gamma_{1y}^{III}\delta_x\right)\delta_{yy}+$$
$$+\left(\Delta w\delta_x^{'''}+2\Delta w_y^{II}\delta_x^{I}+\Delta w_y^{IV}H_x\right)\delta_{yy}^{I}+$$
$$+2\Delta w_y^{III}\delta_x^{'}\dot{\delta}_{yy}+\Delta w_y^{II}\delta_x^{I}\delta_{yy}^{I}+\Delta\gamma_{1y}^{I}\delta_x^{II}\delta_{yy}+$$
$$+2\Delta w_y^{V}H_x\delta_{yy}+\Delta w_y^{IV}H_x\delta_{yy}^{I}+2\Delta w_y^{III}H_x\delta_{yy}^{II}+$$
$$+2\Delta\gamma_{1y}^{II}\delta_x\delta_{yy}^{I}+2\Delta w_y^{III}H_x\delta_{yy}^{II}+\Delta w_y^{II}H_x\delta_{yy}^{'''}+\Delta\gamma_{1y}^{I}\delta_x\delta_{yy}^{Iy}\quad(2)$$

As it is known, at study of stability it is advisable instead the external load P introduce the transverse fictitious load equal to summand of projections of compressive and shear forces on direction of normal to the non-deformed surface. All notifications are same as in (1).

$$P=P_f=T_1w_x^{II}+2Sw_{xy}^{II}+I_2w_y^{II}\quad(3)$$

If in equation (2) it is assumed that the shear modulus G_f (It is modulus of filler) tends to infinity, then the governing equation for a three-layered plate with a cut is reduced to the equation for single-layered plate with a cut; the bending stiffness of this sandwich plate is equal to $2\left[D+B\left(h+\frac{t}{2}\right)^2\right]$ and represents the stiffness of a section with two load bearing layers at distance of 2h from each other. Such equation was obtained in an earlier work [1].

The coefficients Δw and $\Delta\gamma_1$ are determined from conditions of equality to zero of moment and shear force on edges of cut. If only takes place break without divergence of edges, then would be accepted Δw=0, that commonly occurs at external compression on contour. Then the equation of stability will be as stated in previous work (Kipiani, 2008)

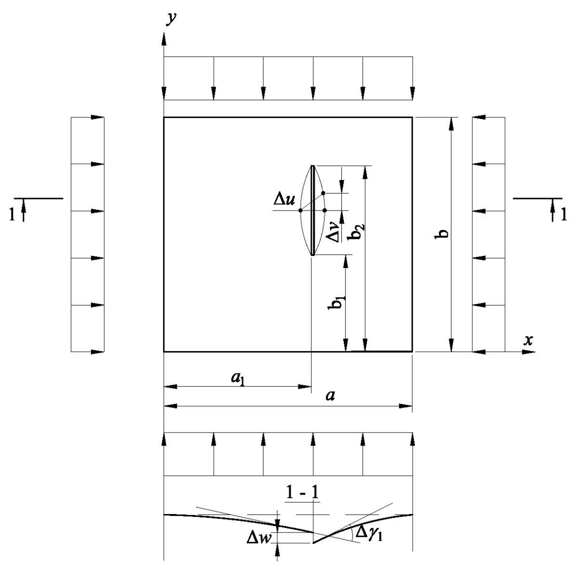

Fig. 2. Design diagram for compressed three-layered plate with cut

$$2\left[B\left(h+\frac{t}{2}\right)^2+2D\right]\nabla^4 w-\frac{Bh}{G_3}\nabla^6 w=$$
$$=\left(1-\frac{Bh}{G_1}\right)\left(T_1^0w_x^{II}+2S^\circ w_{xy}^{II}+T_2^0w_y^{II}\right)+$$
$$+2\left[B\left(h+\frac{t}{2}\right)^2+2D\right]\times$$
$$\times\left[\left(\Delta\gamma_1\delta_x^{II}+\Delta\gamma_{1y}^{II}\delta_x\right)H_{yy}+\Delta\gamma_{1y}^{I}\delta_x\delta_{yy}\right]-$$
$$-\frac{Bh}{G_f}\left[\left(\Delta\gamma_1\delta_x^{IV}+\Delta\gamma_{1y}^{II}\delta_x^{II}+\Delta\gamma_{1y}^{II}\delta_x^{II}+\Delta\gamma_{1y}^{IV}\delta_x\right)H_{yy}+\right.$$
$$+\left(2\Delta\gamma_{1y}^{III}\delta_x+\Delta\gamma_{1y}^{I}\delta_x^{II}+\Delta\gamma_{1y}^{III}\delta_x\right)\delta_{yy}+$$
$$+2\Delta\gamma_{1y}^{II}\delta_x\delta_{yy}^{I}+\Delta\gamma_{1y}^{I}\delta_x\delta_{yy}^{II}.\quad(4)$$

Let's assume that plate is compressed in direction perpendicular to line of cut, by load, uniformly distributed on two apposite edges *x*=0, *x*=*a*. Then $S^\circ=T_2=0$. If we assume that the edge conditions give the possibility in the first approximation represent the function $w_1\Delta\gamma_1$ as

$$w=w_1(x)\sin\beta_1 y;\ \Delta\gamma=\Delta\gamma_{1(1)}\sin\overline{\beta}_1 y,\quad(5)$$

where

$$\beta=\frac{\pi}{b};\ \overline{\beta}=\frac{\pi}{b'}.$$

Then the solution of equation (4) would be written down as

$$w=\left[w_1(x)+\Delta\gamma_{1(1)}f_1(x)\right]\sin\beta_1 y,\quad(6)$$

where the function $f_1(x)$ has an discontinuous character that corresponds for distribution of displacement and angle of rotation and would be written down as

$$f_1(x)=2\left[B\left(h+\frac{t}{2}\right)^2+D\right]\left[\left(\psi_x^{''}-\overline{\beta}_1^2\psi_x\right)a_1^0+\overline{\beta}_1\psi_x a_1\right]-$$
$$-\frac{Bh}{G_f}\left[\left(\psi_x^{1V}-2\overline{\beta}_1^2\psi_x^{''}+\overline{\beta}_1^4\psi_x\right)a_1^0+\left(-2\overline{\beta}_1^3\psi_x+\overline{\beta}_1^3\psi_x\right)a_1-\right.$$
$$-2\overline{\beta}_1^2\psi_x a_1^{'}+\overline{\beta}_1\psi_x a_x^{''}.\quad(7)$$

The function $w_i(x_i)\psi_x^{''}$ represents the solution of the relevant equations

$$2\left[B\left(h+\frac{t}{2}\right)^2+D\right]\left(\frac{d^2}{dx^2}-\beta_1^2\right)^2 w_1(x)-$$
$$-\frac{Bh}{G_f}\left(\frac{d^2}{dx^2}-\beta_1^2\right)^3 w_1(x)=\left(1-\frac{Bh}{G_f}\right)T_1^0w_x^{II};$$
$$2\left[B\left(h+\frac{t}{2}\right)^2+D\right]\left(\frac{d^2}{dx^2}-\beta_1^2\right)^2\psi_x^{''}-$$
$$-\frac{Bh}{G_f}\left(\frac{d^2}{dx^2}-\beta_1^2\right)^3\psi_x^{''}=\delta_x^{''},\quad(8)$$

$h=0,1,2,3,4.$

The function $w(x)$ would be presented as:

$$w(x)=w_1\sin\alpha x,\quad(9)$$

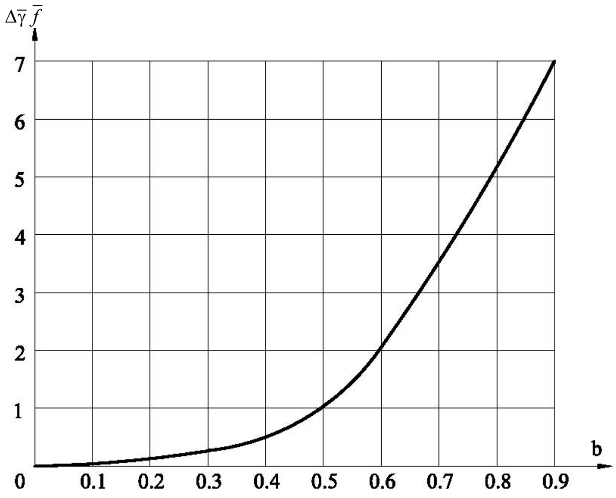

Fig. 3. Diagram of dependency of $\Delta\bar{\gamma}\bar{f}$ from length of cut

where

$$\alpha = \frac{\pi}{a}.$$

The coefficient $\Delta\gamma_{1(1)}$ would be determined from condition to equality to zero of moment on line x=x_1. This condition together with the boundary conditions on the edges x=0, x=a represent a system of simultaneous equations; next, the condition of equality to zero of their determinant forms the basis on which the critical loading is found.

As case let's consider the analysis on stability of square sandwich plate with symmetrically located cut, length of that is equal to one third of side of rectangular contour.

Results of calculation:

In the first approximation $T_{cr} = 0.42T_{cr}^{\circ}$

In the second approximation $T_{cr} = 0.687T_{cr}^{\circ}$

In the third approximation $T_{cr} = 0.774T_{cr}^{\circ}$

In the ninth approximation $T_{cr} = 0.933T_{cr}^{\circ}$

In the tenth approximation $T_{cr} = 0.935T_{cr}^{\circ}$

where T_{cr}° – is the critical loading on continuous sandwich plate.

As the second case is considered same dimensions plate. But the successive approximation process in this case is converged better. So in the first approximation we have

$$T_{cr} = 0.942T_{cr}^{\circ}$$

and in the second approximation $T_{cr} = 0.932T_{cr}^{\circ}$.

At this rather simply would been observed the dependency of coefficient $\Delta\bar{\gamma}\bar{f}$, and accordingly, reducing of critical loading on length of cut. The diagram of dependency of $\Delta\bar{\gamma}\bar{f}$ from length of cut is presented on Fig. 3.

As the second case is considered sandwich square plate with symmetrically located aperture. the length of that is equal to one third of plate external contour length.

The calculation gives the value of critical loading (in the second approximation)

$$T_{cr} = 0.715T_{cr}^{\circ}.$$

For comparison with respect of continuous plate we have

$$T_{cr} = 0.755T_{cr}^{\circ}.$$

By solution of corresponding static task, the process of successive approximation is convergence significantly better.

3. Conclusion

The basic mathematical formulations are presented that can be utilized towards the determination of the critical loading for a rectangular sandwich plate having a cut of finite length within such a plate parallel to one of its sides.

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THE PHENOMENON OF THE SAINT PETERSBURG VARIANT OF THE REGULAR CITY

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Abstract

Saint Petersburg building development formed by the first third of the 19th century is under review. At that time the idea of regular city design was put into effect to the fullest extent. The most significant and specific quantitative data are given based on literature references and drawings. It is shown that besides strict administration over design and construction activities, the following basic conditions influenced on achievement of such result: parceling at the initial stage of the city organization, metropolitan ambitions supported by appropriate financing, considerable volumes of state orders, completion of main building development within a short period of time, involvement of skilled professionals.

Fundamental distinctive features are found out between regular design of Saint Petersburg's urban development and traditional European city as well as an "ideal city".

Keywords

Saint Petersburg architecture, classicism, development control, parceling, order placement

1. Introduction

The paper purpose is defined by the following statement of city planning theoretician V. Ostrovskiy: "In fact, it was not architects who constructed big modern cities, it was legislators who created a structure to be implemented by interested parties. Architects were only allowed to apply external embellishment to unchangeable structure determined by prevailing conditions." (Ostrovsky, 1979) The paper determines influence of particular conditions on organization of Saint Petersburg regular building development. Traditionally, history of architecture focuses on unique facilities such as sanctuaries, palaces, and castles. The main peculiarity of the historic downtown of Saint Petersburg is its architectural complex, where a residential building, as an object of mass building development, is a meaningful element of urban complex. Analysis of residential building development of the city allows to cover the most significant aspects.

2. Materials and Methods

Data of literature, drawings, maps and schemes are used; Vista of Nevsky Prospect in 1830-ies is under thorough analysis (Kotelnikova, 1974; Margolis, Sementsov, 2004). The theory and subjective assessments related to architectural features of Saint Petersburg are compared with documentary evidences given, in particular, in works by S.V. Sementsov (2006(1,2)). The most significant factors of regular city building development organization are found out. Comparison of architectural and urban planning features of Saint Petersburg with a traditional European city as well as with an "ideal" fortress city clearly demonstrates peculiarity of architectural look of Saint Petersburg.

3.1. 1830ies – peak of "regular city design" implementation in Saint Petersburg

In 1833, population of the city equaled 440,000 people. It was one of the largest European metropolises. The number of buildings equaled 7,976, including 5,246 of wooden buildings. Small wooden constructions composed the building development at the northern bank of the Neva river (at the Petrograd Side), at the western part of Vasilyevsky Island and to the south of the Fontanka River. The most of stone buildings (total of 2,730) were located in the center of the city between the Neva and Fontanka rivers. This particular part of the city represented the appearance of the Russian Empire capital city. Its urban planning features complied with the standards of an "ideal city": clearly delineated regular urban planning was applied onto completely flat surfaces of the Neva Delta isles. Intersection of straight streets and loose river bends created exceptional view of this area. By that time all streaming waters within the city center were embanked with granite and cast-iron enclosures, trees were planted at the banks. These made the local landscape charming.

Organization of urban building development complex in the city center was completed. Saint Isaac's Cathedral was still surrounded by construction trestle, but all architecturally iconic buildings were already on their places, i.e. complex of the Spit of Vasilyevsky Island, the Admiralty building, Architect Rossi street, architectural complexes of Nevsky Prospect area and near Palace Square. These dominant structures perfectly harmonized with high-quality background building development, thus ensuring integrity of Saint Petersburg urban environment. A.N. Benois highly appraised the city center landscapes: "Its beauty is in its entirety or, more correctly, solid lumps, large ensembles, wide panoramic views." (Benois, 1902).

Figure 1 shows important compositional function of Nevsky Prospect. To the north its axis looks out onto the group of squares surrounding the Admiralty building and linked with the Neva water area. This place commands a good view on the Spit and the Peter and Paul Fortress. The area of Nevsky Prospect between the Fontanka river and the Catherine Canal opens to Mikhailovskaya Square and Ostrovsky Square along with Teatralnaya Street (Architect Rossi street).

Principle of architectural complexes strongly affected organization of urban buildings image. Image of a building was determined not by its functional purpose, but by its place in general view of urban landscape. There were just a few accenting elements and all other buildings were secondary objects of the composition. Residential and commercial buildings, barracks integrated into single area represented a kind of "fabric" and dominant structures stood out of this background.

Organization of Mikhailovskaya Square complex is typical to this approach. According to the plan of K.I. Rossi, the palace of Grand Duke Mikhail Pavlovich of Russia should be the dominant of the complex: a presentable building was located in the center of the composition at the end of Mikhailovskaya Street with a view to Nevsky Prospect. In parallel with design of the Palace, K.I. Rossi drew developed views along the building lines of the Square and Mikhailovskaya Street and drafted frontispieces of all buildings. Using these sketches, all other architects were involved into development of design and planning of each particular building. Residential buildings, theater and the Gentry assembly did not distinguish from each other.

Architect Rossi street is considered to be a significant architectural complex – the propylaea opening towards the Alexandrinsky Theater. The

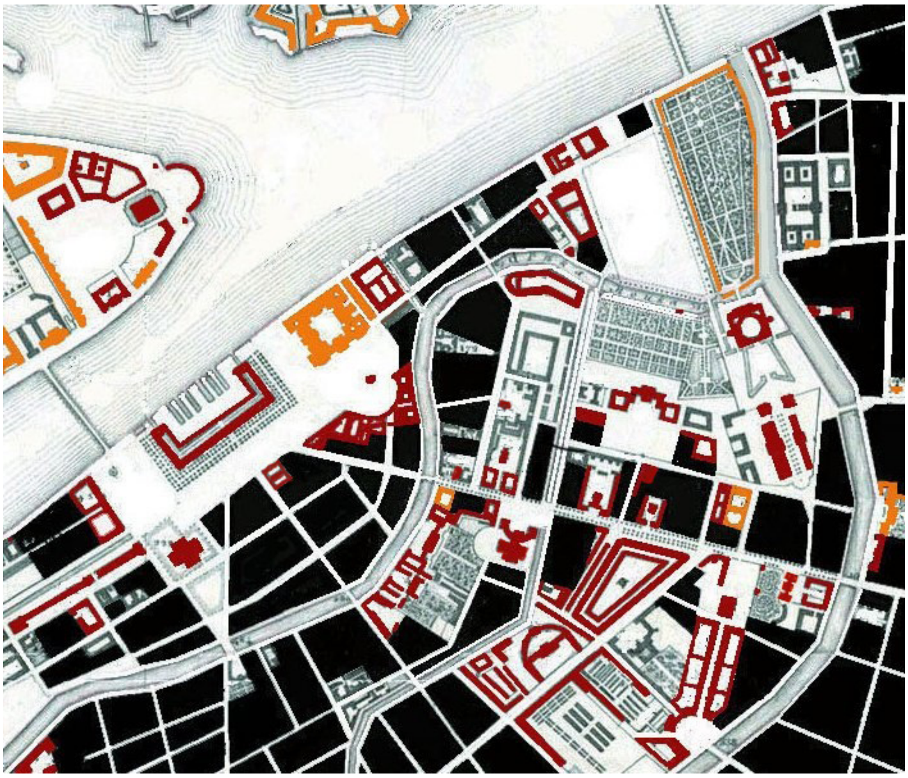

Fig.1. Scheme of the Saint Petersburg central area, 1821. The following colors denote civil buildings: yellow – Baroque, red – Classicism

street is 220 meters long and is formed with two totally symmetrical buildings located opposite to each other. Building # 1/3 was constructed as a place of military educational establishments, but the Ministry of National Education and Ministry of Internal Affairs were finally located there. Building # 2 was constructed as a commercial apartment building and at the beginning its ground level consisted of small shops; while apartments and a hotel were located on the upper levels. But soon the building was assigned to the Directorate of the Emperor's Theaters. There are no differences between appearances of these buildings, their frontispieces are totally identical.

Fig. 2. Appearance of buildings with various types of functionality (contemporary photographs):
a – residential house of Yakovlev, Sadovaya street; b – administrative building of the National Revenue Distribution Center, Sadovaya street; c – barracks of Pavlovsk regiment at the Field of Mars

Presentability of buildings was of importance in the capital. Residences of the Emperor's family members, palaces of the court notables, administrative buildings, objects of the Military Department constructed in the city center set the style. Their solemn but austere and official appearance served as an example for residential building development. If in a "European city" a town hall often looked like a house of a well-to-do man, then in Saint Petersburg a commercial apartment building was constructed to be similar to palace. Fig. 2 shows similar appearances of a residential house, administrative building and barrack in the center of Saint Petersburg.

The main street view of that time is documented in Vista of Nevsky Prospect by V.S. Sadovnikov (Kotelnikova, 1974). It depicts 108 buildings, including 98 residential buildings. Uniformity of the picture is surprising: all structures consist of 3–4 storeys and have long frontispieces. Buildings stretching along 9–13 axes are prevailing, but longer buildings are not rare objects: building # 30 consists of 19 axes, building # 18 consists of 31 axes, building # 20 consist of 37 axes (building # 29 is an exception as its frontispiece opens to Nevsky Prospect and it is 4 windows long). The style is also common: 107 building out of 108 are the examples of classicism (except for Beloselsky Belozersky Palace that is of baroque style). The Vista shows a unique situation as Nevsky Prospect of 1830-ies with its even (in all aspects) building development could be considered as an illustration of utopian "ideal city".

Classical building development prevailed and determined the views of urban landscapes. The squares had clear geometry outlines. The face of building block development stretched along the straight line. Buildings located along the streets had no gaps between, they were adjacent to each other and had the same height. Thus, a wall was created which stretched from one crossroad to another. Its length could equal to 447.3 m on Vasilyevsky Island, to 426–553.8 m at the Liteyny area where building blocks were of significant size.

3.2. 1703–1717 — Saint Petersburg is a "window on Europe", a new city with dispersive urban planning

A.N. Benois wrote: "Rush towards the West has existed from the very origin of Saint Petersburg. Even its foundation was a result of this rush." (Benois, 1902).

Peter the Great considered foundation of Saint Petersburg a determinative step towards implementation of a strategic plan for strengthening relations with Europe and was ready to face all the difficulties in pursuing this goal. And there were a number of difficulties. Saint Petersburg was founded on a sparsely populated territory without any considerable settlements. The nearest Russian city was Novgorod which was located almost 200 km apart. But there were no suitable roadways to travel. Waterway was the only possible method of communication with central Russia. Under these conditions a fortress was erected on a small island in the Neva Delta. On the inland, to the north of this fortress, a temporary settlement was arranged in a random way. At the same time, on the southern bank of the Neva a shipbuilding yard was founded; and buildings of palace type along with the Summer Garden were constructed nearby. The Governor Palace, a building for museum and scientific establishments were founded on Vasilyevsky Island. In 1712, government authorities were re-located to Saint Petersburg. Residential settlements along the rivers expanded. On the bank of the Gulf of Finland in Peterhof, the Palace of Emperor was found. A possibility of main building development on the Kotlin Island was considered. The first decade of Saint Petersburg existence impresses with variety and spread of sites developed. New buildings were constructed in scattered way: there was no documentation to regulate these activities. There was no single general city planning scheme. Peter the Great was to decide on appearance of the Empire's capital.

He found prototypes for a new city when traveled throughout Europe. We cannot determine the particular details of diversity which impressed him the most. A.N. Benois used a general term of a "European city" (Benois, 1902). This form allows to discuss views of multiple populated localities, first of all those of randomly developed historical settlements.

By that time history of the most "European" cities accounted for many centuries. A successful city had a right to its own market, possibility of self-administration based on its own laws, had its own court and clear borders (frequently such borders were represented with fortified walls). A medieval city (in particular, a city with a status granted on the basis of Magdeburg rights) attracted people from non-urban areas with a possibility to get some of the privileges ("atmosphere of a city makes people free"). So, as a rule, such city was densely populated and developed. The community attempted to ensure each citizen with the same rights standard conditions for house construction within the city area; it controlled land-use conditions and established the land size. The idea of such city could not be implemented by Peter the Great: firstly, the rules of Russian absolute monarchy had no allowance for self-administration of cities; secondly, due to desolation of the near-Neva area no one could reckon upon accidental drift of the population within this period of time.

When arranging a European-type city, Peter the Great deemed it necessary to employ the services of foreign specialists. They considered the conditions of Saint Petersburg non-conventional and comparable with overseas foundation of fortified tactical localities by Europeans. When designing the capital of Russia, it was decided to base on experience of "regular city" design in Europe that gained widespread in the form of a residence city or fortress city. Anyway, the developments provided in 1715–1717 by Jean-Baptiste Le Blond and Domenico Trezzini were based on an "ideal city" and were considered as a starting point of Saint Petersburg arrangement on the basis of a planning scheme.

3.3. 1717–1737 – a new city with a center on Vasilyevsky Island

Le Blond and Trezzini's projects took into account the ambitions of the huge Empire capital city. So these projects differed from European concepts of an "ideal city" by giant size of the territory covered and colossal sizes of urban elements (the main square proposed by Le Blond had overall dimensions of 446.2 x 446.2 m (Sementsov, 2006(2)). Le Blond and Trezzini's concepts did not match by many aspects, but both of them agreed that Vasilyevsky Island should be the city center and the location for the bulk of the population. Peter the Great also supported this idea. In a short time, on the eastern point of the Island, the base of the important functional center of the future capital city was founded. Colossal structures that were novel to this area were erected on the swamp meadow: administrative complex of Twelve Colleges was 400 meters long; and perimeter of Gostiny Dvor building equaled 740 meters. These buildings outlined the area of the future large presentable square compatible in its size to those of some medieval European cities.

Upon development of the other parts of the Island, the concept of large scale was implemented as well. Rectangular street breakdown was determined by base dimensions of a residential block, namely 447.3x140.6 m. The main traffic road of the Island was determined by the distance between the lines of building frontispieces equaling 85 meters and the width of cross streets was determined at about 30 m. These overall dimensions were determined based on designing location of channels along the axes of these streets. No commercial cost was determined for the land in the new city, thus large areas could be allocated for the purpose of building development. They had the depth of about 30–40 m. Based on the length of frontispiece line, they were divided into small (one module, 21.60 m in length), average (one and a half module, i.e. 32.40 m) and large (two modules, 43.20 m in total length) ones (Sementsov, 2006 (1,2)).

Two main types of building development were worked out. Within the areas where ambitions of a capital city were of critical importance (the Neva embankments), structural volumes consisted of 2-2.5 storeys, were adjacent to each other with their end walls and formed a continuous face frontispiece of the block. They reminded Peter the Great the landscapes of Dutch cities he loved so much. However, the most common were town mansions designed for living of a single family and capable of cattle handling, as well as garden laying-out. Such houses retained traditional views of Russian cities.

Modest buildings prevailed consisting of one or one and a half storey. It was explained by financial reasoning of building owners and lack of desire to invest in construction activities, rather than technical capabilities. Accommodation of the new capital was a difficult process as the city did not attract people. There was no motivation for construction activities with the purpose of profit generation. So, as a rule, each housing building had one household. If the land was allocated to craftsmen transmigrated to Saint Petersburg, then their houses in some cases were built at the government expense. Individuals were forced to invest into building development. In 1713, the following ordinance was issued: all individuals considered as noblemen and nobility having minimum of 30 peasants were to reside in the capital on continuous basis and have their own houses in Saint Petersburg (this method was approved in France by Louis XIV: only noblemen could get a promotion; if an individual left for another location, he lost his privileges).

Some of the structures were located along the building line. But they were small and occupied only a part of the area; fire breaks were retained between such buildings. Wooden and mud-walled semi-permanent houses prevailed. To make these houses look more presentable, they were painted as red brick walls. Only high-rank officials had ambitious two-storey palaces. House of Menshikov, the General Governor, stood out of the other buildings. It was a three-storey building with a basement.

Only a portion of intentions of Peter the Great was implemented. He died suddenly in 1725. At that time there were 109 stone building in the city; the population reached 40,000 people.

The following conditions determined the city appearance and the ways of its organization at the beginning:

- the city was founded at the location without any infrastructure
and material-supply base;
- the state had the title to land;
- the city accommodation and building development were forced (“administrative resource”);
- open market rules did not work;
- the state was the main investor;
- the system of administrative control over building development started up;
- skilled professionals, mainly the foreign ones, were involved into design and construction activities;
- one of the main objectives was creation of presentable architectural appearance of the new capital.

After the founder’s death, Saint Petersburg faced a sequence of difficulties:

- for the period of 1728–1730. the Imperial Court and governmental establishments were moved to Moscow. The city lost its functions of a capital. Life in Saint Petersburg faded. Many of its citizens who were forced to move to the northern capital left Saint Petersburg.
- in 1736 and 1737 two terrific fires destroyed the city. Only one third of the buildings survived in the central Admiralty area of the city (Lisovsky, 2004). In principle, the concept of the city development had to be renewed after those fires.

3.4. 1737–1837 – a city of baroque and classicism with the center at the Admiralty Side

When the Emperor’s family returned to Saint Petersburg, the city center was renewed at the Admiralty Side. Main government establishments were located here, new Winter Palace was constructed in a short period of time. Construction activities revived. In 1737, immediately after the wild fires, a planning scheme was developed under the direction of P.M. Yeropkin. This scheme determined layout of three main traffic roads of the city that began from the spire of the Admiralty building, and outlined planning solution for other areas, including those located to the south of the Fontanka river. This plan can be considered as an important document. It determined the second stage of Saint Petersburg regular development.

At this stage some of the conditions determining the building development changed:

- forced city accommodation and private construction used at the time of Peter the Great were cancelled;
- the city became attractive and the amount of private investments equaled to the state financial support;
- the state remained the largest investor financing construction of administrative buildings, Ministry of Defense objects, hospitals and educational institutions;
- the required infrastructure was created (main traffic roads and channels); quays in the city center were reinforced and bridges were constructed;
- tough administrative control over the building development was finalized and effectively implemented;
- design and construction activities were carried out by skilled professionals (numerous outstanding architects and engineers worked in Saint Petersburg);

- importance of the capital architectural appearance presentability was highly emphasized;
- stone buildings comprised the smallest part of the city buildings;
- brick structures with plastered frontispieces prevailed in the city center.

By the end of the 18th century, living conditions of Saint Petersburg had changed. Voluntary accommodation of the capital led to the need in new residential buildings. Motivation for civil engineering development appeared. The building development became denser. Extra storeys were added to residential buildings. One- and two-storey mansions were upgraded into multi-storey buildings suitable for several families' living.

In 1765, building regulations were put into effect, aiming at improvement of royalty appearance of the capital city:

- it was decided to arrange buildings in a single frontispiece so that they were adjacent to each other. Breaks within the building row were filled with the structures and the length of each frontispiece was equal to the length of the land along the building line. As a result, building block development became to appear as a single wall from one cross road to another;
- a rule of “single cornice” construction was put into effect. In remote areas one- and two-storey buildings existed within one building block. But in the city center increase in the building height complied with architects’ concepts. In 1765, the Building Committee asked Catherine II to set the height of embankment building at the level of 10 sagenes in order “the structures along the Neva river complied with stone embankment under construction”. By 1830, three- and four-storey buildings prevailed at Nevsky Prospect and at some other areas of importance.

However, when five-storey buildings were constructed, the authorities decided to constrain ambitions of individual building owners. According to the ordinance executed by Nicholas I, height of private buildings could not be above the cornice of the Emperor’s residence, i.e. Winter Palace (23.47 m). This action was taken with consideration of European experience where constrains were applied to building height due to fear of building collapse and for the purpose of fire safety as well as with regard to prestige.

Since mid-18th century, trend of areas consolidation and buildings integration can be observed in the city center. Some particular examples: in 1757–1758, two areas on the Neva embankment were joint together and standard residential buildings constructed here under the project of S.I. Chevakinsky were rearranged into a single building (currently building # 11, Lieutenant Schmidt embankment). In 1791-1793, L. Ruska erected a two-storey building with the length of 27 axes on two consolidated areas of the 1st line on Vasilyevsky Island.

Administrative control related to design and construction activities with regard to style became extremely tough at this period of time. For example, the Committee headed by A. Betancourt targeted at the “correctness, beauty and propriety of each building as applied to the entire city” (Sementsov, 2006(1)). Thus, uniform style of building development in the center of Saint Petersburg was achieved by natural focus of architects on the use of the leading architectural concept of that time as well as by direct measures of control over design and construction activities. A phenomenon of Saint Petersburg comes out in integrity of its appearance as a result of cooperation between a number of outstanding craftsmen of the “age of classicism – the golden age of the capital city art of building” (Benois, 1902).

In the process of restoration, the buildings got their new appearance meeting fashion trends and administrative requirements. The possibility of frontispiece renewal in Saint Petersburg was facilitated with the use of plaster intended for creation of face cover. So, at relatively low costs it was possible to change the style characteristics. The process was widely implemented in the beginning of the 19th century when classical frontispieces were applied to baroque buildings. A quarter of the buildings depicted on the Vista of Nevsky Prospect subjected to this change. Concepts of urban development dated back to the 18th century were retained, but Peter’s Petersburg was buried under the number of added extra storeys and covered with decorative plaster.

3.5. 1840–1917 – post-classicism in Saint Petersburg

The period of the city rapid growth: in 1850 the population of the city was 487,000 people; in 1891 it equaled to 1,033,000; in 1900 it equaled to 1,416,000; in 1917 it equaled to 2,300,000.

The stage of classicism is over. The concept of regular building development is criticized more often. At that, traditional and randomly developed settlements are claimed to be the positive options for “ideal cities” of Europe. According to an emotional saying of N.V. Gogol, “new cities have no appearance: they are too correct, too smooth… An old-style German town with its narrow streets, colorful houses and high bell towers looks more vivid” (Gogol,1950). This saying amazingly complies with the assessment of “ideal” city fortress arrangement from Laugier, a French theoretician (1713–1769): “... tasteless straightness prevails there along with cold uniformity; it makes us regret for our old disordered cities...” (Savarenskaya, et al., 1989).

In early 1840-ies the authorities decided it reasonable to refuse from “uniformity in urban development". Decisions on formation were taken by Emperor's orders of 1843–1844 which dictated to use frontispieces of "non-uniform appearance". By administrative orders it was authorized to "paint residential houses with different colors from the outside" (Sementsov, 2006(1)).

Conditions of building development were changed:

- development and upgrade of the required infrastructure were carried out;
- non-state investments were made in the main construction volume;
- aggregation of capital at individual house owners increased; loan scheme for building owners improved;
- building development control was relaxed; but the rule of constrained building height was still applicable;
- no more attention was paid to presentability of the city architectural appearance;
- design and construction activities were carried out under direction of qualified professionals;
- construction technologies, structures and materials were developed.

Rapid growth of the population, increased demand in residential buildings inspired construction fever. Improved financial abilities of building owners led to increase in building dimensions: "a building owner bought several neighboring areas and created a single large block by means of old structures' rebuilding and overbuilding" (Punin, 1981). Relaxed building regulations were put into effect. Five- and six-storey buildings were allowed in the historical center. Appearance of Petrograd and Vyborg Sides, areas behind the Fontanka river and near Smolny were transformed. In 1887 there were 96.5 residents at the average in a single residential building in Petersburg; 48.5 residents in Moscow; 32.6 residents in Paris; 59.4 residents in Berlin ("barrack of Europe") and 7.8 residents in London (Kirichenko, 1963).

Constraints on buildings height were still in effect in Saint Petersburg, but the city started to lose its individuality. As in any other European metropolises, the rule of building development in a single frontispiece was not effective anymore. Huge residential building blocks were constructed with courts of honour and internal yards systems. The streets were filled with structures having various decorations of their frontispieces. Egyptian and Moorish styles were used as well as neo-gothic, neo-renaissance, neo-baroque ones.

Restoration of number of houses in the city center damaged the landscapes of classical complexes. Frequently, only building frontispieces were renovated in a stylish way. Advertising panels worsened the appearance of streets. Design uniformity of Nevsky Prospect depicted in Sadovnikov's Vista was violated and within further "150 years the prospect turned into a sampler of styles used in Petersburg architecture – from baroque to modern and neo-classicism" (Kirikov et al., 2004).

3.6. 1917–late 1950-ies

The following data on change in the population are evidencing the hard time of the city: 1917 – 2,300,000 people; 1920 – 740,000 people; 1939 – 3,190,000 people; 1945 – 947,000 people; 1959 – 3,320,000 people.

Conditions of building development arrangement:

- breaks in construction activities during the Civil war and the Great Patriotic war;
- the city lost its function of the capital city and, therefore
- its financial possibilities were decreased;
- the state became the main investor again;
- all the city land got the status of state property;
- implementation of tough control over design and construction activities directly applied to architectural style.

At this time rather small amount of construction work was carried out. But it became the history of domestic architecture due to residential areas (*rus. zhilmassiv*), community houses and the houses for skilled professionals of constructivism time. The time of Stalinist Empire style is of interest due to renewal of "single cornice" concept, application of transformed compositional methods and the Russian classicism stylistics. During the post-war period, historical appearance of a number of buildings in the city center was restored.

3.7. 1960–1990-ies

Large-scale housing construction programs were approved; new large "bedroom towns" appeared at the city outskirts, that were designed and implemented as residential communities, so they can be classified as "generated settlements".

4. Results of investigation

Regular building development of Saint Petersburg which had taken shape by 1830 was implemented on the basis of projects developed by skilled professionals and under the conditions of tough administrative control and monitoring (as well as over style of the buildings). However, it was not a one-step action and was implemented in series of steps:

1717 – generation of a concept of a regular city design, its implementation through a number of urban planning and architectural aspects; implementation of parceling idea and allocation of large areas for building development.

1737 – creation of the long-term development plan based on the regular city concept, determination of the city center, setting up of parceling idea and allocation of large areas for building development.

1765 – switching to urban development system with "house-to-house frontispiece" and "single cornice"; increase in private investments in building development.

1830-ies – peak of regular building, uniformity of the city center style.

1840-ies – statutory refusal from tough regulation over architectural style; height constraints retained.

1900–2000-ies – height constraints retained in the city center; generation of the protection system for historical sites.

Table 1 lists the conditions which determined the regular design of building development and style uniformity in the Saint Petersburg center:

- the city was founded in a new place, the state had the title to all lands, and that facilitated freedom of urban planning and allowed to allocate large areas for building development;

- status of the capital city determined state intensive investments (folk words of wisdom: "Moscow was constructed through the centuries, Petersburg – through millions!") and ambitious attitude to the city architectural appearance;

- availability of tough building regulation system covered the aspects of architectural style as well;

- limited period of time for main building development allowed to arrange its architectural properties within prevailing style, i.e. classicism;

Table 1
Comparison of factors determined building development character of Saint Petersburg and of European cities

No.	Feature	Cities		
		Saint Petersburg 18th–early 19th centuries	Traditional "European" city	"Ideal" city fortress
Organizational and urban planning aspects	Status	capital city of the large state	city-state, country town	fortified locality
	Population	several hundred thousand of people	10 000 people maximum	10 000 people maximum
	Clear border line of the city building development	is absent	is desirable	is mandatory
	Time of formation	one century	several centuries	a decade
	Cost of land	at the beginning of development — minimum	at all stages of development - considerable	minimum
	Non-city (state) financing of building development	considerable	in exceptional cases	complete
	Availability of low-density building development of mansion type	is available	is absent	at the beginning of development — minimum
	Building regulation	centralized, overall	moderate	centralized, overall
	Partition wall principle	is complied with	is complied with	is complied with
	Building development along the streets in a "house-to-house frontispieces"	is complied with	is complied with	is complied with
	Administrative constraints related to building height (number of storeys)	is carried out	is possible	is carried out
	Involvement of qualified professionals into design and construction activities	on an ongoing basis	possibly local	on an ongoing basis
Quantity of civil buildings within the city building development	Number of administrative buildings	significant	insignificant	insignificant
	Number of sovereign's and noblemen palaces	significant	insignificant	insignificant
	Number of barracks and military objects	large	insignificant	significant
Residential ribbon buildings	Length of a ribbon area along the main frontispiece	18.4–21.6 m	3–9 m	
	Number of storeys	up to 5	up to 3–7	up to 3–4
	Number of residential units within a building	At the first stage – 1; by 1830 – dozens	1–4	
	Frontispiece presentability requirements	severe	moderate	moderate

- availability of a number of highly qualified professional craftsmen and architects in Petersburg;
- particular features of construction technology (brick-work walls, plastered frontispieces) allowed to restore buildings and change their frontispieces style at relatively low costs.

Summary

The analysis of historical approaches to development of residential buildings' construction in Saint Petersburg discovered solutions for some of up-to-date housing problems. It confirms the importance of concept design stage for building image, rather than architectural ideas:

- credit financing terms for small and large buildings;
- allocation of areas for building development.

The key to architectural issues can be found in administrative and financing field, transferring from macroeconomics to microeconomics level and switching from large residential communities to small projects, allocating relevant credit financing and small land areas.

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THE SEISMIC BEHAVIOUR OF STONE MASONRY GREEK ORTHODOX CHURCHES

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Abstract

The seismic behaviour of structural systems representing Greek Orthodox churches is examined. All these churches are made of stone masonry in various architectural forms. During the years such churches developed damage to their stone masonry structural elements due to the amplitude of the gravitational forces acting together with the seismic forces. In certain cases such damage was amplified due to the deformability of the foundation. The behaviour of structural systems representing Greek Orthodox churches was simulated through linear and non-linear numerical models. The numerical results together with assumed strength values or failure criteria were utilized to predict the behaviour of the various masonry parts in in-plane shear and flexure as well as out-of-plane flexure. The deformability of the foundation partly explains the appearance of structural damage as can be seen both from observations and the numerical predictions. A limit-state methodology is presented whereby the demands obtained from linear elastic numerical models combined with limit-state in-plane behaviour of unreinforced stone masonry walls in shear/flexure or diagonal tension can yield reasonably good predictions of observed behaviour. Furthermore, the possibilities offered by non-linear inelastic numerical analyses as alternative means for examining the performance of unreinforced stone masonry walls is also briefly presented. Towards this objective, non-linear inelastic numerical simulation results are presented that yield reasonably good agreement with the relevant measured behaviour of stone masonry wall specimens of prototype dimensions that were subjected to simultaneous vertical compression and horizontal cyclic seismic-type loading in the laboratory. The obtained results from these specimens were utilized to also validate an expert system based on this limit-state methodology. Again, the observed behaviour was predicted with reasonable accuracy in terms of bearing capacity and mode of failure by this expert system.

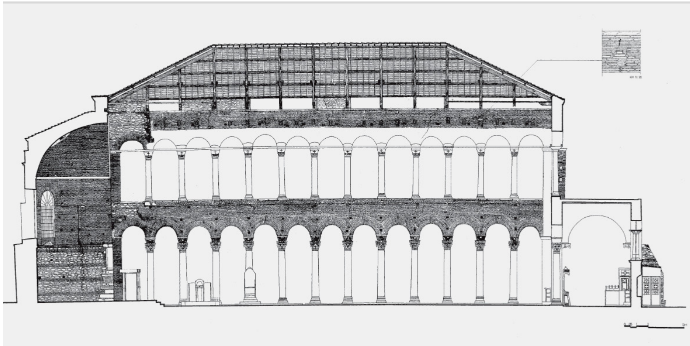
Keywords

Greek Orthodox Churches, Stone masonry, In-plane behaviour, Gravitational forces, Seismic actions, Foundation deformability, Limit-state, Non-linear behaviour

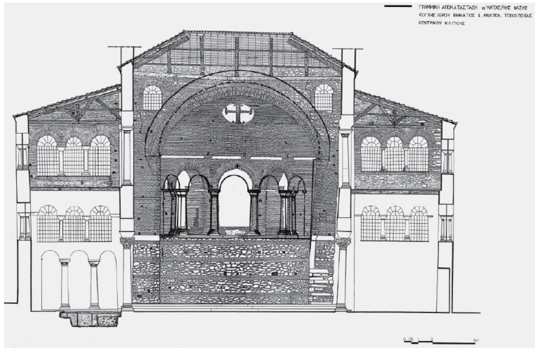
1. Introduction

Various parts of Greece have been subjected during the years to a number of damaging earthquakes (GEER, 2014; Papazachos et al., 2003; Manos, 2011). One of the most demanding tasks for counteracting the consequences of relatively intense seismic events is the effort to ensure the structural integrity of “old” churches that usually sustain considerable damage. There are two distinct structural systems that these “old” churches usually belong to. The “Basilica” architectural form is shown in figures 1a and 1b whereas the so called “crucifix form with a central dome” architectural form is shown in figures 2a and 2b.

The seismic performance of a considerable number of such “old” Christian church structural systems has been studied by Manos et al. (2008, 2009, 2010(1,2), 2011, 2013(1,2), 2012, 2014, 2015(1,2,3)) utilizing a variety of numerical simulations as presented in section 2. In some cases, the foundation was considered to be non-deformable; however, this is a gross approximation as in most cases these churches are founded on deformable soil. In some instances it is evident that the soil-foundation deformability amplified the structural damage (Manos et al. 2013(1), 2014, 2015(2,3)). Consequently, the numerical simulation tries to include in a simple way the effect of the soil-foundation deformability, as presented in section 3. The main objective of all these numerical simulations is to obtain realistic estimates of the demands ($\mathbf{S_d}$) that a particular load combination poses on various

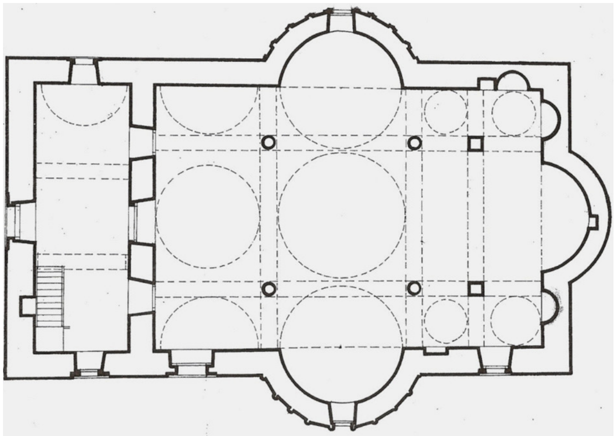


a) Longitudinal East-West section, View from North

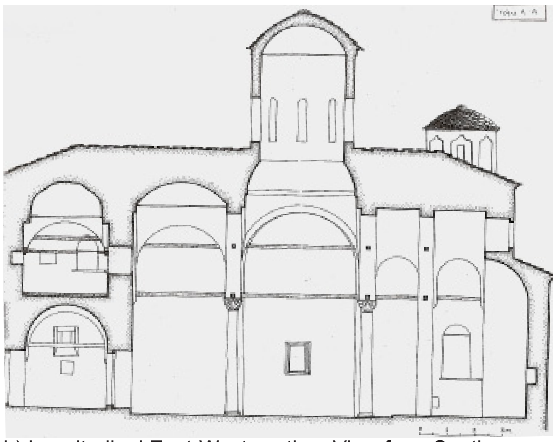


b) Transverse North-South section, View from East

Fig. 1. The Basilica church of Achiropoiitos, Thessaloniki, Greece



a) Plan



b) Longitudinal East-West section, View from South

Fig. 2. The church of Holy Trinity (Agia Triada) at Vithos – Voio – Kozani – Greece

types of stone masonry elements. This is needed in order to be able to monitor the ability of the various stone masonry structural elements to exhibit (or not) satisfactory performance, that is to satisfy (or not) a limit state condition representing either an in-plane shear / flexural mode of failure or an out-of-plane flexural mode of failure. Towards this objective it is necessary to obtain estimates of the relevant in-plane shear / flexural or out-of-plane flexural capacities ($\mathbf{R}_d$). This is briefly outlined in section 4. The most vulnerable masonry parts of these “old” churches are the masonry piers between door and window openings of the vertical peripheral walls. Such structural elements are presented in section 5 together with a particular experimental and numerical study that tries to predict the non-linear behaviour of such stone masonry piers. For this purpose, stone masonry piers of prototype dimensions built with prototype materials, which resemble the ones used in “old” churches, are subjected to a combination of vertical loads and horizontal seismic-type actions in the laboratory. The measured behaviour is then utilised to validate the employed numerical simulation by comparing the predicted with the observed behaviour.

2. Numerical simulation of the seismic performance

2.1. Forming the numerical model: A relatively simple way to numerically simulate the seismic performance of structural systems representing “old” churches is to employ shell finite elements for the stone masonry parts; in this way both in-plane as well as out-of-plane deformations and states of stress develop from transferring the imposed loads. This numerical simulation through shell finite elements follows the mid-plane of the actual stone masonry parts. Frame elements are also employed to numerically simulate the wooden trusses as well as the slender columns of the interior. In order to realistically approximate the stiffness of the masonry parts an appropriate value for the Young’s Modulus is assumed, in the range of 1500MPa or less. This is based on experimental measurements of the deformability of such stone masonry elements when they are subjected to a combination of vertical loads and horizontal seismic-type actions rather than pure axial compression tests.

2.2. Load Combinations: As already stated, the main objective of these numerical simulations is to obtain realistic estimates of the demands ($\mathbf{S}_d$) that a particular load combination imposes on various stone masonry parts that form the total structural system. In all these studies the employed load combination is one that includes the gravitational loads (**G**) together with the earthquake forces **Ex** and/or **Ey** along the longitudinal and transverse directions, respectively. The load combinations 0.9G±1.4Ex or 0.9G±1.4Ey can be used instead of the load combinations G±Ex±0.3Ey or G±0.3Ex±Ey, specified by the seismic codes (Eurocode 8; Manos, 1994; Provisions of Greek Seismic Code 2000; Provisions of Greek Seismic Code, 2003); this is an indirect but simple way to combine the two horizontal components of the seismic action, bearing in mind that the stone masonry walls are checked separately for the in-plane and out-of-plane state of stress (Gulkan at al., 1990). Furthermore, from extensive studies it was demonstrated that the load combinations 0.9G±1.4Ex or 0.9G±1.4Ey lead to more conservative (relatively larger) demands than the load combinations G±Ex±0.3Ey or G±0.3Ex±Ey. In addition, because gravity forces usually lead to compressive state of stress normal to the bed-joints in critical regions of vertical stone masonry walls and thus increase their corresponding shear strength, a reduction of the gravity forces is introduced (0.9 **G**) in order to lead to conservative (relatively smaller) shear strength estimates.

2.3. Linear-elastic numerical analyses: The numerical model simulating the structural system of the "old" stone masonry churches is subjected first to the gravitational forces and then to the earthquake actions. The latter can be applied in various different ways depending on the choice of method of analyses. Initially, a linear-elastic behaviour is assumed for the various stone masonry parts and their connections. A direct consequence is that the results from the earthquake actions can be super-imposed on the results from the gravitational forces. Despite the fact that this assumption is a simplification of the behaviour of stone masonry structures under seismic loading, it has the great advantage of simplicity that leads to numerical solutions for seismic type loads within realistic limits of computer memory and time. Moreover, it also has the advantage of producing various deformation patterns and corresponding state-of-stress for all structural elements that can be easily understood when linked with the expected transfer of loads even if they are a product of this basic simplification of linear-elastic behaviour. One popular method of analysis for earthquake actions is the *dynamic spectral method* whereby the amplitude of the seismic forces is based on accepted design or site-defined spectral curves. A first check of the numerical analyses results involves a thorough study of the resulting dynamic eigen-modes of vibration with their corresponding eigen-frequencies and participation mass-ratios (Manos et al., 2008, 2009, 2010(1,2), 2011, 2013(1,2), 2012, 2014, 2015(1,2,3)). Through a screening process of these dynamic characteristics one has the ability to exercise his structural engineering experience by removing eigen-modes that are the product of this linear-elastic behaviour combined with the rigid connections of the various stone masonry elements assumption, thus not including in the dynamic analysis modes of dynamic response that are obviously unrealistic for stone-masonry structures but still retaining the significance of the translational modes for the structure as a whole or the fundamental in-plane and out-of-plane modes of vibration of the main stone masonry parts. Should the remaining eigen-modes not mobilize a significant part of the total structural mass (e.g. larger than 90%) an appropriate amplification factor is introduced to compensate for this. In defining the earthquake actions through a spectral curve for the longitudinal **Ex** and the transverse **Ey** directions it is necessary to also define a value for the behaviour factor ($\boldsymbol{q}$, (Eurocode 8)) the value for the importance factor ($\boldsymbol{\gamma_I}$, (Eurocode 8)) and the soil conditions for the structure at hand. For unreinforced masonry a value in the range of 1.5 to 2.0 is commonly employed for the behaviour factor ($\boldsymbol{q}$, (Eurocode 8; Provisions of Greek Seismic Code 2000; Provisions of Greek Seismic Code, 2003)). The value for importance factor can be derived from the fact that for common occupancy contemporary residential buildings the value of this factor is 1.0, based on 10% probability of exceeding the design ground acceleration in a period of 50 years (Eurocode 8; Manos et al., 1994; Provisions of Greek Seismic Code 2000; Provisions of Greek Seismic Code, 2003). Prototype earthquake acceleration ground motions and their corresponding response spectral curves can be employed alternatively, provided that this choice is based on sufficient justification for a particular "old" stone masonry church. In this case, a constant ductility response spectral curve can also be employed with a ductility value in the range of the values mentioned before for the behaviour factor ($\boldsymbol{q}$ (Eurocode 8; Manos et al., 1994; Provisions of Greek Seismic Code 2000; Provisions of Greek Seismic Code, 2003)).

The dynamic spectral method of analysis has the advantage of producing results that are based on the dynamic nature of the earthquake actions. However, because these results are produced from a statistical combination of the contributions of the eigen-modes, the deformation patterns and the maximum values of the stresses in the critical locations cannot be easily selected and studied. An alternative method of analysis is an *"equivalent linear pushover"* method whereby all the structural masses are subjected to a constant value of horizontal acceleration. The amplitude of this acceleration is defined so that the resulting base shear value in either the longitudinal **Ex** or the transverse **Ey** direction is equal to the corresponding base shear value that resulted from the spectral dynamic analysis. Finally, the earthquake actions can be alternatively applied through a *dynamic time history analysis*. In this case, a horizontal ground acceleration time history can be introduced in either the longitudinal **Ex** or/and in the transverse **Ey** direction. Moreover, this can be done employing the recordings of a prototype earthquake event provided that this choice is based on sufficient justification for a particular "old" stone masonry church. The results that are obtained through such an analysis are usually of a very large volume that need particular experience and efficient post-analysis tools in order to extract the most significant information. The previously described linear-elastic methods of analyses in order to determine the demands ($\mathbf{S}_d$) at various stone masonry structural elements can be combined next with the definition of the corresponding capacities keeping in mind certain realistic limit-state scenarios, as will be detailed in section 4.

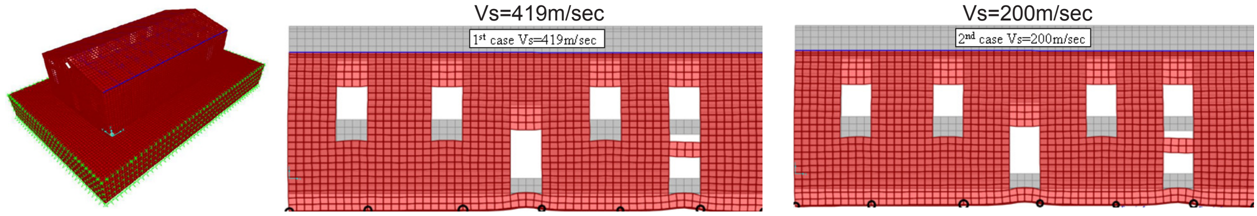
2.4. Non-linear numerical analyses: The same finite element representation that was used for the linear elastic analyses of the stone-masonry churches can also be employed for the non-linear numerical analyses. However, this depends upon the non-linear analyses options that are incorporated in the software to be utilized (Ramalho et al., 2008). Thus, Manos et al. (2008) employed shell finite elements as described in 2.1. Then the LUSAS software package was used to perform non-linear analyses of the church of Agia Triada for seismic type loading, employing three alternative modified Von-Misses failure envelopes in a non-linear, step-by-step incremental analysis (see also section 5). Betti and Vignoli (Betti & Vignoli, 2008) employed brick-type finite elements together with the software package ANSYS to investigate the seismic performance of the Farneta Abbey, a Basilica-type structure, employing a Drucker–Prager perfectly plastic criterion. The most important step in this type of non-linear analyses is to select a non-linear material model that can simulate in a realistic manner the non-linear behaviour of the stone-masonry structural elements under combined gravitational and seismic type loading. Due to the large computational requirements these non-linear analyses are of a static "push-over" nature whereby the structure is subjected initially to the gravitational forces and then to a realistic predetermined form of deformation pattern that gradually increases in a step-wise manner (Manos, 2008). This type of non-linear analyses can be further refined by assuming a number of pre-determined failure modes and then selecting as most probable failure mode the one that requires the lowest level of force for its development.

3. Soil-Foundation Deformability

The foundation of "old" masonry churches is considered to be formed by a peripheral masonry strip that is an extension of the masonry walls in the sub-soil at a certain depth. In order to study numerically the soil-foundation deformability the following process is utilized (Manos et al., 2013(1), 2014, 2015(2,3)). First, a numerical model of the structure and the masonry foundation strip is formed, modelled with shell elements whereas the soil volume underneath is modelled with solid "brick" finite elements with linear elastic properties thus representing the volume of soil under the church extending to a certain depth below the foundation – soil interface. When reliable geotechnical data are not readily available the investigation of the soil-foundation deformability is attempted in a parametric way. Initially, a shear wave velocity value equal to 420m/sec is assumed for example and that value together with a soil density equal to 20KN/m3 leads to a shear modulus equal to 354MPa and a Young's modulus value for the soil equal to 1000MPa, which represents a rather hard soil. Alternatively, a shear wave velocity value equal to 200m/sec leads to a Young's modulus value for the soil equal to 230MPa, which represents a medium stiffness soil (Manos, 2015(2)). A numerical simulation that includes the superstructure, the foundation masonry strip and the soil layers, as shown in figure 3a, is then subjected to the dead weight (**G**). The resulting vertical deformation patterns for the 1st (Vs=419m/sec) and 2nd (Vs=200m/sec) case of soil deformability, are obtained all along the foundation-soil interface (figures 3b and 3c).

Next, in order to simplify the final numerical model of the examined "old" stone masonry churches including the soil-foundation deformability, a simple alternative numerical model of the foundation masonry strip – soil interface is formed. This model retains all the aspects of the superstructure and the foundation masonry strip. However, this time the effect of the deformability of the soil layers is represented by two-node 3-D link elements with such an axial stiffness that when the same dead weight is applied as was done before, the resulting vertical deformation pattern at the bottom surface of the foundation – soil interface is as close as possible to the values obtained before with the full presence of the soil layers (figure 3a, b, c). The following summarise the most significant effects of soil-foundation deformability on their seismic behaviour:

- A lengthening of the eigen-period values and a mobilization of larger modal mass ratios when the deformability of the soil-foundation is included than when the soil is presumed non-deformable. This in turn usually leads to an increase in the values of the demands ($\mathbf{S}_d$).



a) b) Vertical deflections the soil-foundation interface c) Vertical deflections the soil-foundation interface

Fig. 3. 3-D model of the superstructure, the soil-foundation interface and the soil layers

- This increase in the demands (**S_d**) becomes more pronounced when the structural system is non-symmetric or the deformability of the foundation is non-uniform in plan. This in turn result in significant stress concentration for the structural elements, even only for the gravitational forces, that can lead to spectacular structural failure, as was the case of the church of “The Assumption of the Virgin Mary” at Dilofo-Kozani-Greece (figure 4, (Manos et al., 2014, 2015(1,3)).

4. Expected seismic performance on the basis of the linear analyses demands

The demands (**S_d**) for the various stone masonry structural elements, as obtained from the linear-elastic numerical models are utilized in the following two ways: First, the results can be studied in terms of normal and shear stresses $\boldsymbol{\sigma}_{11}$, $\boldsymbol{\sigma}_{22}$, $\boldsymbol{\tau}_{12}$. These results are usually presented in terms of coloured diagrams; by studying them, areas of stress concentration can be identified that can be compared with damaged areas as a first qualitative check, as seen in figures 4 and 5.

Next, certain commonly used masonry failure criteria can be adopted for either in-plane tension/compression or shear/compression or out-of-plane tension. All the masonry parts are examined in terms of the obtained in-plane and out-of-plane stress demands (**S_d**) against the corresponding normal and shear stress capacities (**R_d**). In defining these capacities, use can be made of existing guidelines for the design of contemporary masonry structures (e.g. Eurocode 6) or experimental data which can be substantiated as bearing some resemblance to the stone masonry structural elements at hand (Eurocode 6). Table 1 lists the assumed mechanical characteristics for the stone masonry of the church of Holy Trinity (Agia Triada) at Vithos – Voio – Kozani – Greece (Manos et al., 2014, 2015 (1,3)). Moreover, a Mohr-Coulomb failure envelope was adopted for the in-plane shear limit state of the stone masonry, when a $\boldsymbol{\sigma}_n$ normal stress is acting simultaneously with a shear stress demand. This is defined through the equation 1 (Eurocode 6).

$$\mathbf{f}_{vk} = \mathbf{f}_{vko} + 0.4\ \boldsymbol{\sigma}_n \qquad \text{(Eq. 1)}$$

where: $\mathbf{f}_{vko}$ is the shear strength of the stone masonry when the normal stress $\boldsymbol{\sigma}_n$ is zero; $\mathbf{f}_{vko}$ was assumed to be equal to 0.19 N/mm2.

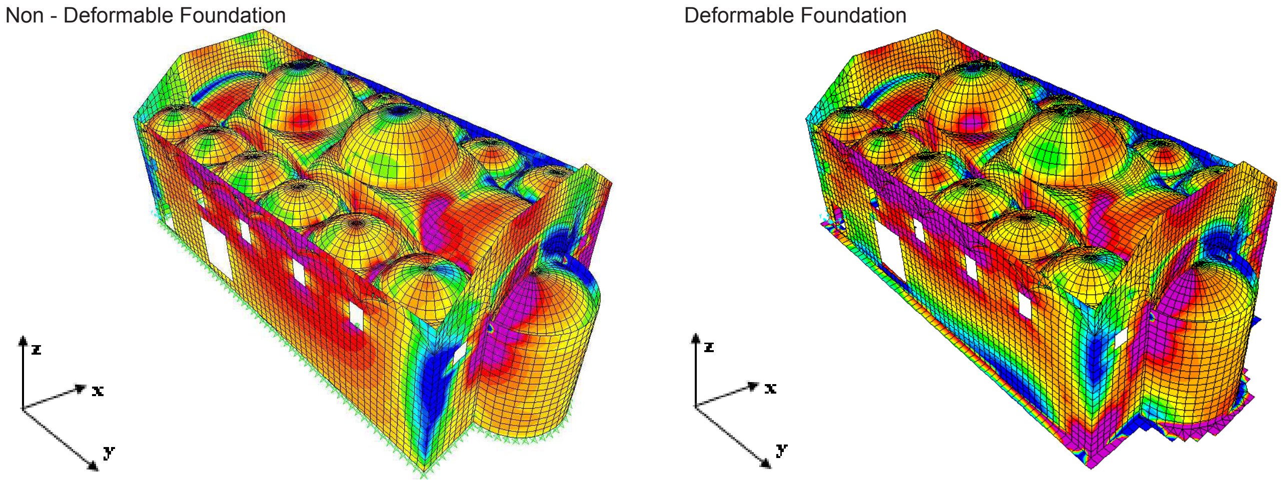
By comparing the capacities (**R_d**) with the demands (**S_d**) the following performance criterion is checked:

$$\mathbf{Ratio = R_i = R_d / S_d > 1} \qquad \text{(Eq. 2)}$$

a satisfactory performance is signified. The opposite is true when **Ratio = R_i = R_d / S_d < 1** whereby a non-satisfactory performance is signified. In order to further detail the use of the above performance criterion the following capacity over demand ratios are defined: **R_σ** is the ratio of the in-plane tensile

Damage to the South-East corner Damage to the North longitudinal wall

Fig. 4. The stone masonry church of “The Assumption of the Virgin Mary” at Dilofo-Kozani-Greece



a) σ_{11} bottom face, $_{max}\sigma$= 0.51 MPa **b)** σ_{11} bottom face $_{max}\sigma$= 0.64 MPa

Fig. 5. Load Combination 0.9G+1.4(+Ex). The stone masonry church of "The Assumption of the Virgin Mary" at Dilofo-Kozani-Greece

strength value ($\mathbf{f_{xk1}}$ or $\mathbf{f_{xk2}}$) over the corresponding normal tensile stress demands σ_{22} or σ_{11}, respectively. $\mathbf{R_\tau}$ is the ratio of the in-plane shear strength value ($\mathbf{f_{vk}}$, equation 1) over the corresponding shear stress demand τ_{12}. Finally, $\mathbf{R_M}$ is the ratio of the out-of-plane tensile strength value ($\mathbf{f_{xk1}}$ or $\mathbf{f_{xk2}}$) over the corresponding tensile stress demand σ_{22} or σ_{11}, respectively. As already stated, ratio values smaller than one ($\mathbf{R_\sigma}$, $\mathbf{R_\tau}$, $\mathbf{R_M}$ <1) predict that the corresponding limit state condition was exceeded thus signifying prediction of structural damage. It was shown by numerous studies that the methodology described here correlates quite well with observed damage or with predictions of structural damage through the non-linear approach (section 5).

Taking this rationale one step further the following can also be stated as being valid. By obtaining in the same location of a structure (a section of a structural element) ratio values $\mathbf{R_\sigma}$, $\mathbf{R_\tau}$, $\mathbf{R_M}$ with all or some of them being smaller than 1 ($\mathbf{R_\sigma}$, $\mathbf{R_\tau}$, $\mathbf{R_M}$ <1) the damage scenario that can be predicted as prevailing is the one that corresponds to the limit-state with the smallest ratio value. Figures 6a and 6b depict the $\mathbf{R_\tau}$ ratio values of the in-plane shear strength / shear demand for the load combination 0.9G+1.4Ey for the internal transverse wall, which separates the main church from the women's quarters situated at the west portion of the church of Holy Trinity (Agia Triada) at Vithos – Voio – Kozani – Greece (Manos et al. 2014, 2015(1,3)). Shear damage is predicted by these numerical analysis results as can be seen from the $\mathbf{R_\tau}$ ratio values that are well below one ($\mathbf{R_\tau}$ < 1) in many locations. Moreover, when comparing the ratio values between figures 6a and 6b for the same locations it can be seen that the deformability of the foundation leads this ratio to obtain even smaller values than for the case of the non-deformable foundation. This demonstrates the detrimental effect of the flexibility of the foundation for this church, as described in section 2. The structural element capacities can be found either directly from the strength values, when one performs an evaluation of the performance of a damaged structure or with the introduction of the appropriate safety factors for stone masonry (γ) when one performs an evaluation of the performance of a structure for design purposes (Eurocode 6). Initially, the $\mathbf{R_\sigma}$, $\mathbf{R_\tau}$, $\mathbf{R_M}$ ratio values are found for the maximum values of stress demands which are obtained through the numerical simulations. However, finding ratio values ($\mathbf{R_\sigma}$, $\mathbf{R_\tau}$, $\mathbf{R_M}$) smaller than 1 locally, through such maximum stress demand values, does not imply that the limit-state capacity of a structural element is exceeded. An alternative approach has been proposed by Manos et al (2015(1)) that is based on making this capacity-over-demand checks in the level of a horizontal cross-section for vertical stone masonry structural elements. This can be extended for cross-sections of different orientations and for stone masonry structural elements other than vertical. However, as the most significant structural elements for the safe earthquake performance of stone masonry Greek Orthodox churches are the vertical stone masonry walls and piers this approach is further

Table 1
Assumed Mechanical Characteristics of the Stone Masonry

	Young's Modulus (N/mm²)	Poisson's Ratio	Compressive Strength (N/mm²)	Tensile Strength normal /parallel bed-joint (N/mm²)	Shear strength $\mathbf{f_{vko}}$ (N/mm²)
Limit values	1500	0.2	3.8	0.250 / 0.800	0.19

Ratio $\mathbf{R}_{\tau}$ values of in-plane shear strength / shear demand.
The church of Holy Trinity (Agia Triada) at Vithos – Voio – Kozani – Greece

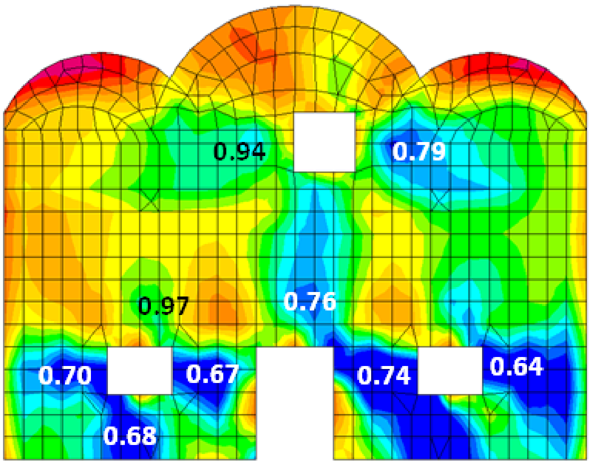


Fig. 6a. Non-Deformable Foundation
Internal Transverse Wall, 0.9G+1.4Ey

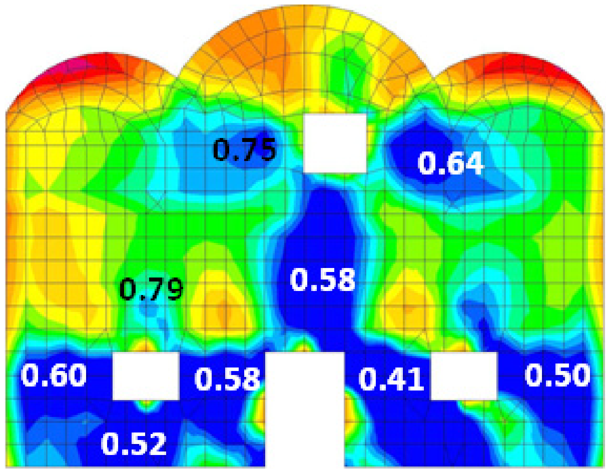


Fig. 6b. Deformable Foundation
Internal Transverse Wall, 0.9G+1.4Ey

detailed here for the in-plane limit-state behaviour of vertical masonry walls and piers. For this purpose, the results of the linear-elastic numerical analysis, in terms of stress resultants (***N, M, H,*** figure 7), are then utilized.

4.1. In-plane bearing capacity against sliding together with the flexural capacity

For this purpose use could be made of the provisions of Eurocode 6. For the masonry pier which is studied here the shear strength of the masonry (f_{vk}) is the minimum value of the following:

$$f_{vk} = f_{vk0} + 0.4\,\sigma_n \text{ (Eq. 1 stated before)}$$

where σ_n is the value of the average normal stress, f_{vk0} the shear strength of the masonry for zero normal stress that is specified by the provisions of Eurocode 6.

$$f_{vk} < 0.065\, f_b \qquad \text{(Eq. 3)}$$

where f_b is the compressive strength of the masonry unit.

$$f_{vk} < f_{vklim} \qquad \text{(Eq. 4)}$$

where f_{vklim} is the upper shear strength of the masonry, as specified by the national appendix of each member state.

The distribution of axial stress (σ_n) normal to a bed joint with thickness equal to the pier thickness that develops at this horizontal section is assumed to be one of the four simple cases depicted in figures 8a to 8d, which are incorporated in many design provisions. In order to obtain the shear capacity against sliding one should properly choose which of these four cases of normal stress distribution develops based on the geometry, the stress resultants (N_y, Q_y, M_y) and the masonry compressive and tensile strength values, f_{kd} and f_{xk1d} respectively. These strength values as well as the checks being performed are based on the provisions of Eurocode 6; however, provisions from other codes can be easily incorporated. Together with the normal stress distribution, the length of the compressive zone (l_c) is also calculated as well as the value of the average normal stress (σ_n), which is assumed to act in this compressive zone as depicted in figures 8a to 8d. Use is made of both the compressive zone length and the average normal stress value for calculating next the masonry shear capacity against sliding (see equations 1, 3 and 4).

4.2. In-plane bearing capacity of a masonry pier against diagonal tension

This is done based on the following formula (Eq. 5) given by Bernardini et al (1980) by Turnsek and Cacovic (1971) and by Tomazevic (2009). It is assumed that the tensile strength of the masonry $\sigma_t = f_{xk1}$ depends on the maximum average shear stress of a horizontal section of the masonry pier and on the average compressive stress $\sigma_d = N / A$ that develops at the same location where A is the area of this section and N the compressive load.

$$\sigma_f = f_{xk1} = \sqrt{(\sigma_d / 2)^2 + (b\tau_{\max})^2} - \sigma_d / 2 \qquad \text{(Eq. 5)}$$

Where b represents the shear stress distribution factor, which is related to the stress distribution on the section and the slenderness ratio of the wall. It can be assumed that $b = h / l$, where h is the height and l is the length of the pier. In this case $b = 1.5$ is the upper limit value and $b = 1$ is the lower limit value. From the above relationship the value of can be obtained based on the values of $\sigma_t = f_{xk1}$ and σ_d:

$$\tau_{\max} = \frac{f_{xk1}}{b}\sqrt{f_{xk1} + \sigma_d} \qquad \text{(Eq. 6)}$$

The formulas and procedures described in 4.1. and 4.2 are incorporated in an expert system that can predict the bearing capacity of a given pier and the expected mode of failure of a given stone masonry pier (Manos et al., 2015(1)). The validity of this expert

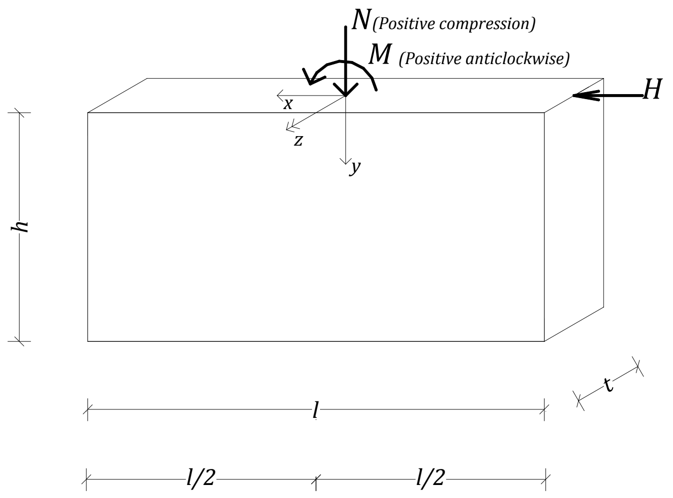
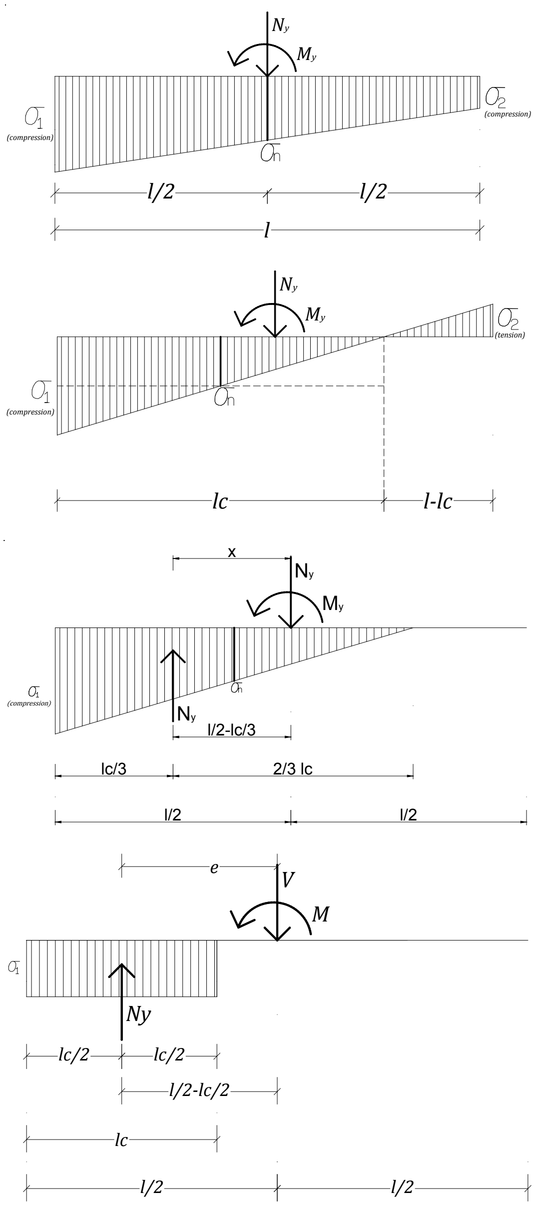


Fig. 7. Single stone-masonry pier being stressed at its upper boundary

Based on these values, limit-state patterns of axial stress (σ_n), normal to a bed joint for a given horizontal cross-section are found, as depicted in figures 8a to 8d, which are realistic for stone masonry elements. The objective here is to predict whether the transfer of these stress resultants can be done successfully or not, depending on the shear and flexural capacity of this structural element, as defined at a cross section located at a distance equal to ***y*** from the upper boundary N_y, Q_y, M_y . This is briefly outlined in 4.1 and 4.2. for the in-plane behaviour.



a. ***Case 1***: Axial stress distribution is compressive along all the length of the examined mortar bed joint of the pier:

$\sigma_1 \geq 0$ and $\sigma_2 \geq 0$

$\boldsymbol{\sigma_1}$ < than the masonry compressive strength f_{kd}

b. ***Case 2*** **:** $\boldsymbol{\sigma_2}$ < than the tensile limit stress f_{xk1d} ; consequently, the tensile zone is assumed to be active:

$\sigma_1 > 0$; $\sigma_2 < 0$; $|\sigma_1| \leq f_{xk1d}$
and $\sigma_1 < f_{kd}$

c. ***Case 3*** **:** There is tension at the right fiber of the cross section, larger than the tensile limit stress f_{xk1d} ; consequently, the tensile zone is assumed to be inactive:

$\sigma_1 > 0$; $\sigma_2 < 0$; $|\sigma_2| > f_{xk1d}$
and $\sigma_1 < f_{kd}$

d. ***Case 4*** **:** The tensile zone remains inactive but the compressive zone becomes narrower than before.

$\sigma_1 > 0$; $\sigma_2 < 0$; $|\sigma_2| > f_{xk1d}$

$$\text{and } e = \frac{M_y}{N_y l} \geq 0,45$$

(Towards flexural failure):

Fig. 8. Normal to the bed-joint axial stress distribution patterns

system was confirmed by comparing its predictions with the observed laboratory performance, in terms of capacity and mode of failure, of a considerable number of masonry piers. A new series of such comparisons are currently under way, as briefly described in section 5.

5. Expected seismic performance on the basis of non-linear numerical simulations

As already mentioned, non-linear inelastic "push-over" types of numerical analyses were also performed whereby stone masonry structures and specimens are subjected first to the permanent vertical loads and then the seismic horizontal forces (Manos et al., 2008, 2015(1,2)), Vintzileou, 2008). In what follows, the numerical non-linear step-by-step approach will be applied to simple stone masonry specimens that have been subjected in the laboratory to certain load combinations that resulted in a state of in-plane stress distribution resembling the state of stress of vertical stone masonry piers subjected to stress resultants from the combined gravitational and seismic forces.

5.1. Numerical simulation of the behaviour of a "short" pier stone masonry specimen

A number of "short" stone masonry specimens with dimensions 500mm by 500mm in plan and 300m in height, were built at the laboratory of Strength of Materials and Structures of Aristotle University using lime mortar and natural stones (Manos et al., 2015(2,3)). The lime mortar had such a composition as to be representative of old relatively weak mortars commonly used in the past. A series of such samples were tested accompanied by constant vertical compression with variable horizontal shear load at the top having fixed the base of each specimen against sliding, as shown in figure 9. This set-up was designed in an effort to obtain an estimate of the shear strength at a limit state which represents the failure of the mortar bed-joint as depicted in figure 10.

Based on these experimental results, the Mohr-Coulomb limit state criterion (equation 1) can be approximately quantified. The numerical model of this tested specimen is depicted in figure 11, being supported and loaded in the same way as the "short" pier specimen during testing. The non-linear behavior was simulated utilizing the capabilities of the commercial software ABAQUS (Hibbit et al., 2010).

The obtained numerical against the measured behaviour of this specimen, in terms of shear stress (τ) versus shear strain (γ), is plotted in figure 12. As can be seen, the agreement between the non-linear numerical predictions and the observed behaviour, in terms of τ-γ plots, is reasonably good (Manos et al., 2015(1)). The numerical predictions of the distribution of the plastic strains as well as the tensile principal stresses are plotted in figures 13a and 13b, respectively. When comparing the distribution patterns depicted in figures 13a and 13b with the observed failure mode during testing, depicted in figure 10, reasonably good agreement can again be seen.

5.2. Numerical simulation of a "square" stone masonry pier specimen with simultaneous com. pression and horizontal load

A stone masonry almost "square" pier is examined next having a length equal to l=1500 mm, a height equal to h=1400 mm and a thickness equal to t= 500 mm, thus a length over height ratio (l/h) equal to.1.071 (quite close to 1.0). This square pier specimen was built at the laboratory of Strength of Materials and Structures of Aristotle University with the same stones and mortar that were used to construct the short pier specimen presented before. A number of square and short pier specimens using mortars of different compositions were also constructed and tested; however, space limitations prohibit the presentation of their performance. The square pier specimens were subjected to a uniform compression equal to approximately 0.12Mpa at their upper boundary. The horizontal load, as shown in figures 14a and 14b, results from the imposed horizontal displacement at the upper boundary which reaches a value equal to 10mm at the final stages.

Figure 14a depicts the experimental set up showing the resulting horizontal load at the top of the stone masonry pier whereas figure 14b is the corresponding numerical simulation. The vertical load is also shown as applied at two locations at the top of a steel beam resting on top of the specimen. This steel beam is capable of sliding horizontally through a sliding interface between this steel beam and the stone masonry specimen. The sliding surfaces have a coefficient of friction less than 2%, thus providing very little horizontal resistance during the application of the horizontal load to the specimen. Moreover, a system of load cells and low-stiffness springs is provided at each location of vertical load application. Using these low-stiffness springs the variation of the vertical load that results from the vertical deformations of the specimen during the combined horizontal and vertical load sequence is minimized. In any case, the amplitude of the vertical load and its variation in each one of these two locations is recorded continuously by the two load cells. The locations where these vertical loads are applied together with the locations of the sliders (whereby these vertical loads are transferred to the specimen) are chosen in such a way as to have an almost uniform distribution of axial stresses normal to the bed-joints of the stone masonry specimen. The vertical loads, which were applied at this stone masonry pier at the initiation of testing, were equal to 49KN at each vertical loading location. The special gravity of the masonry is considered equal

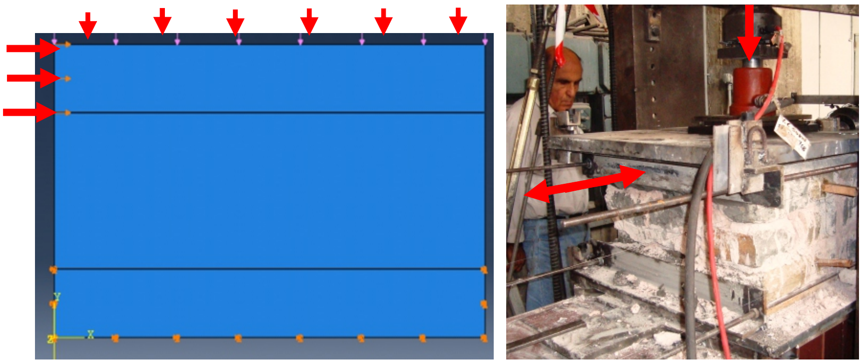


Fig. 9. Compression and shear loading applied at the short stone masonry piers utilizing hydraulic actuators

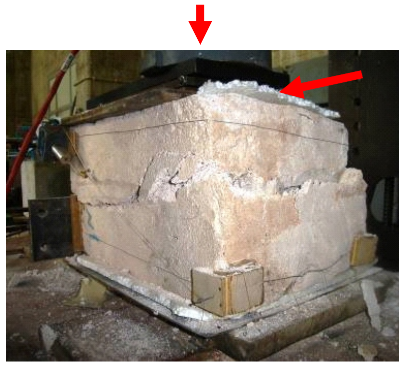


Fig. 10. Observed damage patterns of the short pier

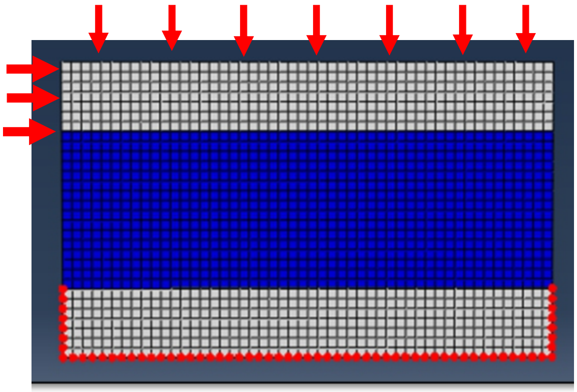


Fig. 11. Numerical simulation of the short pier tested specimen

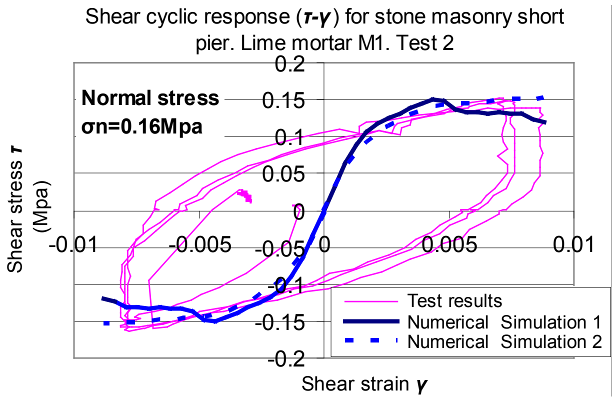


Fig. 12. Short pier observed and numerically predicted shear stress versus shear strain

to 22.0 KN/m3. This "square" pier is assumed to be constructed by relatively weak masonry; therefore, the Young's modulus for the horizontal load is presumed to be equal to 200 MPa. Both the average normal and shear stress at a horizontal cross-section of the specimen, corresponding to an ideal bed-joint was obtained by dividing the measured applied total horizontal and vertical load by the net cross-section (approximately 75% of the gross cross-section due to the building detail). Figures 15a and 15b depict the variation of the shear stress versus the shear strain (blue line). The horizontal load was applied in a cyclic low-frequency seismic-type manner.

In figures 15a and 15b the variation of the compressive axial stress normal to the bed-joint is also plotted (pink line using the far right axis with the negative values). As can be seen in figure 15a, the maximum shear stress was equal to 0.09MPa for Test 6. For this test the corresponding average axial stress value was equal to -0.16MPa exhibiting a relatively small variation around this value during cyclic loading. On the contrary, the variation of the axial stress during cyclic test 7 reached a value equal to -0.26MPa that corresponded to a shear stress value equal to 0.15MPa. At the reverse cycle, the axial stress value was equal to -0.12MPa when

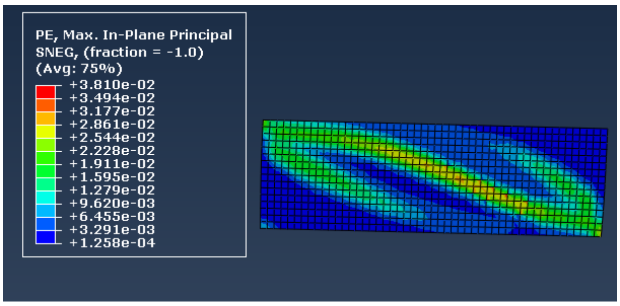


Fig. 13a. Distribution of predicted plastic strains for the short pier

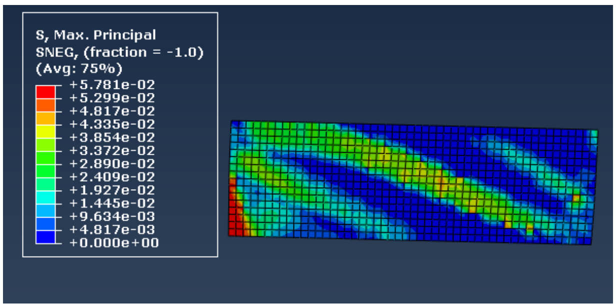


Fig. 13b. Distribution of predicted tensile principal stresses for the short pier.

the corresponding shear stress value was equal to 0.09MPa, as was the case for test 6.

The measured variation of horizontal load (H) versus horizontal displacement (δ) at the top of the specimen for tests 6 and 7 is depicted in figure 16. In the same figure the numerical predictions are also plotted as resulting from the non-linear numerical simulation. As can be seen, the comparison between predicted and observed H-δ response is reasonably good for Test 6. For test 7 there is a distinct difference between observed and predicted response when the horizontal load attains negative values. This is attributed to the variation of the vertical load at this stage of the experiment that reached much higher absolute values, as already mentioned before on the basis of figure 15b. This was not accounted for up to now in the numerical simulation that kept constant throughout the vertical load, as already described and equal to 49KN at each location. Additional numerical simulations are required to account for the observed vertical load variation. The numerically predicted horizontal deformation pattern resulting from the non-linear simulation is depicted in figure 17; this deformation pattern is for the maximum horizontal load equal to 57.71KN, which was reached for horizontal displacement at the top of the pier equal to δ=1.85mm.

Figure 18 depicts the damage patterns in the form of wide diagonal cracks that were formed in this stone masonry wall specimen (S1M1) and were photographed after the end of Test 7. Figure 19 depicts the distribution of the plastic strains that developed in the non-linear numerical simulation when the horizontal load dropped for the first time from its maximum value to 49.96KN and horizontal displacement at the top of the pier reached a value equal to δ=2.4mm. As can be seen, the formation of the observed diagonal crack pattern is to a certain extend predicted by the non-linear numerical simulation. Next, the expert system, which was briefly

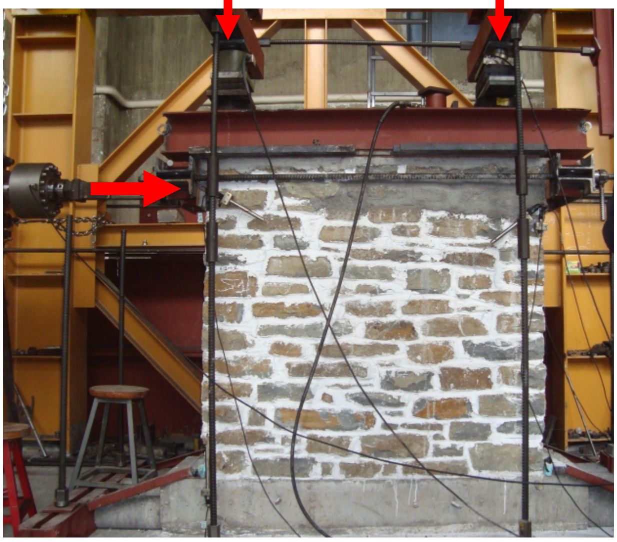


Fig. 14a. Stone masonry wall specimen S1M1. Loading arrangement being utilized

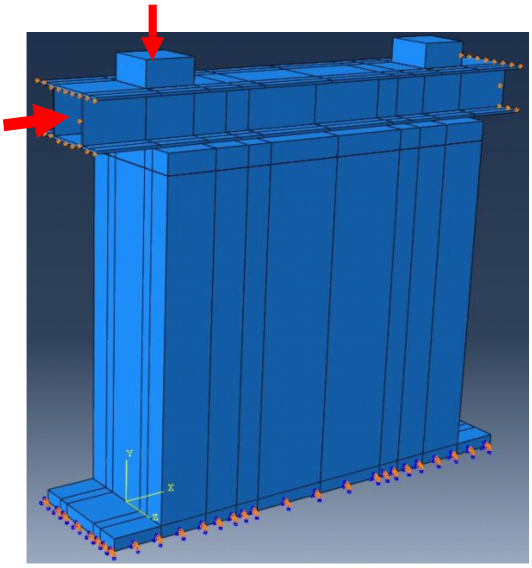


Fig. 14b. Numerical simulation of the loading arrangement for wall specimen S1M1

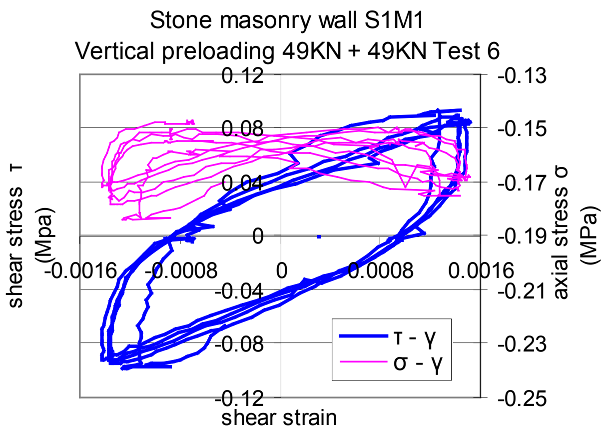


Fig. 15a. Variation of the measured shear / axial stress versus the shear strain. Test 6

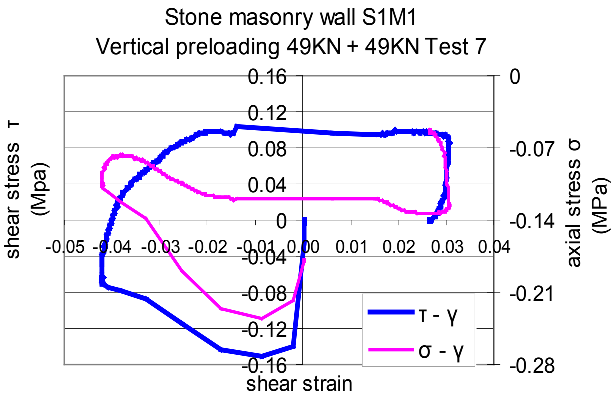


Fig. 15a. Variation of the measured shear / axial stress versus the shear strain. Test 7

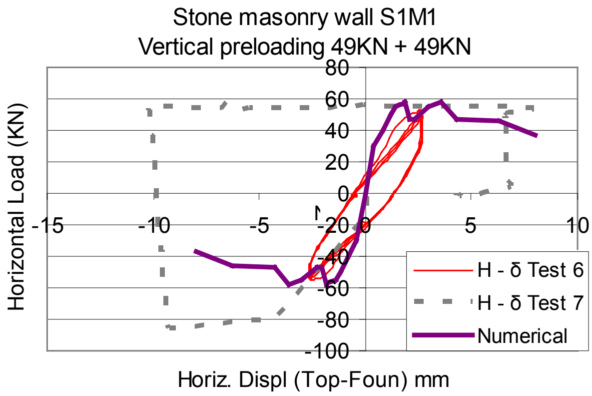


Fig. 16. Variation of the horizontal load versus the horizontal displacement at the top. Tests 6, 7 together with numerical predictions

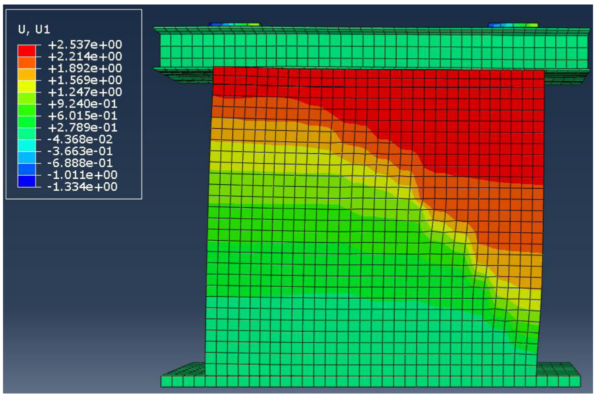


Fig. 17. Numerically predicted horizontal deformations for the stone masonry specimen. Maximum value at top 1.85mm

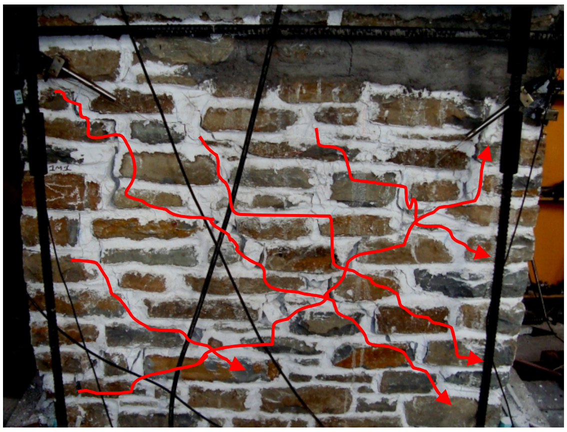


Fig. 18. Diagonal cracks that developed in the stone masonry specimen S1M1

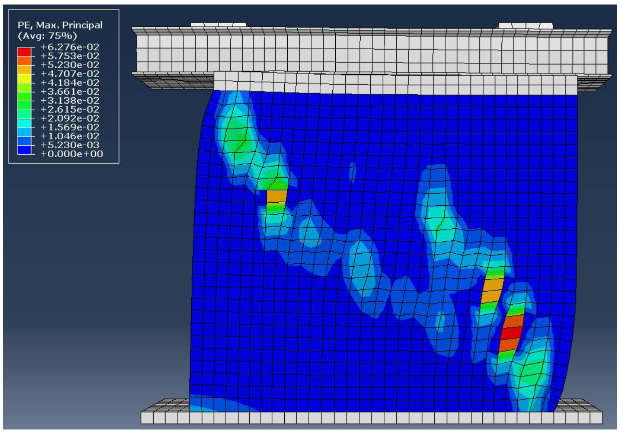


Fig. 19. Distribution of numerically predicted plastic strains for H = 43,41 KNt Ux = 6,58 mm

described in section 4.2, was employed for this tested stone masonry specimen (S1M1) with the following particular geometric and mechanical characteristics that are relevant to this specimen: Length=1.5m, Height=1.2m, Thickness=0.4m (to account for 25% reduction), Compressive strength $\mathbf{f_k}$=4.35MPa, Initial shear strength $\mathbf{f_{vko}}$=0.06MPa, Tensile strength normal to the bed-joint $\mathbf{f_{xk1}}$=0.05MPa, Initial compressive force ***N***=98KN, Initial bending moment ***M***=0. The subsequent performance check was for a cross-section at a distance of ***y***=0.7m from the top (see section 4 and figures 7, 8) the maximum horizontal load capacity, predicted by this expert system, is ***H***= 64.45KN and the predicted mode of failure is diagonal tension (section 4.2). Both predictions by the expert system agree reasonably well with the observed performance (figures 16 and 18).

6. Conclusions

1. The seismic behaviour of stone masonry Greek Orthodox churches is examined in this paper. This study was based on long term observations of the seismic performance of this type of structures that developed structural damage when subjected to actual moderate to strong earthquake ground motions during the last 50 years.

2. A methodology is first described that was based on a linear numerical simulation and a dynamic spectral method of analysis to obtain the demands. The corresponding capacities were next obtained by considering in-plane and out-of-plane limit-states that are well established from experimental investigations and are included in current design procedures for contemporary masonry structures. The difficulty here was to reach with a good degree of approximation the relevant mechanical old stone-masonry strength values that are required in order to calculate these capacities. In-situ and laboratory tests can be mobilised for this purpose. Moreover, the significance of the soil-foundation deformability was discussed.

3. Good agreement was achieved when comparing the predicted with the observed performance by applying this linear limit-state methodology in a number of stone masonry Greek Orthodox churches whose seismic behaviour has been well documented in the past.

4. An expert system was developed that can be used to facilitate the results of the above methodology for vertical stone masonry structural elements. The basis of this expert system for

calculating the capacities for in-plane demands was briefly described.

5. Next, certain type of non-linear numerical simulation was examined towards reaching realistic predictions of the observed performance of stone masonry structural elements. For this purpose use was made of experimental set-ups employing relatively simple stone masonry specimens of prototype dimensions which were constructed with materials resembling old stone masonry. These specimens were subjected to in-plane combined compression, shear and flexure in such a way as to develop stress fields that are similar to the state of stress prototype stone masonry structural elements develop during combined gravitational forces and seismic actions. The obtained measured behaviour from these tests was then utilized to validate the employed non-linear numerical simulations. As was demonstrated from these comparisons a realistic estimate was obtained of the observed seismic type behaviour by these non-linear numerical simulations. Following further validation these non-linear simulations will next be employed in more complex structural components.

6. The obtained results from these specimens were utilized to also validate the expert system based on this limit-state methodology. Again, the observed behaviour was predicted with reasonable accuracy in terms of bearing capacity and mode of failure by this expert system.

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ANALYTICAL AND FIELD EVALUATION METHODS OF THE BEARING CAPACITY OF DEEP PILES AND BARRETTES IN SOFT SOIL AT ST. PETERSBURG

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Abstract

The standard procedure of the field evaluation methods of the bearing capacity of piles used in Russian Federation is described.

A pilot deep pile (length L=52.8m and diameter D=1,2m) constructed under a multi-story building within large layers of soft soil is utilized as an example of the in-situ loading system at a construction site; the results of such a static test are presented and compared with analytical predictions of the bearing capacity for such a pile. Similarly, a pilot deep barrette with rectangular cross section (3,3m x1,1m) and a length of 65 m was also constructed at the same construction site with its bottom end reaching a layer of stiff Vendian clay (bed-rocks of pre-glacial origination). The tests of the pile were performed by using O – cells. The deformation sensors were mounted into the reinforced frame of the pile at 9 levels during the installation of the pile. The tests of the pilot barrette were performed in two stages. During the first stage the standard Top-Down tests were done along the whole length of the pile. During the second stage tests using O-cells were performed transferring the loading in two directions (up and down) after reaching maximal load. An O-cell was placed at the depth of 50 m within the layer of stiff clays.

The diagrams of the pile movement against the applied loading during the first and second test stages are presented together with the predicted total bearing capacity of the barrette found by employing different analytical methods.

Keywords

deep piles, barrettes, soft soils, tests by using O – cells, pile bearing capacity, field tests, soft soils

1. Introduction

The evaluation of the pile's bearing capacity by analytical methods and further checking of the results during field tests is an important stage of the pile foundation design. There are numerous analytical methods for the calculation of the bearing capacity of an individual pile which consider the geometry of the pile and the properties of the adjacent soil. However, quite often these calculated bearing capacity predictions differ from corresponding field tests results of such a pile under vertical loading. As a rule the calculated pile bearing capacity obtained by using Russian norms SNIP 2.02.03-85 is significantly lower than the bearing capacity obtained through standard vertical load testing in-situ (GOST 5686-94). Moreover, there are no Russian norms regarding field testing for barrettes-piles.

During the construction of the foundation for the business center "OCHTA-CENTER" with a height of 396 m, two pilot borrow piles were constructed having a diameter of 1,2 m and a of length 52,8 m and the protection of the external pipe; moreover, five pilot barrettes-piles, with a cross-section of 3,3x1,0m and a length of 65 m long, were also installed.

One of the tasks of the field tests was to determine the total bearing capacity of the borrow piles and barrette. Moreover, to be able to identify what portion of this total bearing capacity was contributed from the sides of the pile (friction) and what was the remaining portion contributed from the bottom of the pile.

2. Engineering and geological conditions of the area for pile tests

Along the depth of drilling (170 m) the following layers of the base were found: artificial sediments tgIV; lake-sea sediments m,lIV; lake-glacial sediments (upper sediments) of the Baltic glacial lake lgIIIb; lake-glacial sediments of Luga moraine (lower sediments) lgIIIz; glacial sediments of Luga moraine gIIIlz; mainland Vendian clays Vkt2. The main average properties of the soil are given in Table 1.

3. Evaluation of the bearing capacity of the pilot pile, diameter D=1,2m and length L=52,8m

The analytical bearing capacity of the friction pile is calculated according the formula in accordance with the Russian norms.

$$F_d = \gamma_c \, [\gamma_{cR} \, R \, A + u \sum \gamma_{cf} \, fi \, hi \,] \qquad (1)$$

Where:

γ_c – coefficient of the work conditions of pile in soil, for driving piles γ_c =1;

R-calculated soil resistance under the lower end of the pile; *A*- area of the cross section of the pile, m^2; *u* – perimeter of the cross section of the pile,m; f_1 – calculated resistance of *i*-layer of the soil along the side surface of the pile, kPa; h_1 – thickness of the *i*-layer touching the side surface of the pile, m, taken as h_i < 2m; γ_{cR}, γ_{cf} – coefficients of the work conditions of soil under the lower end and along the side surface of the pile, considering the method of pile manufacturing.

The piles with the diameter 1,2 m and length 52,8 m under vertical static loading were tested using hydraulic jacks till maximal loading 3500 tones (35000kN) For this purpose, a special loading platform was constructed from crossed **steel** beams and supporting a 33 drill-injected anchors like steel piles “Titan” (Fig.1).

According to the Russian norms (GOST 5686-94) the total bearing capacity of the pile is calculated from the loading applied on the pile when it causes the vertical deformation equal Δ = 20mm. In the examined case it corresponds to the measured loading value equal to Fd,site = 25000kN (see Figure 2). Figure 2 depicts the obtained during testing load-deformation performance of the studied pile.

Analytical calculations against formula (1) resulted in a total bearing capacity value for this pile equal to Fd,calc = 8320 kN (SNIP 2.02.03-85, SP 50-102-2003) 71% of the total bearing capacity (5985 kN) refers to the end of the pile and only 29% of the total bearing capacity (3145kN) referred to the surface side.

The predicted load-deformation behavior obtained by employing the software PLAXIS 3D (a widely known geotechnical programme based on the FEM) – is shown in figure 3. The total bearing capacity value of the studied pile obtained from this methodology is equal to Fd,plaxis = 11000kN (Fig. 3). In calculations under program PLAXIS soils characteristics given in tab. 1 and value of the vertical loadings applied to pile according to values specified on fig. 2 and 3 were taken into account.

In table 2, the predicted total bearing capacity values obtained as explained before are compared with the measured pile vertical loading capacity measured during the in-situ testing.

The predicted tested pile total bearing capacity values, which were obtained by different methods, are listed in Table 2 column (2) and are compared

Fig. 1. General view of the loading platform for testing the piles with vertical static loading

Table 1
The main average properties of the soil

N	Soil type	Geological Index	γ, кN/м^3	W	e	I_L	E, MPa	φ, degree	c, MPa
1	Fill soil	tg_{IV}	17,5	-					
2	Sea and lake sediments	m,l_{IV}	19,6	0,256	0,68	0,71	14	24	0
3	Upper lake-glacial sediments	$lg_{III\,b}$	18,6	0,360	0,98	1,1	4.5	7	0,006
4	Lower lake-glacial sediments	$lg_{III\,z}$	20,4	0,220	0,60	0,87	10,5	17	0,03
5	Moraine sediments	$g_{III\,lz}$	21,5	0,160	0,43	0.25	17	22	0.036
6	Deployed Vendian clays*	V_{kt2}^{1}	21,4	0,176	0,50	-0, 35	16	14	0,13
7	Not deployed Vendian clays	V_{kt2}^{2}	22,3	0,129	0,37	-0,79	113	22	0,84

* Deployed soil – soil having infringements, cracks and inclusions

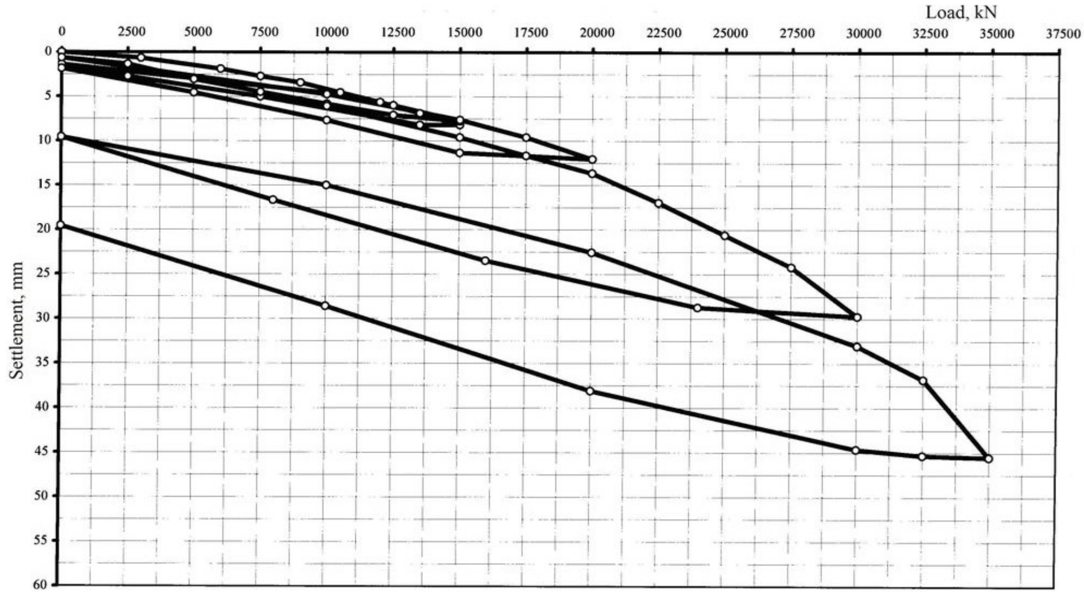

Fig. 2. Vertical static loading test results for the pile with 1.2m diameter and a length of 52,8 m

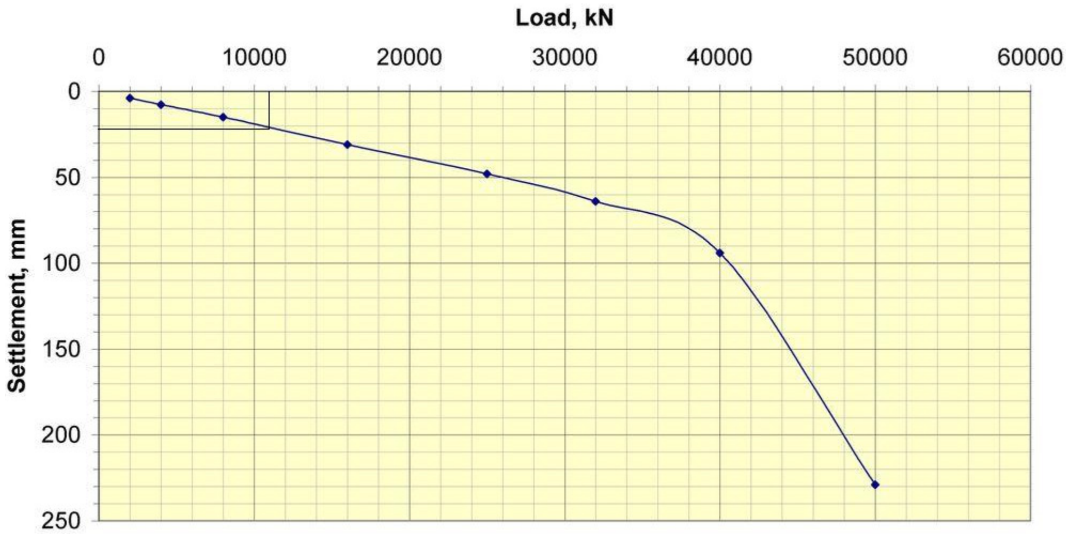

Fig. 3 Predicted vertical load-deformation performance of the tested pile obtained by the PLAXIS 3D software

Table 2
Predicted and measured Total bearing capacity of the test pile (D=1,2 m and L-52,8 m) F_d, kN

Description of bearing capacity values	Total bearing capacity of the test pile (kN)	Ratio of measured / predicted values
(1)	(2)	(3)
Predicted value $F_{d, calc}$ according to (SNIP 2.02.03-85, SP 50-102-2003)	8320	3.005
Calculated value based on PLAXIS 3D, $F_{d, PLAXIS}$	11000	2.273
Measured value during the field tests, $F_{d, site}$	25000	1.0

to the measured total bearing capacity value which is listed in the same table. In column (3) of table 2 the ratio of the measured over the predicted values is also listed. As can be seen, the measured total bearing capacity value of the tested pile is 2.273 times larger than the PLEXIS 3D prediction and 3.005 times larger than the prediction based on the Russian norms.

4. The evaluation of the bearing capacity of pilot barrette, size lxb=3,3x1,0m length L=65m

The pilot test of the barrette, size lxb=3,3x1,0m length L=65m with its bottom located in the mainland soil - Vendian clays Vkt2 was performed at the same construction site. It was supposed to use "O-Cell" and deformation sensors for the test. The plan of their location is given in Fig 4.

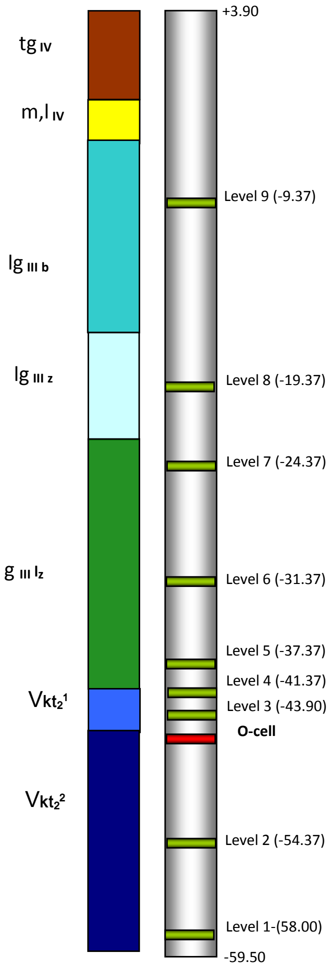

Fig.4. Plan of the location of O-cell and sensors of moving on the pilot barrette

The pilot barrette was tested by applying vertical static loading in two stages. At the first stage the barrette was tested by vertical loading applied to the top of barrette (top-down). After reaching maximal possible loading the second stage started using "O-cell" on the absolute level -45.00. Due to that the loading was transferred in two directions - up and down.

The test results of the pilot barrette at the first stage of loading, when the "Top-down" method was applied, are given in Fig.5. At the maximal loading 30000 kN the settlement was less than 20 mm.

The extrapolation of the diagram loading-settlement till horizontal line, corresponding Δ=20mm allows to evaluate the total bearing capacity of the pilot barrette during its initial loading Fd,Top-Down = 32000kN.

The analytical calculations of pilot barrette according the formula (1) (SNIP 2.02.03-85, SP 50-102-2003) gave the total bearing capacity Fd,calc = 31244, the result close to the test results according Top-down method.

The calculations by PLAXIS 3D showed the total bearing capacity of barrette Fd,PLAXIS = 27800 kN (Fig.6), which is less than the results received by "Top-down" method.

The results received by the tests with O-cell, made after Top-down tests, are shown in Fig.7.

The red line in Fig.7 corresponds to the Top-down test of the lower part of the pile and allows evaluating the bearing capacity of the bottom end of the barrette.

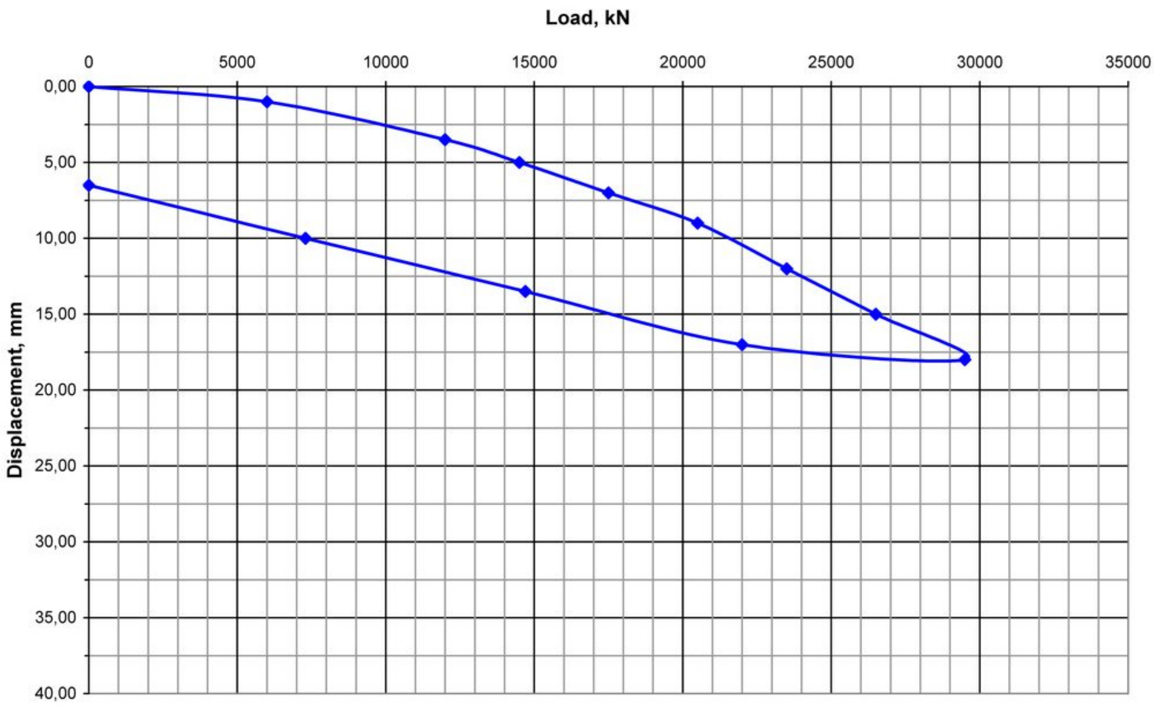

Fig 5. Moving of barrette due to applied vertical loading

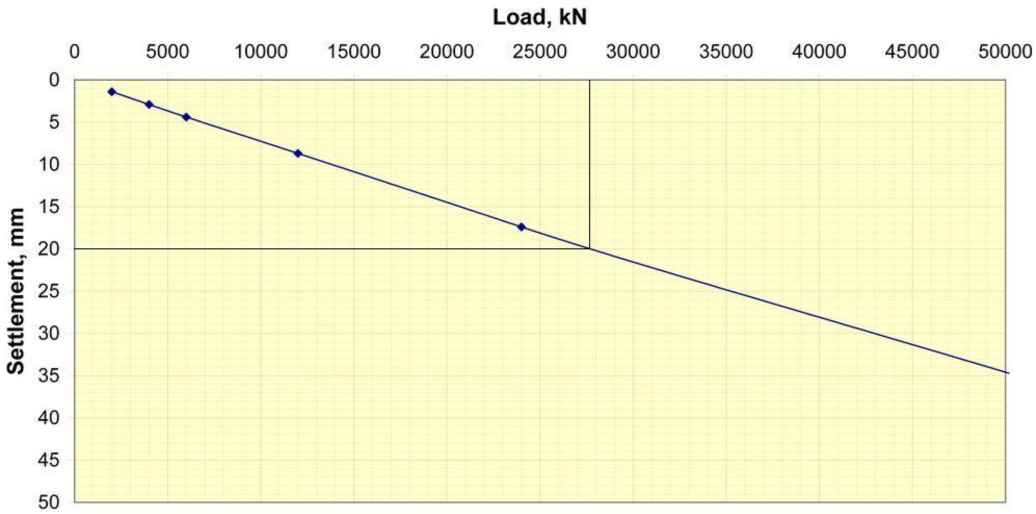

Fig.6. Calculation results of the vertical loading (Top-down) made by PLAXIS 3D

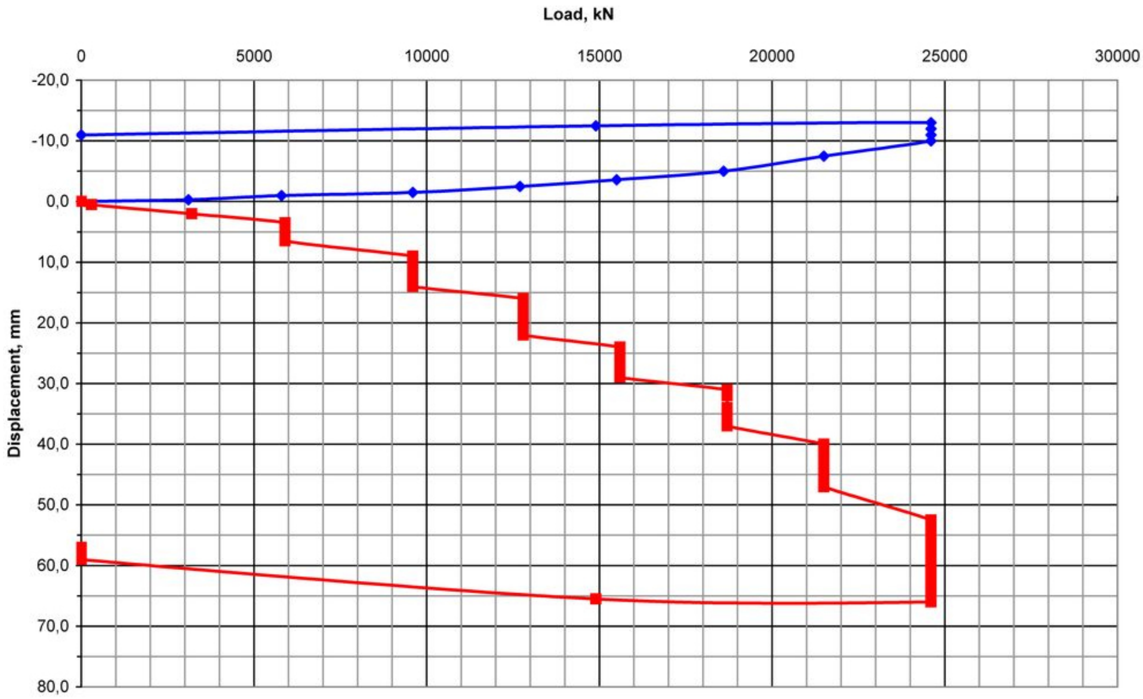

Fig.7. The diagram of vertical movement of the pile due to applied vertical loading by the “O-cell» method

Table 3
Predicted and measured Total bearing capacity of the test barrette 3,3x1,0 m and L-65, F_d (кN) for the first and next loading

Description of bearing capacity	Total bearing capacity of the test barrette (kN)	Ratio of measured (first loading / predicted values	Ratio of measured (next loading) / predicted values
(1)	(2)	(3)	(4)
Predicted value $F_{d,\ calc}$ (SNIP 2.02.03-85 and SP 50-102-2003)	31244	1.024	1.360
Calculated value based on PLAXIS 3D, $F_{d,\ PLAXIS}$	27800	1,151	1,529
Results of the field tests Top-Down (first loading), $F_{d,\ \text{Top-Down}}$	32000	-	
Results of the field tests «O-cell» (next loading), $F_{d\ ,\text{O-cell}}$ $F_{d\ f,\ O\text{-}cell}$ $F_{d\ R,\ O\text{-}cell}$	42500 29500 13000	-	

The extrapolation of the upper part of the diagram “loading” (blue line) till the crossing with horizontal axis, which correspond to Δ = 20 mm allows to evaluate the bearing capacity of the side surface of the pile as $F_{df,o\text{-}cell}$ = 29500 kN. The lower part of the diagram (red line) shows the bearing capacity of the bottom end of the barrette at least $F_{d\ dR,o\text{-}cell}$ =13000 kN.

The total bearing capacity of the pile calculated by this test method is at least $F_{d,o\text{-}cell}$ = 42500 kN

As it was expected the total bearing capacity of the barrette during the next loading by the “O-cell” method was higher than during the first loading by the “Top-down” method (by more than 30%).

The test results given by the “O-cell” method and by analytical calculations are presented in summary in Table 3.

5. Conclusions

1. The tests results showed that the measured bearing capacity of the pilot pile with diameter D=1,2 m and length l=52,8m was significantly higher (more than 200%) than the corresponding value calculated either according to Russian norms (SNIP 2.02.03-85, SP 50-102-2003) or by PLAXIS 3D.

2. The tests results showed that the measured bearing capacity of the pilot pile with a cross section 3,3m x 1,0m and length L=65 m was quite close to the corresponding value predicted according to Russian norms (SNIP 2.02.03-85 and SP 50-102-2003) (this calculated value is only 2,5% smaller than the measured value). The corresponding value calculated by PLAXIS 3D is 15% smaller than this measured value.

3. The bearing capacity of the soil at the bottom of the barrette (13000kN) represent one half (1/2) was of the bearing capacity offered by side surface of the barrette (29000 kN) in spite of the thick layers of soft soil that lie all along the length of the tested barrette.

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FUNCTIONALITY OF SAINT PETERSBURG HISTORIC GROUPS OF BUILDINGS FOR ARRANGEMENT OF BUSINESS TOURISM EVENTS

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Abstract

The paper demonstrates that the fractional infrastructure of establishments has been developed within Saint Petersburg to-date suitable for arrangement of business tourism events and capable of servicing each of three business tourists main groups (state, corporate / associate and individual levels). It differs from many other European congress cities with intensive use of historic and adapted buildings. The paper reviews existing historic buildings of Saint Petersburg built in the 18^{th}–19^{th} centuries with the point of possible use in terms of business tourism events arrangement. The structure of such buildings' main premises was reviewed for the purpose of their intended use for conferences, meetings, exhibitions, etc. It is shown that the existing potential of historic buildings and groups of buildings is considerably actual; there are prospects of their use as well as limitations of such possibilities.

Keywords

Saint Petersburg architecture, business tourism establishments, historic buildings and groups of buildings, functional zoning, congress and exhibition activities.

1. Introduction

Industry of business tourism is one of the most fast-growing and highly profitable branches of the global economy. About 40 mln events with the participation of around 2 bln people are being held in this segment. Annual investigations carried out by World Tourism Organization (WTO) show that the share of business tourists within the overall world tourist flow is about 15%, however "up to 60% of overall tourism turnover is accounted for business tourists". Business meetings industry yields the big cities 4 (four) times higher revenues, rather than arrangement of other social public events.

In the course of this investigation it was established that fractional infrastructure of establishments has been developed within Saint Petersburg to-date, which is capable of servicing each of three business tourists main groups (state, corporate / associate and individual levels). It differs from many other European congress cities with intensive use of historic and adapted buildings. This paper considers the possibility of using the following types of historic buildings for the purposes of business tourism: palaces, mansions, theatres, maneges, libraries.

2. Materials and methods

Comprehensive approach forms the methodological foundation for the investigation and includes classification and synthesis of the results obtained in the course desktop study of existing buildings, such as Saint Petersburg palaces, mansions, theatres, maneges and libraries, by main typological features and qualitative characteristics in order to review architectural concepts of these buildings and identify their abilities to be used for arrangement of business tourism events.

3. Results

As of today, there is an assemblage of establishments in Saint Petersburg available for hosting business tourism events which is described with the wide variety of building types constructed during different period of times and having different capacity and scope of services provided.

Historic buildings of Saint Petersburg are widely used for arrangement of business tourism events. It is due to the fact that these types of buildings include premises exhibiting characteristics suitable for arrangement of business tourism events. Besides that, these buildings are very highly attractive in terms of the quality of historic interiors and location in the downtown. The following types of buildings are considered as historic buildings and group of buildings suitable for the purposes of business tourism events: palaces, mansions, theatres, maneges, libraries built within the pre-revolutionary period.

Considering the premises these buildings consist of, they can be used for the purpose of hosting conferences, exhibitions, symposia, presentations, special events and other business tourism events. Based on the investigation results, Summary Table 1 is generated to describe the types of historic buildings and group of buildings suitable for arrangement of business tourism events in Saint Petersburg.

3.1. Palaces

During the pre-revolutionary period palaces were considered as ceremonial residences of Emperors' or upper class representatives. Palaces also played a role of significant cultural and business centers: ceremonies, honorable receptions, balls, festive events and assemblies took place there, as well as meetings of state authority bodies.

After the revolution 1917, the main function of palaces was abolished, no new palaces were constructed and the bodies of soviet authorities, administrative and social organizations, museums were housed in existing palaces (Kirikov, Andreeva, Shmeleva, 2003).

Palaces of Saint Petersburg can be classified as follows:

- in terms of size: large (capacity of the largest room is 500–960 persons), medium (capacity of the largest room is 250–500 persons), small (capacity of the largest room is up to 250 persons);
- in terms of type: city or country.

Palaces located in historical center of Saint Petersburg are included into the ribbon building or deep within the site with an open space at the front door (Bunin, Savarenskaya, 1979). Due to the lack of surrounding recreation space, city palaces may have small gardens.

Palaces in suburbs of Saint Petersburg — Petergof, Tsarskoye Selo (Tsar's Village) — are larger in terms of capacity. They are very popular among facilitators of business events, especially in summer. The specific feature of country palaces is the availability of recreation space i. e. surrounding parks with small palaces, pavilions, fountains, park arrangements. Isolated park pavilions can be very convenient for negotiation purposes.

Activities related to business tourism are carried out in such palaces as the Tauride palace (architect I.E. Starov, 1783–1789; rebuilt by A.R. Bakh in 1905–1906), the Palace of Bezborodko the chancellor — Central museum of communication named after A.S. Popov (architect D. Quarenghi, 1783–1795; rebuild and restored in 2000–2003), the Palace of Grand Duke Vladimir Alexandrovich — House of Scientists (architect A.I. Rezanov, 1764–1772), the Catherine Palace in Tsar's Village (architect B.F. Rastrelli, 1752–1756; reconstructed after the Great Patriotic War in 1957–1980s by architect A.A. Kedrinskiy), the Petergof Grand Palace (architect J.-F. Braunstein, J.-B. Le Blond, N. Michetti, 1710–1724; reconstructed after the Great Patriotic War in 1971–1990s) (Petrov, Petrova, Raskin, 1983).

The specific feature of the palaces included into Saint Petersburg business tourism infrastructure is the compliance of functional structure to major requirements applicable to the event site.

These palaces have the entry zone with large entrance hall and lobby, availability of multi-functional rooms of various capacity: from study rooms for several persons up to large rooms for several hundreds of participants. There are restaurants in the palaces or it is possible to arrange catering service.

It is possible to arrange state (federal or regional) and corporate or associate summits, congresses, symposia, conferences, negotiations, exhibitions, special events, banquets in historic places depending on the level of safety.

The main advantage of the palaces is their magnificent interiors. The possibility to hold an event in a ceremonial atmosphere is their unique feature. However, these buildings exhibit some deficiencies preventing them from being the same worth as up-to-date centers:

1. Engineering constraints. Palaces arrangements are not suitable for placing numerous technical facilities and units (simultaneous translation equipment, acoustic systems, and climate control systems). Besides, machinery can be harmful for historic atmosphere of palaces' rooms. Arrangement of events is highly regulated due to specific nature of these premises and required long-term finalization.
2. Rooms of the palaces are not capable of being transformed.
3. Insufficient servicing for convenient staying of a business tourist. Absolute lack of extra servicing.
4. Limited capacity. The Duma room of Tauride palace is the largest one in terms of capacity (960 persons). Most of the palaces have rooms with the capacity below 250 people.

The Congress Palace (former Konstantin Palace) located in Strelna can be the example of the palace most complying with up-to-date requirements to arrangement of business events.

As a result of large-scale reconstruction completed in 2003, a contemporary complex was created within the historic area which is the most exclusive place for hosting business events in the North-West region. This place was used to carry out G8 and G20 summits. Numerous rooms, developed infrastructure and up-to-date technology support effective activities. However, the Palace of Congresses has considerable limitation in terms of arrangement of business tourism events. This complex is mainly intended for meetings at the higher state level. Besides, the capacity of its largest room is relatively small and equals to 270 persons only. (N. Michetti, 1720; F. Rastrelli, 1750; reconstructed in 2003).

3.2. Mansions

During the pre-revolutionary period city mansions were private houses intended for wealthy families of higher societies to live in. As opposed to the palaces, mansions are more compact and they were mainly functioned as residential buildings. On relatively small occasions they were used to host festive events, meetings, ball events, celebrations [37]. Nowadays mansions are the places for administrative and public organizations and museums.

Mansions buildings in Saint Petersburg differ by great variety of space, lay-outs and artistic solutions. "Their architectural composition and finishing depended on social and financial status of the customer, his family needs and personal preferences. So the range of mansions architectural variety was great: from luxurious palazzo in the downtown to modest one-storey buildings in the outskirts" (Punin, 2009).

Mansions are used for the purpose of business tourism, if their functional structure is in compliance with the main requirements (availability of front entry door — entrance hall and lobby, multi-purpose rooms, possibility to arrange restaurant / banquet room / catering); areal location; attractive exterior and interior of the mansion [46, pp. 35–37]. The following mansions are the most popular ones for the purpose of business meetings arrangement: mansion of M.F. Kseshinskaya — State museum of Russian political history (architect A.I. von Hogen, 1904–1906), mansion of A.A. Polovtsov (the House of Architect) (architect A.C. Pel, 1835–1836).

The advantage of Saint Petersburg mansion is its high executive class. The deficiencies are as following: engineering constraints, lack of possibility to transform the rooms, insufficient extra service, limitation in capacity (rooms are compact and their average capacity is about 100–150 persons).

3.3. Theatres

Theaters in Saint Petersburg appeared from the second half of the 18th century in order to exercise entertaining functions (Punin, 2009). The core of a theater is a large audience space with 250–2,000 seats. Besides, theater buildings are equipped with some additional functional areas (large entrance hall; cloak room; entrance lobby; canteen / restaurant / banquet room) which make this type of establishment suitable for arrangement of congress events. New stage of the Mariinsky Theatre in Saint Petersburg was constructed with consideration of possible business tourism events.

The following classification may be suitable for theatres: in terms of capacity, it terms of construction time (historic or modern).

The following theatres are among the most actual ones for business tourism events: new stage of the Mariinsky Theatre (architect Jack Diamond AB Diamond and Schmitt Architects, 2008–2013), the Hermitage theatre (architect D. Quarenghi, 1783–1787; reconstructed in 1991).

Theaters may be used for hosting of large state, corporate and associate forums and congresses with up to 2,000 participants.

Theatre sites in Saint Petersburg differ from palaces and mansions by technical support: their specific technical equipment is intended for main function — i.e. entertaining. The deficiency is in difficulty or, in most cases, impossibility of audience space transformation, change in audience seats arrangement for specific event. Theatres are not able to be used as centers of business tourism in continuous mode due to their main function and may be only used on special occasions.

3.4. Maneges (or riding halls)

The first Manege in Saint Petersburg was built in 1756–1759 for the First Cadet Corps. Initially maneges were intended for horse riding trainings: horse schooling, ceremonial horse dressing-out. However, maneges lost this function with time.

Currently 2 (two) buildings are used as the objects where business events are taken: Manege of the First Cadet Corps and Mikhailovsky Manege. Each of these maneges is specialized in hosting specific events.

Manege of the First Cadet Corps is a trendy site intended for cultural and business as well as leisure events and is equipped with high-end up-to-date equipment (architect J. Borkhard, 1756–1759; reconstructed into the multi-purpose room in 2003–2005).

Mikhailovsky Manege is the place for exhibitions and it was used as such even in the pre-revolutionary period: Exhibition of Imperial Russian Society of Gardeners in mid. 19th century, The 1st All-Russian Hygiene Exhibition, first automotive exhibitions, in 1909 – International Show of Modern Inventions (Nikitin, 2004).

In 1950s the building subjected to internal redevelopment and was adapted for Winter Stadium. Currently Mikhailovsky Manege is a multi-purpose house and it still used for the purpose of sport competitions. The site is also used as a place for exhibitions, arrangement of city cultural and business as well as leisure events, conferences (architects V. Brenna, C.G. Rossi, 1798–1800, 1824; reconstructed in 1974).

The Horse Guards Manege was adapted for a garage during Soviet time and in 1967 it was passed to Union of Artists in order to host exhibitions. Occasionally it was used as the place for cultural and business as well as leisure events. Currently the building of the Horse Guards Manege is under overall reconstruction and repair aimed at creation of up-to-date world class exhibition house (architect D. Quarenghi,1804–1807; overall reconstruction and repair started in 2013).

Rooms of former maneges are suitable for holding various event: forums, congresses, banquets, festive events at the corporate / associate level. The advantage of these houses is their ability of being easily transformed as well as their capacity (the area of the Horse Guards Manege and Mikhailovsky Manege is about 4,400 sq.m). The deficiency is complete lack of extra and related services.

3.5. Libraries

The first library in Saint Petersburg was the Imperial Public Library (currently — National Library of Russia). It was built in 1795. Generally, libraries, besides their intended use as a book storeroom, served as particular cultural center, where lectures, exhibitions, evening events were taken. Nowadays, available space of libraries makes them suitable for hosting business tourism events.

Libraries of Saint Petersburg subject to the following classification: of federal importance (The National Library of Russia, Boris Yeltsin Presidential Library), of city importance (Central City Public Library named after V. Mayakovsky), of regional importance.

Libraries of federal importance are suitable for business events to the fullest extent. They have larger rooms and higher level of technical support.

Boris Yeltsin Presidential Library is located in Synode building reconstructed with consideration of social and business function: a roof was created above the inner yard and an atrium was arranged – a room intended for round and ordinary conferences with participation of about 300 persons.

The area of multi-purpose multimedia center of the Presidential Library is intended for arrangement and carry out of lectures, video presentations and workshops.

There is an exhibition area which is intended for joint exhibitions with participation of museums, libraries and universities (architect C.G. Rossi, 1829–1836; reconstructed in 2007–2009)

Libraries host corporate / associate subject conferences, presentations and scientific workshops with participation of up to 300 persons. Boris Yeltsin Presidential is the only library in Saint Petersburg taking state conferences, symposia, exhibitions. Except for the Presidential Library and the National Library of Russia, all other libraries do not meet particular requirements, so their use for takin events is limited.

4. Summary

In the course of the performed investigation it was observed that the structure of numerous historic groups of buildings involved into hosting

Table 1
Description of buildings and buildings groups with functional zoning adapted for hosting business tourism events. Saint Petersburg

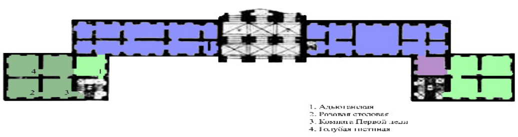
Type of Establishment	Examples of Establishments	Main Premises	Present-Day Use
1	2	3	4
Palace	   Konstantine's palace N. Michetti (1720), F. Rastrelli (1750), 2003 — reconstruction	Residential premises	—
		Ceremonial rooms	Social and business function (symposia, conferences, exhibitions, banquets)
		Study-rooms	Negotiations
		Parks, gardens and relative pavilions	Negotiations is pavilions located in parks

Table 1. Continued

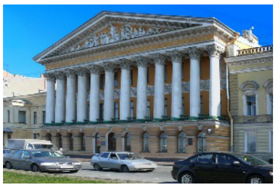
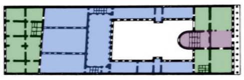
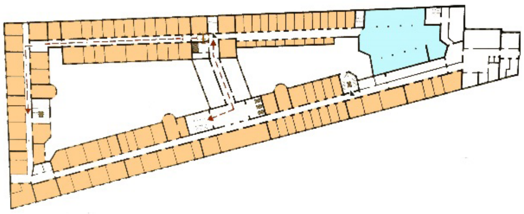
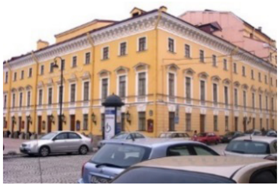
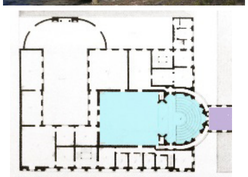
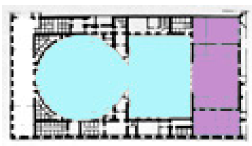
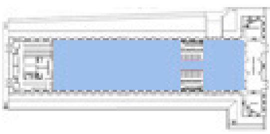
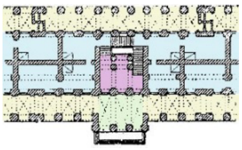
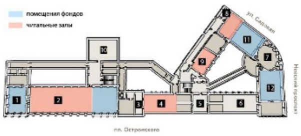
1	2	3	4
Mansion	    Rumyantsev Mansion V.A. Glinka 1824; Polovtsov Mansion A. C. Pel 1835–1836	Residential premises	—
		Ceremonial rooms	Social and business function (symposia, conferences, exhibitions, banquets)
		Study-rooms	Negotiations
Hotel	   Oktiabrskaya Hotel A.P. Gemilian (1847–1857), A.S. Khrenov, O.R. Muntz (1896–1912)	Residential premises	Residential function
		Restaurant	Catering function
		Meeting rooms Negotiating rooms	Conference center
Theatre	    The Hermitage Theater D. Quarengh 1783–1787; Mikhailovsky Theater A.P. Brullov 1831–1833	Audience space	Entertaining function (performances, music events), social and business function (conferences)
Manege	  Manege of the First Cadet Corps J. Borkhard 1756–1759	Horse riding area	Multi-purpose room for exhibitions, banquets, conferences

Table 1. Continued

1	2	3	4
University	 	Lectures rooms Study-rooms Laboratories	Educative and teaching activities (lectures, workshops, practice)
		Event hall	Ceremonies, social and business function (conferences)
	  St. Petersburg State University D. Trezzini 1722–1742 Saint Petersburg Mining Institute A.N. Voronikhin 1806–1811	Lobby	Exhibitions
Library	 	Book storeroom	Book storing
		Reading rooms	Reading of books
		Lectures rooms	Conference room
	Imperial Public Library C.G. Rossi (1796–1801) E.G. Sokolov (1828–1834)	Halls for books exhibitions	Exhibitions

of business tourists' events is out of date and requires architectural and technical upgrade with consideration of trends in the world experience. The availability of large amount of adapted buildings explains low rating of our city among congress cities. However, adapted buildings have found their business segment at the market as the places for arrangement of business events due to their appeal to historic interiors, famous names and location in the downtown.

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STRUCTURAL FEATURES OF NANOMODIFIED CEMENT STONE

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Abstract

The article reviews the effect of mixing water pre-modified with fulleroid nanomaterial on cement paste rheology and cement stone structure. Also pH value of modified mixing water is determined and its variation under the effect of nanomaterial is shown. Typical acidulation of obtained slurry is noted and time of its activity (aggregative stability) beneficial to formation of cement stone structure is determined. Analysis of the obtained results is evidencing changes in properties of cement systems prepared with the use of nanostructured water, namely increase in cement paste setting time and improvement of its fluidity with rheological properties maintained; enhancement of pore structure with improved uniformity of distribution throughout the volume, leading to increase in strength properties of cement stone.

Keywords

Fulleroid material, nanomodifier, cement stone structure, cement composites: structure and properties, nanomodifiers

1. Introduction

For a long time, cement science was aimed at studying of cement and cement fillers' properties and their efficient usage. Less attention was paid to mixing water. At the same time, water, being a structural component of raw mixture, is of importance in formation of cement composite structure and properties.

In prior years methods were developed for mixing water structurization with magnetic and electromagnetic field exposure, electrochemical, acoustic and plasma activation, etc. (Bad'in, 2006; Leonov, 1997; Yudina, 2005). However, methods of physical activation of mixing water were not of extensive use due to objective difficulties at the stage of industrial implementation. For example, mixing water activated with magnetic field completely loses its re-acquired properties in a very short period of time. At that, extra fine equipment adjustment is required for the purpose of effective magnetic activation. In case of electrochemical activation of water, the main problem is in setting of effective electrochemical processing parameters (electric field intensity, current density, processing time) as they are dependent from various factors (properties of materials in use, physical and chemical properties of source water, ambient temperature). Such parameters can only be determined under workshop conditions by experiments. But the most essential problem impeding methods of physical activation is in need of processing lines' upgrade with costly equipment for water activation and complete revision of operating procedures.

Due to development of nanotechnology (Bal'makov, 2005; Gusev, 2005; Korolev, 2006) new possibilities can be considered for influencing water structure and properties (Kovaleva, 2008; 2006). Therefore, targeted control of structure forming processes and properties of cement composites that represent a complex multistage system (including a nano-level as well) is considered as up-to-date and promising way of concrete technology development.

2. Materials and Methods

Within the framework of experimental studies, effect of fulleroid material (Fig. 1) consisting of 20–200 nm particles was investigated. Working slurries were made based on distilled water processed with the use of distillation unit of DU-23 type by dilution of initial (highly concentrated) slurry down to required concentrations.

To determine any changes in water, upon introduction of carbon clusters, pH value was measured using pH meter of Delta 320 type.

In order to investigate the effect of nanostructured mixing water, experiments were carried out to determine cement paste setting time and fluidity as well as cement stone structure and properties as applied to various cements: PC 500-D0 (JSC Oskolcement), PC 400-D0 (CJSC Belgorodskiy cement), PC 500-D0-N (JSC Mordovcement), CEM I 42.5R (Holcim, Germany). The cement compositions under investigation varied in water-cement ratio; hardening conditions and modes.

3. Results

The results of pH value measurements at different nanomodifier concentrations are shown in Fig. 2.

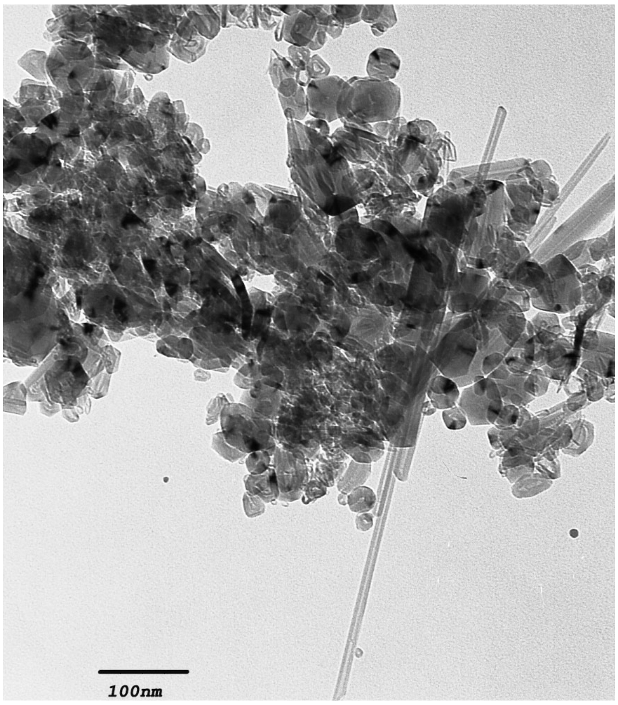

Fig. 1. Fulleroid material

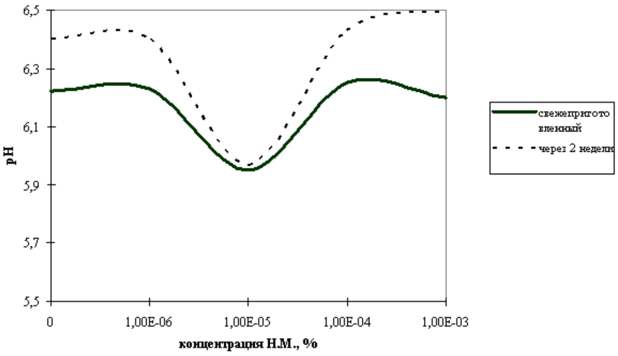

Fig. 2. Change in pH value of nanomodified mixing water

The test results given in Table 1 were obtained for samples of CEM I 42.5R Portland cement (Holcim, Germany) with water-cement ratio (W/C) of 0.26 corresponding to normal density.

The strength test results obtained for nanomodified cement stone are shown in Fig. 3 and 4.

4. Summary

The effect shown in Fig. 2 is evidencing quite a narrow nanomodifier concentration range iin which the peak of slurry pH decrease occures.. This effect can be explained only by change in ionic product of water, caused by sorption of hydroxyl groups at the surface of nanoparticles. As it is known, the degree of water electrolytic dissociation under normal conditions is relatively low (10^{-9} at the temperature of +20 ^{0}C) (Zatsepina, 1987). At that, concentration of hydroxonium ions [H_3O^+] and, therefore, of hydroxyl groups [OH^-] equals to 10^{-7}. In simplified form, reactions of water dissociation are described by the following scheme:

$$[H_2O] \rightleftarrows [H_3O^+] + [OH^-]$$

At particular concentration of nanomodifier, the above dynamic equilibrium displaces to the right due to sorption of [OH^-] ions at the surface of nanoparticles.

Table 1
Effect of nanostructured water on cement paste and stone properties

Properties	Concentration of nanomodifier in mixing water, %					
	0	10^{-7}	10^{-6}	10^{-5}	10^{-4}	10^{-3}
Setting time, min: - start - end	 200 230	 200 235	 225 250	 210 250	 235 270	 220 245
Fluidity determined by flow in shaker apparatus, mm - upon preparation - in 60 min - in 150 min	 160 152 115	 159 150 122	 162 159 134	 173 156 140	 168 163 142	 155 154 139
Water sorption at capillary rise, kg/(m^2 $h^{0.5}$)	0.395	0.377	0.356	0.265	0.332	0.315
Uniformity of pore size, α	0.288	0.259	0.295	0.312	0.317	0.297
Average pore size, λ	1.623	1.652	1.595	1.611	1.619	1.601

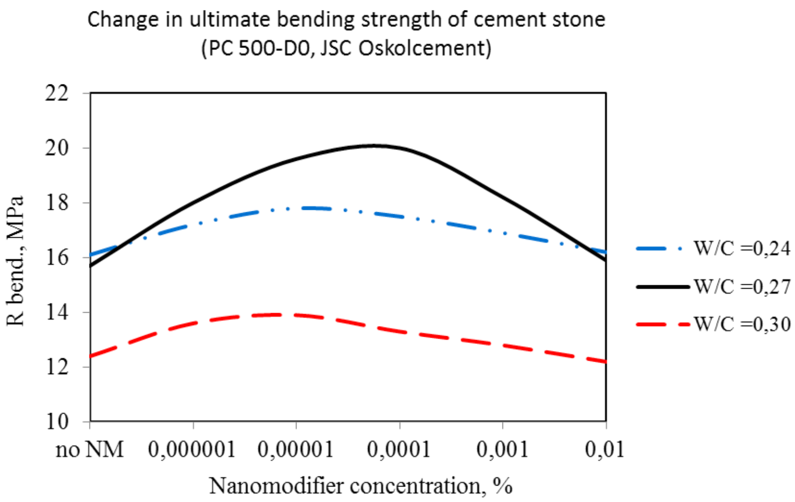

Fig. 3. Ultimate bending strength of nanomodified cement stone

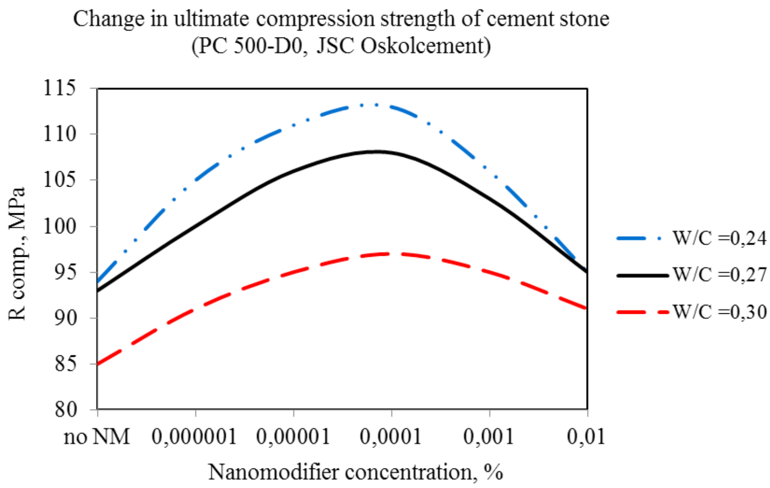

Fig. 4. Ultimate compression strength of nanomodified cement stone

Obviously, carbon nanoclusters act like stabilizers of water system self-organization process. Continuation of the process under investigation leads to generation of a secondary structure, i.e. fractal volume mesh (Fig. 5) which is located throughout the entire volume of water and locally changes concentration of hydroxyl groups, resulting in volumetric pH change; and at the stage of cement stone formation it allows to organize crystallization process and decrease size of crystalline hydrates (Bal'makov, 2005; 2006).

Noted acidification of slurry has beneficial effect on processes of cement stone formation, as in this case neutralization reaction can occur between the most soluble form of calcium hydroxide and active hydroxonium ions. Water is the product of this reaction and it further couples with less active products of portland cement hydration. At that, new water generation as a product of chemical reaction within the system will facilitate cement paste plasticization. Further increase in nanomodifier concentration leads to its sorption ability decrease due to aggregation of its active particles and, consequently, of its total surface, that causes pH value increase.

In our case an approved method (Sheykin, 1979) was used to determine indirect porosity characteristics: α – uniformity of pore distribution, λ – average pore size. Using the obtained values, it is possible to calculate capillary size, but in our case precise determination of size in not of importance. Change in λ describes differences in capillary size only. I.e., λ describes an average capillary size and α describes uniformity of capillary size distribution. The given parameters allow to qualitatively describe differential porosity and perform quantitative evaluation of the broad range of pore and capillary sizes.

Analysis of the obtained results shows more uniform distribution of pores throughout the volume of induraed modified cement stone. It is proven by increase in α. At the same time, average pore size slightly reduces (decrease in λ).

Thus, pore structure of nanomodified stone undergoes changes: generation of large amount of microcapillary is observed in comparison with

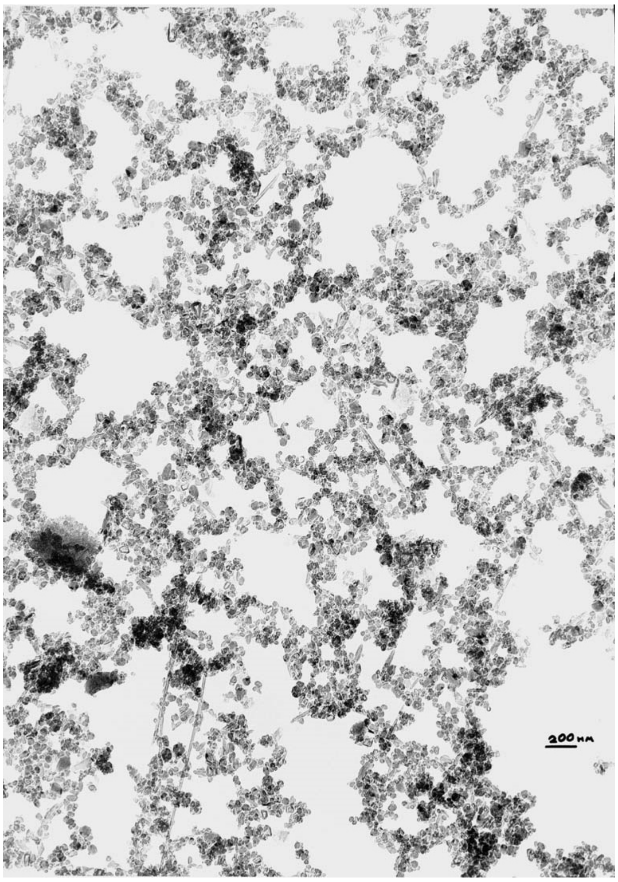
Fig. 5. Fractal mesh

ordinary cement stone that contains larger amount of macrocapillary. At that, pore space in nanomodified cement stone is more ordered. However, it should be noted that similar to changes in physical and chemical properties of water, behavior of pore space variability in nanomodified cement stone is of polyextreme character. But despite nonlinear dependence of generating capillary from nanomodifier concentration, significant change was determined in generating pore space upon cement mixing with slurries of different nanomodifier concentration.

Such significant changes in behavior of nanomodified cement stone pore structure cannot be explained only in terms of introduction of nanosized carbon objects, which are physically located in capillaries, due to insufficient quantity of nanomodifier introduced into cement system in order to stop macrocapillary growth. In this case generation of the large amount of microcapillaries is affected by change in hydration process. This fact was proven earlier by the example of cement paste setting time and fluidity.

Analysis of the obtained results shows change in properties of cement systems prepared with the use of nanostructured water, and the following can be concluded:

- when concentration of nanomodifier in mixing water equals 10^{-4} …10^{-6} %, that correspond to the range of decreased pH, some improvement of cement paste setting time and fluidity occurs as well as maintaining of its rheological properties in time;
- within the same concentration range a tendency to improvement in cement stone strength properties within 15-20% is observed (depending on cement type, water-cement ratio and other factors).

It should be noted that effective concentration of nanomodifier in mixing water changes depending on types of used fulleroid compounds and, thus, results in change of cement system properties. Therefore, there is a possibility of adjustment using complex of required cement paste and stone properties and, at the same time, using concrete mixture and concrete properties (Kovaleva, 2008).

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A COMPARATIVE STUDY OF WATERFRONT USE AS ECOSYSTEM SERVICES IN SAINT-PETERSBURG AND OSAKA

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Abstract

Based on the concept of ecosystem services, this study aimed to clarify a direction for the use of precious waterfronts remaining in congested large modern cities. Rivers and canals of Saint Petersburg, Russia and Osaka, Japan were selected as the study subjects. The actual status of waterfront use in both cities was examined through qualitative analyses from the viewpoint of ecosystem services. As a result, it was revealed that there were differences between the two cities in their use of waterfronts, and that the creation of waterfronts with high environmental value based on the consideration of ecosystem services has a positive effect on tourism, land price, and health.

Keywords

Use of waterfronts, ecosystem services, Saint Petersburg, Osaka, rivers, canals

1. Introduction

Today's cities are becoming congested. Cities are characterized strongly by the fact that they use land vertically, as opposed to rural regions which use land horizontally. This clear distinction in land use is brought by major advances in modern civil and building engineering technologies. Such progress has enabled urban buildings to be both extremely large and tall, and encouraged the vertical use of small urban spaces, i.e. utilization of space high into the air. The causes also lie in the fact that urban land is not replaceable, is high in price, and difficult to obtain in the midst of over-concentrated and highly-developed urban functions, resulting in the need for effective vertical use. Thus urban spaces become smaller resulting in a more closed feeling every year.

In such closed urban environments, rivers, canals and parks provide open space. Until now, parks have been considered in urban planning as they were regarded as a necessary facility from the standpoint of fire protection and health. On the contrary, however rivers and canals were often regarded as a means of waterway transport and viewed in terms of flooding. In fact, the open spaced rives and canals remaining in cities today flourished as waterway transport systems for daily commodities in the old days and industrial raw materials and goods after the Industrial Revolution. However, with the advancement of large-scale industrial production and cargo shipment, the distribution function needed for commerce and trade as well as industrial production were transferred to the city periphery or neighboring locations, and the role of such urban waterway transport was reduced. Therefore, some urban rivers and canals were filled and used as parks and roads.

With a good amount of vegetation and organized open space, urban parks have high environmental and health value, providing city dwellers and workers with comfort and relaxation. Likewise, the waterfront spaces of rivers and canals in cities also represent a valuable and precious urban space consisting of water in an open continuous linear form, comparable to parks. Rivers and canals feature expansive ecosystems, unlike urban parks which are limited in this regard, as they comprise ecosystems which link to watersheds or the sea.

It is now time to review rivers and canals for city dwellers, workers and visitors, in order to further enrich urban ecosystems by enhancing their environmental and health value, as a legacy of open spaces with rich ecosystems.

2. Study objectives

Urban waterfront spaces such as rivers and canals are valuable assets which provide natural elements to congested cities and their open atmosphere has the potential to further increase environmental and health value. Thus such waterfront spaces have been regarded as important for city planning in terms of maintenance and the creation of a pleasant urban landscape. They have been primarily considered as a backdrop of ‘dry’ inorganic building blocks alongside and to harmonize with the form, height and colour of the buildings, or as a formation element of an urban landscape to supplement a lack of expansiveness and depth of urban space. In other words, waterfront spaces were thought to be necessary in order to create scenes which soften a ‘dry’ inorganic urban landscape, that is to say, places for landscape interaction between the ecosystems of waterfronts and humankind. This type of urban landscape formation, although contributing to the improvement of environmental and health values, simply creates limited value which lacks diversity. Meanwhile what is required more than ever for modern cities is to create various valuable services of nature-rich waterfront spaces, thereby enhancing the attractiveness of cities for citizens and visitors who have a greater diversity of values.

Based on an awareness of these issues together with the conventional view concerning urban waterfronts, this study aimed to clarify a direction for the urban waterfront spaces of this century; through introduction of a concept of ecosystem services which views the interaction between ecosystems and humankind in an integrated manner, comparison of ecosystem services between similar cities, and review of the environmental and health value of waterfront spaces, in order to provide more depth and options. As subjects of the study, the cities of Saint Petersburg and Osaka were selected for their similarities in natural topography and conditions of location.

3. Past studies

It was during the 1980s when the decline of large cities began in the United States, that urban waterfront spaces became the focus of attention as a ‘third urban space’ which was not the area of a continent or body of water, but an area where both of these overlap. At that time, studies concerning the use of urban waterfronts mainly concerned the reconstruction of pleasant cityscapes and development methods for their formation elements with changes in land use. These major urban developments resulted from the renewal and innovation of urban functions through urban planning and reorganisation of land use, including urban redevelopment by reconstructing decrepit existing buildings, construction of new buildings on sites where stagnant secondary industry had been located, and revitalisation of old harbour districts with obsolete distribution functions. As the urban planning and reorganisation of land use primarily aimed for the renewal and innovation of urban functions while at the same time creating a new cityscape in waterfront spaces, they succeeded in raising landscape functions in some areas, but did not lead to improvement of the waterfront function of the city as a whole, nor dramatically enhance the attractiveness of waterfront spaces.

In Japan, domestic heavy industry also began to decline in the 1980s, due to the industrial advancement of developing countries, and effective use of old factory sites became a social problem. Although the regeneration of urban waterfronts drew attention, a large part of the development plan targeted the creation of venues for recreation and tourism within the city, while focusing on development methods concerning urban planning and land use. The development methods included expansion of urban facilities by responding to the needs of the time, such as water parks to raise city attractiveness and high-rise buildings and cultural facilities integrated with waterfronts. In other words, the regeneration of waterfront spaces was to develop urban facilities integrated with waterfronts through the creation of long-awaited urban spaces with high amenity and meeting the demands of tourism, while raising the usage value of land having a waterfront as its backdrop, and ultimately anticipating an increase of real estate. This contributed to a quantitative expansion of urban amenities mainly related to waterfront spaces, however it was not sufficient in terms of the interaction between nature and humankind.

Such regeneration of waterfront spaces resulted in the creation of ‘dry’ inorganic spaces and failed to create truly pleasant waterfronts, as its emphasis was placed on the creation of cityscape function. It will be necessary in the future for studies concerning the use of waterfront spaces to cover not only the conventional scope of study, which focuses on the creation of cityscape function, but to extend the creation and development of high-quality waterfront spaces which can express multifaceted ecosystem functions and services, rather than through a single approach, by focusing on the diversity of interaction between the ecosystem and humankind.

4. The ecosystem services concept

From ancient times, humankind has been a constituent of the ecosystem and benefited tremendously from it. At the same time, humankind has effected the ecosystem while modifying and altering it in some respects, in other words, humankind and the ecosystem have continued their interactive relationship. Ecosystem services are the benefits by which such relationship was reviewed from an economic perspective.

Ecosystem services are the concept proposed by the United Nations as shown in Figure 1. The Millennium Ecosystem Assessment called for by the United Nations relates ecosystem services to the welfare of humankind in order to improve the benefit of humanity. This study also quotes the UN’s concept of ecosystem services.

Ecosystem services proposed by the UN are categorized by the following four services:

Firstly, Provisioning Services; this is to provide materials concerning human activities such as livelihood and industry, including water, food, fuel, fibre, chemicals and genetic resources. They are the services viewed from the perspective of the utility value of natural resources.

Secondly, Cultural Services; this is to utilize natural resources from the ecosystem indirectly and as a background element, including spiritual value, ideas, recreation, aesthetic benefits, education, benefits as a collective, and symbolic benefits. They have favourable influences on the human spirit and intellectual activities, and are based on rare and unique values.

Thirdly, Regulating Services; this is to mitigate the risk effect that nature has on people, including the control of climate, disease and flood, and detoxification. Such risk increases in reverse, in proportion to the degree of excessive human activity and the progress of overdevelopment.

Fourthly, Supporting Services; this is the foundation which supports the ecosystem, including soil formation, nutrient cycling and primary production. Their quantitative and qualitative soundness has a great influence on the productivity and growth of the tropical natural environment.

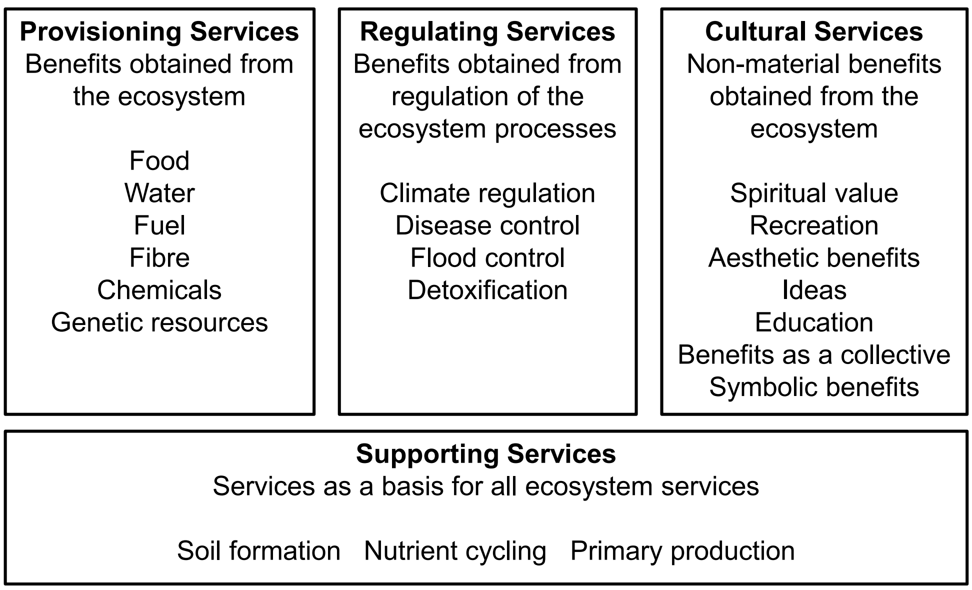
In this study, the multi-faceted nature of the use of urban waterfront spaces was considered based on the concept of ecosystem services.

5. Overview of study subjects; Saint Petersburg and Osaka

The cities of Saint Petersburg and Osaka selected as the study subjects established a sister-city relationship on 16th August, 1979, and since have actively promoted economic and cultural exchanges. Saint Petersburg and Osaka have populations of approximately 4.93 million (2011) and 2.68 million (2013) respectively, meaning that the former has about 1.8 times the latter. With respect to the city areas, Saint Petersburg is 1,400 km^2 and Osaka 221 km^2, meaning that the former has an area about 6.3 times that of the latter (Fig. 2).

Despite the above different social conditions, the two cities are similar in natural conditions and both have a history as harbour cities. Regarding natural conditions, Saint Petersburg is situated in a delta region at the mouth of the Neva River originating in Lake Ladoga, the largest lake in Europe, with Neva Bay to its west. Osaka is situated in a delta region at the mouth of the Yodo River originating in Lake Biwa, the largest lake in Japan, with Osaka Bay to its west. Both the Neva River and Yodo River are 74 km long, and their natural conditions are therefore similar (Fig. 2). In addition, both cities have a prevailing strong westerly wind in winter, due to the relative position of the bay and city.

Thus both are harbour cities located at the mouth of rivers. Saint Petersburg was founded as the imperial capital by Tsar Peter the Great in the early 18th century, and has since flourished as a harbour city through overseas trade. Osaka developed in the 17th century and further flourished as a harbour city serving as the base port for water routes for domestic trade. Experiencing high waves caused by westerly winds prevailing in winter due to their locations at the mouth of rivers, it was difficult for the cities to dredge their harbours to secure the water depth to counter the frequent berthing and unberthing of ships and



Note: Cited from Seibutsu Tayosei wa Naze Taisetsu ka (Why Is Biodiversity Important?), edited by Toshitaka Hidaka (2010)
Fig. 1. Ecosystem Services

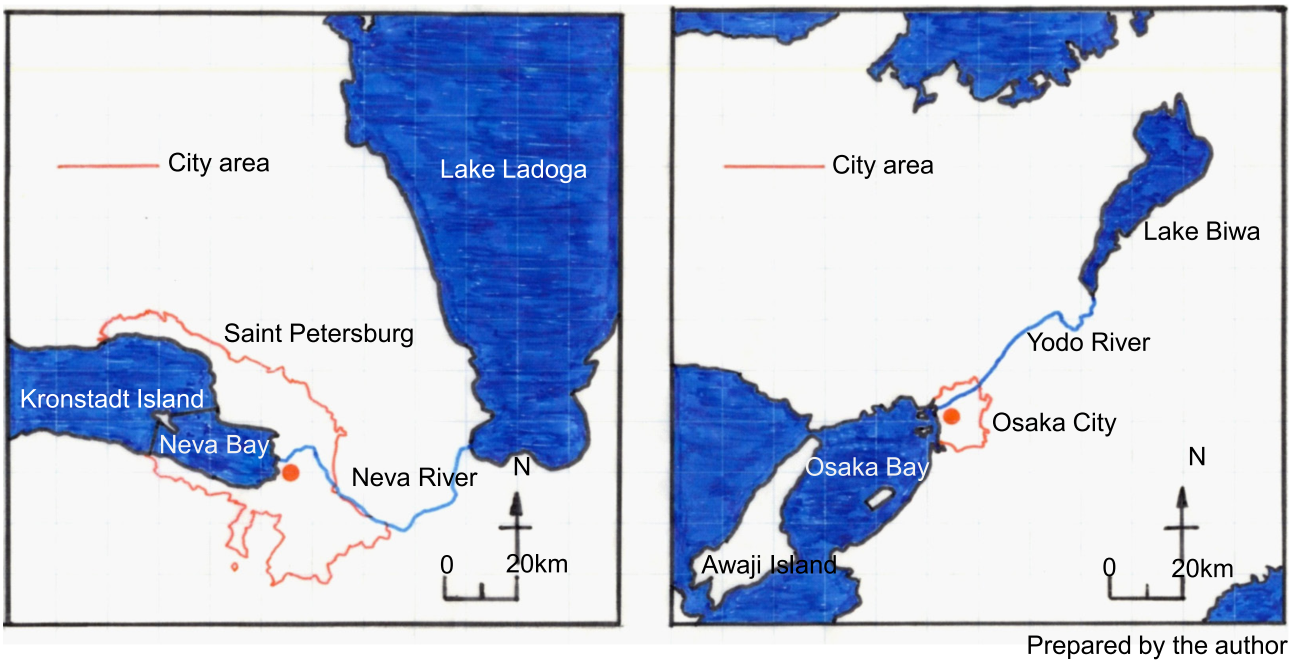


Fig. 2. Location of Saint Petersburg and Osaka

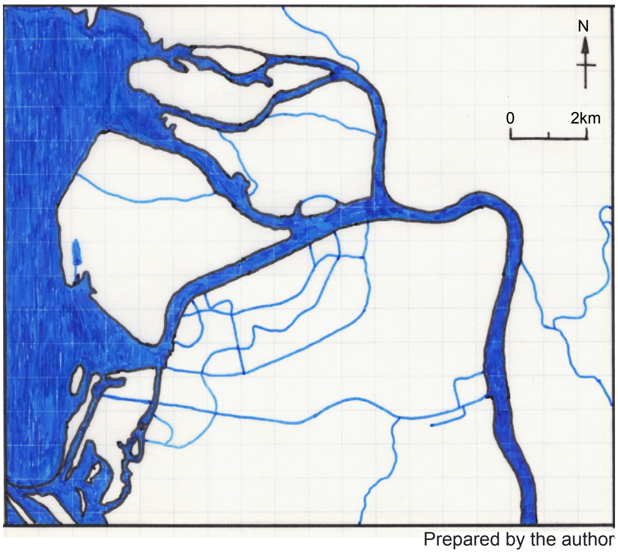


Fig. 3 Main river and canal network in Saint Petersburg

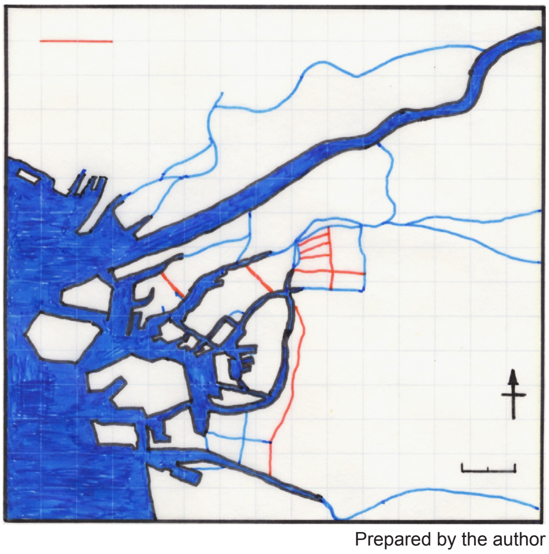


Fig. 4 Main river and canal network in Osaka

the deposit of a large amount of sediment from the rivers. Therefore it was disadvantageous for them to build harbours at the mouth of rivers, and instead harbours were built upriver. To respond to the increasing demand for water transportation, canals were built to connect these rivers, thereby providing better access to the city centres. The current river and canal networks are as follows (Figs. 3 and 4). Some of the canals in Osaka were filled and became roads in order to relieve the traffic congestion that was worsened by motorization during the 1960s (Fig. 4). The river and canal networks which once played a major role for water transportation in the two cities are now drawing attention as valuable waterfront spaces. Further, it is anticipated that such spaces will generate diversity for their use from the additional perspective of ecosystem services.

6. Consideration of waterfront space use by the sister cities

A waterfront is an attractive space which allows us to have contact with nature i.e. water and living creatures. Therefore, for a large densely built city of today, whether it has a waterfront space or not determines the degree of its attractiveness. Being sister cities and similar in terms of location, both Saint Petersburg and Osaka are metropolises blessed with such waterfront spaces consisting of rivers and canals spread through their city centres.

Presently, these spaces in both cities appear to be regarded as resources for tourism to attract visitors. However, although they serve as important assets for tourism in large cities, it is also necessary to develop them as high-quality symbiotic spaces between waterfronts and humankind, by re-examining them as one of the urban resources benefitting those who live there. To this end, introduction of the ecosystem service concept becomes effective. In this study, representative examples of waterfront use in Saint Petersburg and Osaka are considered from the viewpoint of ecosystem services in four locations; the tourism district, city centre, the district near the mouth of the river, and the mouth of the river.

6.1. Waterfront use in the tourism district

Griboedova Canal and Dotombori Canal were selected as representative examples in the tourism district of Saint Petersburg and Osaka respectively (Photos 1). In these districts, sightseeing boats shuttle along the canals, and tourists enjoy the townscape and waterscape on the banks from the boat with a lower viewpoint than ground level. With respect to the difference between the canals of the two cities, Griboedova Canal has a constant space on both sides between the bank and buildings where tourists and vehicles can pass. Dotombori Canal on the other hand has no such space as the buildings extend to its banks, and temporary pedestrian walkways are provided along both banks. However, in terms of the distance between the water surface of Dotombori Canal and tourists, it is closer than that of Griboedova Canal.

Considering the use of waterfront spaces in the case of the two canals from the perspective of ecosystem services, the canal water and its underwater life provide an intangible service to relieve the mental stress of tourists. At the same time, tourists apprehend the improvement of water quality and habitat of the underwater life. This will result in tourist's requesting a maintainer for the canal through the Internet or other means, and the maintainer, in response, will act upon the ecosystem. In this way, an interaction between the ecosystem and humankind is generated.

6.2. Waterfront use in the city centre

A part of Griboedova Canal, downstream from Lion Bridge, and a part of the Dojima River, close to the business quarter were selected as representative examples in the city centre (Photos 2). In these areas, sightseeing boats and pleasure boats shuttle along the rivers and canals, and tourists as well as citizens enjoy the cityscape from the boats. As mentioned above, Griboedova Canal has a constant space between the banks and buildings facing the canal which is designed to allow people and vehicles to pass. While the Dojima River has a pedestrian walkway in some sections on both banks. A road was also built along the Dojima River in some parts, but one cannot see the river from the road. In addition, an urban motorway was constructed above and along the Dojima River which spoils the cityscape, and affects the environment of the river's ecosystem.

With respect to the use of waterfront spaces of the canal and river, having roadside trees along its banks, Griboedova Canal has an apparent ecosystem comprised of water, underwater life and vegetation. Buildings with uniform heights also stand along the canal. As an ecosystem is created in such a way, a cultural landscape service is provided here. On the other hand, in the case of the Dojima River, with the urban motorway running along it, and the height of buildings being inconsistent, it provides no cultural landscape service. This is due to the priority placed on intensive land use, and it will be essential in the future to act upon its ecosystem to create ecosystem services by for example increasing the number of trees.

6.3. Waterfront use in the district near the mouth of the river

The Fontanki River and Tosabori River were selected as representative examples in the district

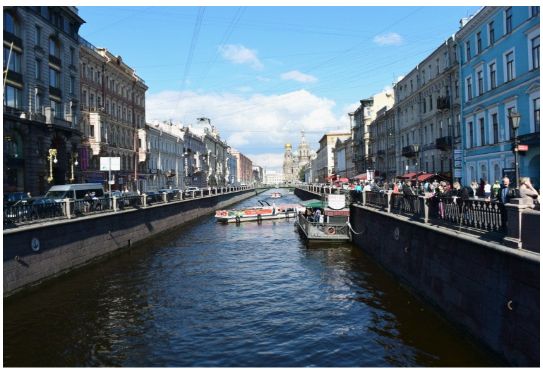


Photo by the author

Griboedova Canal

Photo by the author

Dotombori Canal

Photos 1. Waterfront use in the tourism district

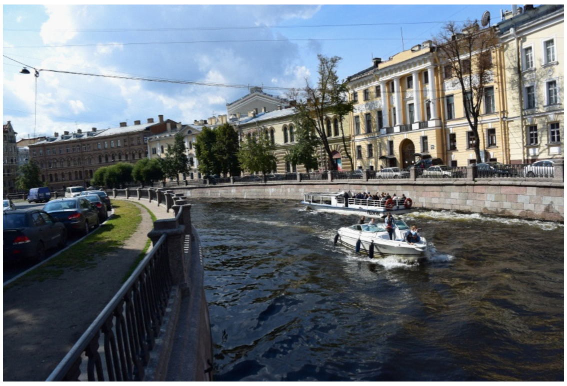


Photo by the author

Griboedova Canal

Photo by the author

Dojima River

Photos 2. Waterfront use in the city centre

near the mouth of the river (Photos 3). In these districts, one finds small transport boats going by and mooring as they are near the sea. The Fontanki River has business related and residential buildings on both its banks. Likewise the Tosabori River has such a mix of buildings, except there are more high-rises compared with the Fontanki River. On the banks of the Fontanki River, there are not only roads for people and vehicles but also docks where people can access and have direct contact with river water. On the other hand, the Tosabori River has no docks and people can only enjoy the view of water from the road. This is because of the high revetment built for flood prevention.

In the case of the Fontanki River, people can benefit from ecosystem services directly from the river, as they have access to it, while in the case of the Tosabori River, people indirectly benefit from ecosystem services as all they can do is to view the river. However, as such indirect ecosystem services can also be beneficial to those living in high-rises on both sides of the river, its services are wider ranging. In the future, the Tosabori River should be improved so that people can have access to it, and directly benefit from its ecosystem services.

6.4. Waterfront use in the mouth of the river

The Neva River and Aji River were selected as representative examples in the mouth of rivers (Photos 4). Situated in brackish-water regions, these mouths of rivers are blessed with rich ecosystems. The mouth of the Neva River enjoys a magnificent landscape with bridges connecting to historical monuments and Vasilevskiy Ostrov Island, and has heavy marine traffic with tourist boats and cargo vessels. On the other hand, at the mouth of the Aji River, the Osaka city-run ferry connects both banks for the convenience of people's travelling, as there is no connecting bridge. It has a heavy traffic of cargo vessels as there are a large number of port related facilities on its banks. There is also tourist boat traffic as it serves as part of a tour route. It also features high-rise hotels built in connection with a large-scale theme park, Universal Studios Japan, and high-rise apartment buildings.

In the case of the Neva River, people can benefit from ecosystem services from short and long distance views centred on the Neva River, from either its banks or tourist boats. For this purpose, there are many jetties for tourist boats on the banks

Photo by the author

Fontanki River

Photo by the author

Tosabori River

Photos 3. Waterfront use in the district near the mouth of the river and lots of tourists enjoy the tour around the mouth of the river, while benefitting from ecosystem services. In the case of the Aji River, people likewise enjoy, except that they can experience ecosystem services of the river more closely from the ferry crossing the river. Also, the guests of high-rise hotels and residents of high-rise apartments indirectly benefit from ecosystem services afforded by the spectacular views from where they stay or live. Taking advantage of the fact that the mouths of both the Neva River and Aji River are situated in an ecologically diverse brackish-water region, it will be important to actively promote water quality improvement, greening and the provision of places to make contact with the rivers, thereby allowing people to directly benefit from quality ecosystem services.

6.5. Summary

As above, waterfront use in Saint Petersburg and Osaka, as the study subjects, was qualitatively analyzed and considered from the perspective of ecosystem services. A summary of the results obtained through these considerations is as follows:

Firstly, the rivers and canals in Saint Petersburg retained stone banks and docks in a condition close to the original. In Osaka, they were also retained, but steel and concrete were widely used, resulting in a rather flat and not ecofriendly structure.

Secondly, the rivers and canals in Saint Petersburg have little impact from motorization, while in Osaka, several canals were filled and turned into roads due to motorization. As a result, places for the ecosystem were destroyed, thus losing the possibility of benefitting from its services, which in the end accelerated the city becoming more artificial.

Thirdly, in Saint Petersburg, there are pavements and roads between rivers/canals and buildings, while in Osaka there are no pavements or roads as buildings extend to the banks of rivers and canals. Currently pedestrian walkways are constructed on the river banks or canals, in order to benefit from their ecosystem services.

Fourthly, the rivers and canals in Saint Petersburg are accessible for people to make contact with the water, while in Osaka, entry to rivers and canals is limited to prevent people from having accidents through contact with the water. This at the same time reduces an opportunity for people to benefit from ecosystem services and negatively impacts on people's approach toward rivers and canals.

Fifthly, it is essential to regard and plan the use of waterfront spaces from a comprehensive viewpoint. To this end, it is vital to review such spaces in relation to tangible and intangible ecosystem services and to quantify such relations.

7. Conclusion and tasks

The use of waterfront spaces is an important study subject for urban design and planning. However it has been predominately planned based on design elements such as form, colour and relative distance between waterfront and buildings. When recalling the fact that humankind is a part of the ecosystem, one has to take an integrated view of the use of waterfront spaces by giving deep consideration to the tangible and intangible relationship with the ecosystem. Waterfronts are valuable legacies as ecological spaces with the unity and continuity found in large cities. The effective use of such spaces represents an important issue with respect to the sustainability of metropolises.

In this study, qualitative comparative analyses and consideration were carried out from the viewpoint of ecosystem services on the present rivers and canals of Saint Petersburg and Osaka. This paper suggests the quantitative and qualitative significance of ecosystem services in order to enhance the attractiveness of the use of waterfronts. In addition to increasing the appeal of tourism resources, this can include contributions to an increase of environmental value and the incidental rise of economic value of real estate assets, reduction of public health and medical expense such as relief of mental stress and prevention of sickness.

Photo by the author

Neva River

Photo by the author

Aji River

Photos 4. Waterfront use in the mouth of the river

As for key future tasks, it will be necessary to advance experimental studies which enable more concrete analyses, by quantifying ecosystem services related to the use of waterfronts, while for example studying the introduction of indices. Furthermore, ecosystem services involve not only the ecosystems of rivers and canals in cities, but also for example Neva Bay and Osaka Bay. What one can do to solve this is to expand the scope of spaces covered, while specifically clarifying ecosystem services related to the use of waterfronts including seashores, that is to say, the relationship between ecosystem services and humankind.

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