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Yuryk Ya. M.

ESTHETICS AND ETHICS OF THE DEVELOPMENT OF THE ARCHITECTURAL ENVIRONMENT OF A CITY

Lviv Polytechnic National University

Introduction

In terms of integration into the global and European space, the priority is to preserve the architectural environment of a historical city. Historical and cultural heritage of the city materializes and reflects the multicultural memory of towns, as well as it creates optimal conditions for a healthy and safe life of their inhabitants. Nevertheless, the priority direction of the cities' functioning is their sustainability.

Analysis of recent research and publications. The existing international instruments relating to the principles of modern urban development are analyzed. The current state of the research of the aesthetic phenomenon in the architectural environments is clarified. Some aspects of the covered subject are partly highlighted in the works of A. Hriunbakh, V. Glazychev, Ya. Geyl, A. Ikonnikov, K. Lynch, A. Marder, E. Shvydkovskyi, S. Hasiyeva, V. Shimko, N. Shebyk, S. Chubovich S. Lishaieva, M. Mihaylov, I. Rozelson, V. Timohin, Z. Yargina, Ya. Yuryk.

The aim of the study is to examine the architectural environment of a city in the context of its aesthetics and ethics of the life of its inhabitants and the activity of the architects and designers, as well as the decisions on management of a city by its leadership.

Formulation of the problem. The fundamental challenge for the government while making the decisions concerning management, first of all, of the historical city had to be the creation of a qualitative living environment for the life of residents. The identification and evaluation of the aesthetic potential of the cities had to be a priority during the execution of the tasks regarding the management of spatial development of a city and in the contemporary architectural practice.

Annotation. This study revealed features of the preservation of memory in the architectural of the city the context of its aesthetics and ethics of the life.

Keywords: *architectural environment, the city, perception, esthetic, ethics, memory, the memory.*

Discussion main material. B. Tymokhin point out that: "the opinions on the integration of a space-time organization of the historical city have yet not been found, the aesthetic doctrine of the urban environment has not fully been formed" [1]. A common problem for the inhabitants of modern cities, apart from the environment quality problems, safety, accessibility, ecological problems, is the degradation problem and the loss of social and cultural functioning of a city space as "meeting places and communication for the residents"[2].

It is known that modern cities should operate on the principle of sustainable development, taking into account the socio-cultural dimension of constant development. There are principles of "sustainable development" of cities defined in the In Rio de Janeiro Declaration on the Environment and Development [3]. "The vision of the sustainable development of human settlements" that will allow to



maintain health, safety and happiness of the residents is confirmed in the Istanbul Declaration. [4]. The city for all" should be safe and beneficial to health, it should preserve local conditions and bring people together – these are the main provisions of "the New Athens Charter" which specify the city direction. However, the problems of formation of aesthetic architectural environment of a city are also the priority [5]. "Emotional hunger" is felt by the inhabitants of modern cities, which became a global phenomenon, so as to say, in today's typical building system. «Емоційний голод» відчують мешканці сучасних міст набув глобального масштабу набуло явище т. з. в умовах сучасної типової забудови». This is attributed to the growing interest in "... aesthetic evaluations of the urban environment, the development on the basis of aesthetic design programs" [6, с. 57- 77].

The city aesthetics is a continuous connection between a man and nature. The evolution of these relations of "the opposition of a city to the nature of the ideas of cooperation, the concepts of "the ethics" of the relationship of a man with nature, "the confluence" [3; p.13, 16] with the nature, led to the modern recognition of the priority of environmental approach in the architectural environment of a city.

For example, the Spanish city of Vitoria was recognized as a European "green capital" in 2012, which is a perfect example of sustainable development. A "radical" ecological and planning transformation was implemented in its area [7].

"Another form of urban mobility model" was fixated in the Manifesto of the European Charter of the Cities II [8]. A great importance in this document was also paid to socio-cultured and informational aspects of the urban environment. In particular, it has been stated that "social dimension" is the constant factor of sustainable development [8]. Architectural environments of different city groups are characterized by different socio-cultural potentials [9], which are not the same in the different periods of their development, thus, with a primarily different aesthetics of the environment and ethics of the activity in these spaces. It is necessary to note that only the integrity perception of a person includes the overall historical context giving it a member status of a community [10]. A modern man is "a free worker" seeking "his or her place in life" [11]. "The aesthetic meetings with the space" become open for him or her, "the aesthetic perception...opening the landscape" is revealed [11]. Furthermore, namely the aesthetic perception and of the in-groups and the out-groups of this architectural environment are held at various levels with varying degrees of importance to each of them.

The city leaders must identify and take into account the social and cultural potential of a city. It should be taken as the basic criterion of development. These actions make it possible to eliminate "the contradiction between the parameters of the urban environment and the needs of people" [12, с.71].

Conclusion.

The solution to any problem is impossible without its awareness at different levels and implementation of scientific analysis.

The paper highlights the phenomenon of sustainable urban development in the context of aesthetic pleasure and the environmental needs of the in-groups and the out-groups in its environment, and also while making the decisions about managing the space development of a city.



The creation of the environment "of high quality" requires a change in the social consciousness, in the ethical environment by the architects and designers creating a culture of environmental awareness and the inhabitants of modern cities.

It should be noted that the creation of a "perfect – of high quality", aesthetically pleasing and functional architectural and spatial environment of a city is an ongoing process. Its priority should be in the strengthening of the interests of a community, environmental approach and ethical management activities.

To save "the aesthetic memory" and the formation of environmental, ethics and aesthetics are the important tasks that require a multidisciplinary approach for their solution.

"Places of Memory" need further study and marking in the space.

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**J11604-002**

Tarasov V.V., Iushin P.S., Tarasov O.V.
THE MECHANISM FOR ACTIVE PROTECTION FROM ICE LOADING
OF SEA OIL&GAS PLATFORMS WITH USING PNEUMATIC STRETCHY
CAPSULES

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Main existing methods of Offshore Oil&Gas Structure (OOGS) active protection from the impact of ice floes have been shown, newly developed methods of their ice protection deploying soft pneumatic inflatable capsules (SPIC) have been dwelled upon, design was made in CAD Solid Work software.

Keywords: ice-resistant offshore platforms, methods of protection from ice, soft inflatable capsules.

Active protection methods imply initial impact of ice on special devices located in the vicinity of or directly on the structure to be protected, followed by the contact of already broken ice with the structure [1,4]. Hence the ice drive force onto the structure is several times less, while the character of the ice impact is changed. Measures minimizing the drive force or eliminating the impact of ice on a structure are as follows:

- use of ice-breakers to destroy ice floes;
- trenching (ditching) in the ice covering the structure's surroundings;
- deployment of specialized devices to have the ice destroyed (devices are mounted for example on the monopod offshore platform's support casing column , or on the platform base legs);
- use of directed explosions for ruining ice floes and ice jams;
- heating the structure parts in the way of ice contact for the purpose of melting it and reducing its solidity;
- deployment of pneumatic or hydraulic devices for pumping some hot media (well head gas, air, steam, exhaust gases, water) beneath the ice cover around the structure;
- deployment of drillships to have the ice cover destroyed by their rolling.

Of all the above mentioned methods of active protection from the impact of ice floes it is the use of ice-breakers that has proved to be most commonly used and reliable [3]. To have the ice drive force reduced there's a wide deployment of making trenches or ditches in the ice cover. Thus with a circular in plan artificial island or structure in the surrounding ice cover they make a network of circular in plan and radial cuts. With ice striking the island, the booming of ice takes place primarily along the system of cuts, being the weakest points, with the resultant decrease in overall drive force. There are both wet (blind) cuts and wet (through) cuts. In case of dry cuts a trench is

made so that its icy bottom prevents from water entering the cut (and the ice surface). In case of wet cuts trenches are through ones (till water). Water in such a



trench freezes from the ice surface downwards with the speed of ice formation in thickness from 15 cm to 30 cm a day [1,4]. Cuts are made both vertically and inclined. There may also be made a combination of dry and wet cuts.

Use of directed explosions is generally considered to be a reliable method of protection, but these involve damage to the environment, sometimes personnel of platforms need to be evacuated, and so on.

With the purpose of reducing the ice drive force on structures there are practical cases of heating the structures in points of their contact with ice. The recommendations in this case are to use exhaust gases from gas

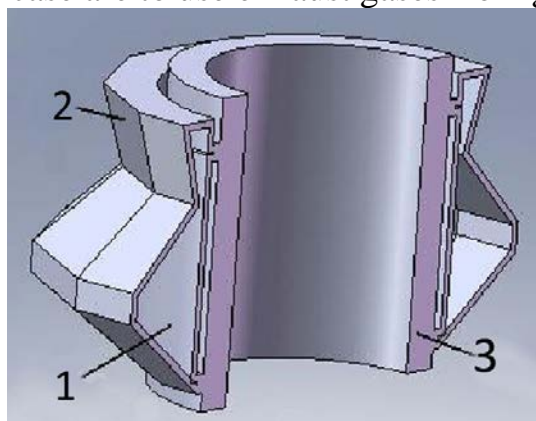


Figure 1.1 Diagram of a bandage vertically moving along platform base legs, where 1- ballast compartments, breaking ice before the platform base leg; 2- compartments with pumps and water heating; 3- bandage body. turbines, which heat is to be transferred to heat transfer medium - sea water with ant freezing agent [2]. The heat transfer medium in its turn should heat up corresponding structure parts.

There is some progress in deploying pneumatic or hydraulic devices for ruining the ice cover. There is a practice in using pneumatic method of ruining ice with air jets from perforated pipes laid down on the bottom. The rest of the methods are under research and development [2,3].

1. Devices of special design for ice booming.

A thorough consideration should be given to the issue of deploying devices of special design for ice booming. Most commonly regarded are anti-ice protective devices of bandage type (barriers, booms, etc.) mounted on the platform's support casing column, or on the platform base legs within ice-exposed areas. The bandage itself is usually a steel pontoon divided into compartments inside and circular or multifaceted in plan and with inclined (conical as well) sides for booming the ice. The pontoon may be flooded with a hot agent (air, steam, water, etc.).

One of well-known technical proposals [3] of Norwegian and other experts in the field of active ice protection of platforms with base legs is arranging ice-breaking bandages on base legs in the way of the waterline (Fig.1.1, 1.2). Each bandage is weighed to the wires of winches established in the above-water part of the platform, and is free to move in vertical direction. Outside bandage surfaces are inclined so that the ice may destruct both with bandages moving upwards and downwards.

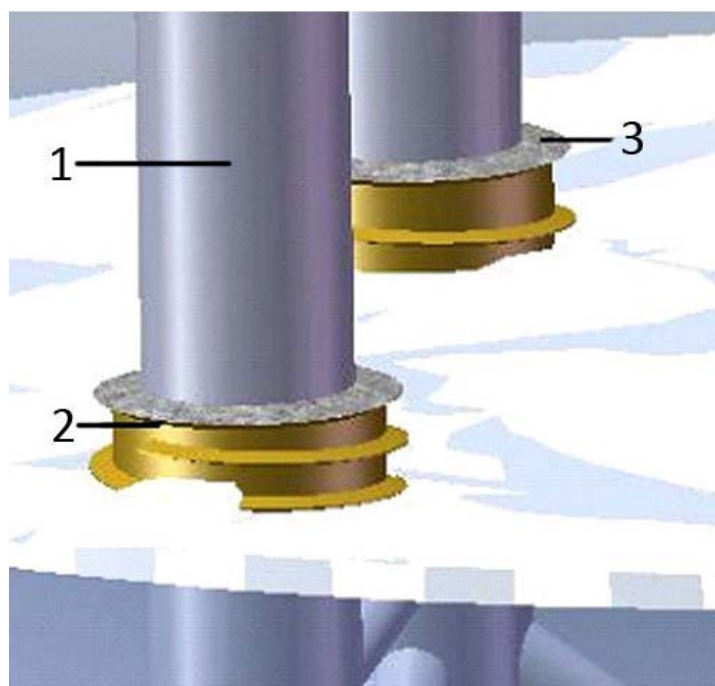


Figure 1.2 Diagram of ice-breaking rotating bandages on a base leg of a multileg platform, carried out in Solid Work software: 1-base leg; 2- helically-cammed rotating body with thread; 3- helical ice-cutting part of the body, breaking the ice.

A bandage of a gravity type has a considerable own weight, so when moving downwards it rots the ice by its gravity (like ice-breakers do it).

Vertical moves of the bandage can be provided for by several gear, and, in the first head, by winches. Furthermore, the upper part of the bandage contains ballast compartments to be ballasted with preheated water and then to be blown down. Heating the ballast water would prevent bandage freezing on ice. Also, vertical moves of the bandage can be provided for by installation of a hydraulic drive inside the bandage. The base leg in the way of the ice-breaking bandage is strengthened, while the bandage itself would move along the roller guides. It is obvious that the ice-breaking bandage can be positioned within the waterline area only at times of ice movements. Otherwise the bandage to be in a raised position by winches at the platform above-water structure and not to be exposed to ice and waves.

There is another known proposal active ice protection of platform base legs is by means of rotating ice-breaking bandage (Fig.1.2). The latter is pivotally secured from downside to the seating ring on the base leg and is free to rotate. The bandage body has icebreaker lines therefore the ice is broken in a similar manner as it occurs when an ice-breaker proceeds. The bandage may also be fitted with ice-booming arrangements, such as drilling machines, ice-cutting

rotary tools, hot gas or liquid feed-up, etc. The bandage inner space is divided into watertight ballast compartments, filled up with heated water (heated up to prevent freeze up). Compartment ballasting is controlled.

Figure 1.3 Model of ice-cutters of screw type on the support casing column of monopod platform, carried out in Solid Work software: 1-fixed monopod; 2-screw structure rigidly fixed on the monopod for cracking the ice.



To have the bandage moving in a vertical direction, hanger cables to winches, hydraulic drives, compartment ballasting and blowing up, etc. can be used. A scheme of rotating bandage is on the platform base legs. Some inventors [3] proposed active protection of base legs of a multileg platform by ice-breaking bandages in the form of truncated cones with opposing tops (Fig.1.2, 1.3). The bandages are located on the base legs, are capable of rotating in plan and simultaneously of moving vertically. Such movements of bandages should be secured by drives of some kind. When in contact with ice the bandages rotate in opposite directions and should make opposed vertical movements. As a result the ice floe is subject to bending strain with fractures, as well as to twisting effect. Fragments of ice are then supposed to leave the vicinity of the structure, thus excluding ice hummock formation. This idea of active protection seems to be of interest. To have it put to practice it is necessary to continue designing work as to the bandage dimensions, their motion parameters relating to specific ice condition [2,5].

Some other proposals of interest which are known both abroad and at home concern arranging cylindrical bandages with helically-cammed outer surface on platform base legs (Fig.1.2). The bandage is driven with an electric motor, and while the ice is destroyed, horizontal and vertical load components take place. From upwards and from

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Cheremnykh A.A., Kleveko V.I.
CHOICE OF RATIONAL FOUNDATION FOR CONSTRUCTIONS
WITH GREAT HORIZONTAL LOADS

Perm National Research Polytechnic University,

Введение.

This work covers two constructions of shallow foundation for a construction with significant horizontal loads on a weak basement. The first design is a classical foundation on a natural basement, calculated by an analytical method in compliance with special documents of Russian Federation, and the second is a foundation with a reinforced foundation pad, calculated according to European standards. The investigation showed that the usage of reinforcement in soil is an effective method to improve bearing capacity of the basement, and it also shows the necessity to use reinforced ground constructions in the geotechnical conditions of Perm.

In the world, the trend to growth of the cities is monitored, because of the high rates of population growth. Building, in most cases, carried out in certain areas, where favorable soil conditions are not always found. Therefore, their improvement is an important task.

There are many effective solutions of this problem. For example, the use of a reinforced foundation pad. Such construction is a reinforced earth structure with directionally oriented reinforcement. In this structure, the soil with low bearing capacity is replaced [1-5].

Nowadays, there is no standard method of calculation for the bearing capacity of such a reinforced soil construction in Russia. That is the reason why we have to use European standards, but they are not adapted to the calculation rules of the Russian Federation.

This article compares two solutions of the foundation design for the construction of the mast. Initial data for calculation are specific soil conditions (tab 1) and the loads on the foundation in the top level: $N=18,63$ kN, $Q_x=8,14$ kN, $M_x=84,34$ kNm.

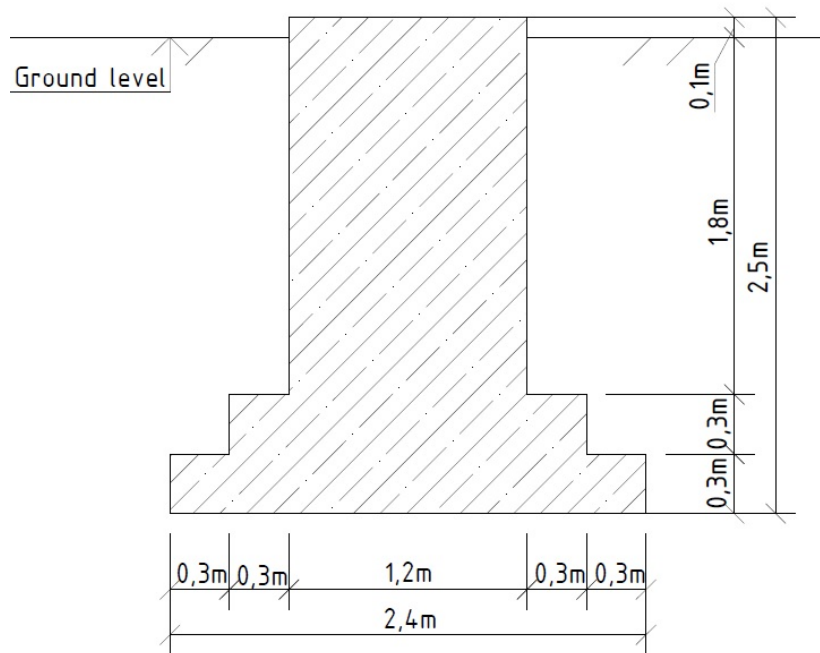
Table 1

Characteristics of soils

Layer	Depth, m	Soil density, g/cm ³	Angle of friction, degree	Specific cohesion, kPa	Modulus of deformation, MPa	Name of the soil in accordance GOST 25100-2011
1	1,5	1,97	22	47	30,4	Semisolid clay
2	8,0	1,9	15	11	2,7	Flowable-plastic clay loam
3	12,4	1,97	22	22	9,3	Tight-plastic clay loam



The first type of construction is a classic shallow foundation (fig. 1). The special documents of the Russian Federation have been used for the analysis of the foundation. The calculations were made by an analytical method. First, the foundation was calculated on the bearing capacity and then on the deformations.



Picture 1. Scheme of a classic shallow foundation

The analysis of bearing capacity included check the condition (1)

$$F \leq \frac{\gamma_c F_u}{\gamma_n} \tag{1}$$

The calculation was made on the adopted size of the foundation. The value of the design load (F) equals 355,36 kN. The value of the limit force of resistance base (F_u) – 2104,66 kN, the value of this force, taking into account coefficients equals 1647,13 kN. Consequently, the condition was met.

In calculating on the deformations the results were: value of settlement – 6 mm., value of tilting– 0,0103.

The second type of construction is a foundation with a reinforced foundation pad (fig. 2).

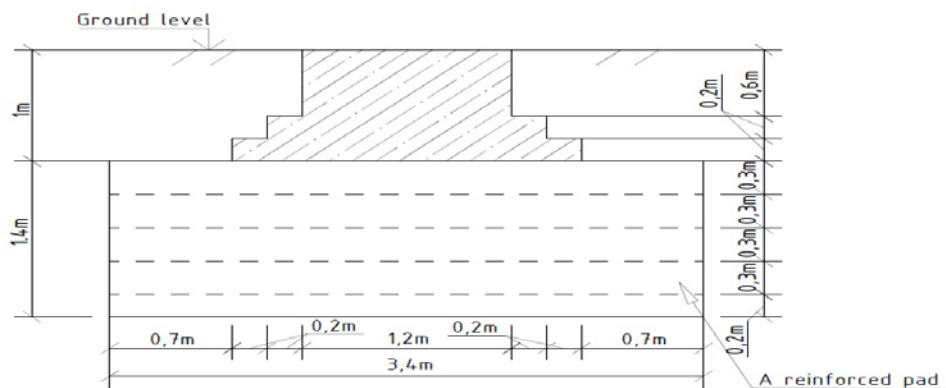


Fig. 2. Scheme of a classic shallow foundation with a reinforced foundation pad



The analysis was based on the idea that the stability of reinforced foundation pads can be analysed using the same failure model as conventional foundations. This theory was supported by observations on small-scale tests. The effect of the stronger fill soil compared to the existing ground was taken into consideration using corrected bearing capacity coefficients in the bearing capacity analysis.

For the calculations adopted the fill soil (a mix of sand and gravel) with characteristics: volumetric weight of the soil – 18 kN/m³, angle of friction – 40,0°, soil cohesion – 0,0 kN/m².

The following analysis shall be performed as part of the dimensioning process:

- bearing capacity failure, unreinforced case (on the first group of limiting states);
- bearing capacity failure, reinforced case (on the first group of limiting states);
- sliding failure, reinforced case
- position of resultant (eccentricity);
- settlements (on the second group of limiting states);
- tilting (on the second group of limiting states);
- failure of the geosynthetic reinforcement.

Reliability against failure of the soil a reinforced foundation pad must be calculated with the correction factors k_b , k_t , k_c . These factors must be multiplied by the dimensionless bearing capacity coefficients N_b , N_t , N_c of the ground. The

$$k_b = C \times k_{b,\delta} + 1, k_t = C \times k_{t,\delta} + 1, k_c = C \times k_{c,\delta} + 1 \quad \begin{array}{l} \text{correction factors} \\ k_b, k_t, k_c \text{ are given} \\ \text{by (2)} \end{array} \quad (2)$$

where $C = 1,0$ is adopted for fill soils with friction angles, whose values greater than or equal to 40°. Then the coefficients are equal: $k_b = 3,06$, $k_c = k_t = 1,57$.

The correction factors $k_{b,\delta}$, $k_{t,\delta}$, $k_{c,\delta}$ are taken from graphics. They are necessary for finding the parameters: slip plane angle of failure wedge and theoretical pad thickness for a load inclination. In this calculation, they are equal to 2,06, 0,57, 0,57 respectively.

Reliability against destruction of soil of the reinforced foundation pad was proved according to a condition (3)

$$Q'_{p,d} - V_d \geq 0 \quad (3)$$

where $Q'_{p,d}$ – bearing capacity of a reinforced foundation, which is calculated with a resultant component of efforts in the armature. This component increases the load-bearing capacity.

Further, proofs of operational suitability were provided. These include estimates of various parameters: settlements, tilting and failure of the geosynthetic reinforcement, which value was calculated by the formula (4)

$$R_{Ai,d} = 2 \cdot f_{sg,d} \cdot (V_d / b \cdot l_{in,i} + \sigma_{v,i} \cdot l_{u,b}), \quad (4)$$

где $l_{in,i}$ – length between failure wedge and foundation footprint edge in the n^{th} layer, $\sigma_{v,i}$ – stress from the ground surcharge in the n^{th} layer, $l_{u,b}$ – length of



reinforcement for foundation width b protruding over the foundation footprint.

The formula (4) was used for each reinforcing layer (tab 2)

Table 2

The results for the individual reinforcement layers

Layer i	$l_{in, i}, m$	$\sigma_{v, n}, kPa$	$R_{Ai, k}, kN$	$R_{Bi, k}, kN$
1	0,136	24,4	20,90	53,34
2	0,272	29,8	30,62	53,34
3	0,408	35,2	40,35	53,34
4	0,544	40,6	50,06	53,34

In this case the pull-out resistances of all layers $R_{Ai, k}$ are the governing factors. The total force $\Sigma R_{Bi, k}$ is therefore adopted for the following analyses:

$$\Sigma R_{Bi, k} = 20,90 + 30,62 + 40,35 + 50,06 = 141,93 \text{ kN}$$

After that, the calculation of bearing capacity of a reinforced foundation pad had been made and it showed that the strength condition is satisfied. The value bearing capacity of a reinforced pad $R_{n, d} = 197,82 \text{ kN}$ exceeds the value of the calculated $E_d = 124,23 \text{ kN}$.

Выводы:

The following conclusions were based on the calculation results:

1. The using of a reinforced foundation pad is an effective method for increasing the bearing capacity of the ground.
2. The application of a reinforced foundation pad allows to reduce the dimensions of the foundation and, consequently, reduces the cost of building.

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**J11604-004****Kleveko V.I., Shangina Y. M.****THE IMPACT OF THE USE FIBER REINFORCED SAND AS
BACKFILLING ON THE VALUE OF ACTIVE PRESSURE ON THE
RETAINING WALL***Perm National Research Polytechnic University*

The results of analytical calculation to determine the active earth pressure on retaining walls at various characteristics of the soil backfill are presented in the article. As backfilling were considered fine sand reinforced with standard strength characteristics and the same sand, but with improved strength characteristics through the use of polypropylene fiber. As a result, research has revealed the effect of sand reinforced with fibers of fiber content of 0.5%, 1.0%, 1.5% on the magnitude of the active pressure retaining wall height of 3, 6, 9, 12 and 15 meters.

Successful development of modern cities is impossible without improving the operation of their transport systems [1, 2]. Sustainable transportation systems in big cities requires the use of a wide range of artificial structures, including retaining walls [3]. Retaining walls are artificial structures designed to ensure sustainability beyond it the soil. These structures are in complex geological conditions, i.e. to perceive a significant horizontal forces from earth pressure and live loads, e.g. from transport or foundation. Reduction or complete absence of horizontal pressure on retaining walls will significantly alleviate them and to minimize the cost of construction [4].

To decrease active earth pressure is possible by improving its strength characteristics, i.e. reinforcement. Mainly to reinforce the backfill of retaining walls using geosynthetic materials [1-6]. In addition to these materials there is a technology of embedding in the ground randomly distributed discrete fiber – fiber reinforcement [5]. Fiber – polipropylene fiber, it has long been an effective reinforcing additive for concrete. This material for reinforcement of the soil is almost never used because its properties are not yet fully understood.

To explore the possibility of using vibropack, in the quality of the soil backfill of the retaining walls, it is necessary to consider the effects of its strength characteristics on the magnitude of active pressure.

The laboratory results show that the fibers of the fiber embedded in the soil, increase the strength characteristics of sandy soil [6]. The joint work of individual fibers and particles of soil contributes to the appearance of the sandy soil, the unit coupling and high angle of internal friction. This material has good filtration properties and will provide a smaller active pressure on the fence due to the increased angle of internal friction and specific cohesion.

To analyze the magnitude of active pressure on the retaining wall was considered the classical analytical method of soil mechanics].

Consider the condition of limit equilibrium of the elementary prisms is cut from the prism of the collapse of the back side of a retaining wall with a horizontal surface and a vertical back face of the retaining wall, when $C=0$. The horizontal and vertical landing pads of the prism, when the wall friction of zero will be the principal stresses



σ_1 and σ_2 .

From the condition of limit equilibrium, we obtain:

$$\sigma_1 = \gamma \times z; \tag{1}$$

$$\sigma_2 = \sigma_1 \times \text{tg}^2\left(45 - \frac{\varphi}{2}\right); \tag{2}$$

σ_2 – the horizontal earth pressure, the value of which is directly proportional to the depth z , i.e. the earth pressure on the wall will be distributed according to the law of the triangle with the ordinate $\sigma_2=0$ on the surface of the soil and $\sigma_{2\text{max}}$ at the foot of the wall. According to condition (2) lateral pressure at a depth H equal:

$$\sigma_{2\text{max}} = \gamma \times H \times \text{tg}^2\left(45 - \frac{\varphi}{2}\right); \tag{3}$$

Active pressure is characterized by an area of plot (Fig.1):

$$E_a = \frac{H}{2} \times \sigma_{2\text{max}} = \frac{\gamma \times H^2}{2} \times \text{tg}^2\left(45 - \frac{\varphi}{2}\right); \tag{4}$$

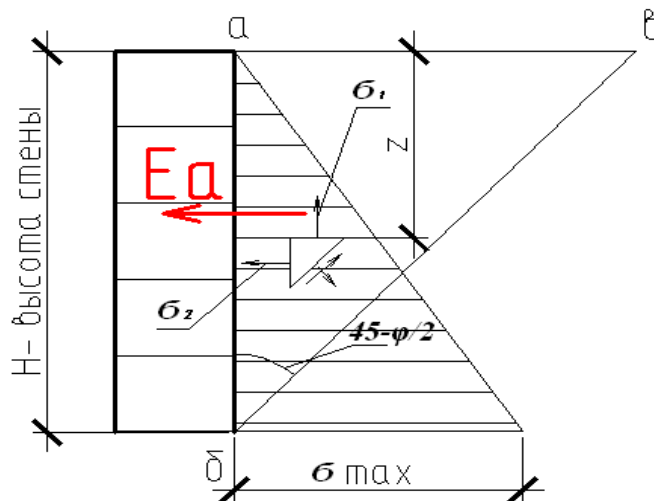


Fig.1. The scheme for determining soil pressure without specific adhesion

The conditions of limit equilibrium for soil having the same internal cohesion:

$$\sigma_2 = \sigma_1 \times \text{tg}^2\left(45 - \frac{\varphi}{2}\right) - 2c \times \text{tg}\left(45 - \frac{\varphi}{2}\right); \tag{5}$$

Comparing (4) and (5) note that expression (4) describes the pressure of a granular soil without regard to the specific cohesion, and (5) shows how decreases the intensity of pressure due to the fact that the soil has cohesion. Then this expression can be represented in the form:

$$\sigma_2 = \sigma_{2\varphi} - \sigma_{2c}; \tag{6}$$

$$\sigma_{2\varphi} = \gamma \times H \times \text{tg}^2\left(45 - \frac{\varphi}{2}\right); \tag{7}$$

$$\sigma_{2c} = 2 \times c \times \text{tg}\left(45 - \frac{\varphi}{2}\right); \tag{8}$$

Thus, the adhesion of the soil reduces the lateral pressure of soil on the value of



σ_{2c} over the entire height. It should be noted that the primer having specific cohesion is able to hold a vertical slope h_c determined by the formula:

$$h_c = \frac{2 \times c}{\gamma \times \text{tg}(45 - \frac{\varphi}{2})}; \tag{9}$$

The full active pressure is defined as the area of the triangular plot (Fig. 2):

$$E_a = \frac{\sigma_2 \times (H - h_c)}{2}; \tag{10}$$

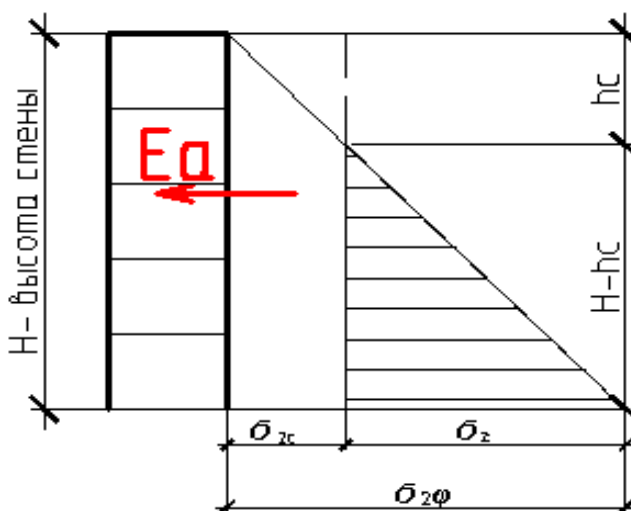


Fig.2 The scheme of determining the soil pressure considering the specific cohesion

Initial data for calculation are the characteristics of a fine sand with a specific gravity 15,74 kN/m³, and different values of specific cohesion and angle of internal friction, which was derived from laboratory tests according to GOST 12248-2010 "Soils. Methods for laboratory determination of strength and deformability".

The calculations were made for retaining walls of different heights: 3, 6, 9, 12 and 15 meters. The results are presented in the tab.

Table

The results of the calculation of the value of active pressure

The parameters of the soil	The magnitude of active pressure in kN/m at the height of the retaining wall, m				
	3	6	9	12	15
Sand (C=0, $\varphi=35^\circ$)	19,15	76,61	193,65	306,44	478,8
Sand+0.5% fiber (C =16 kPa, $\varphi=39^\circ$)	2,93	6,96	39,33	105,36	203,2
Sand+1% fiber (C=38 kPa; $\varphi=42^\circ$)	0	0	0	5,85	82,25
Sand+1.5% fiber (C=67 kPa; $\varphi=42^\circ$)	0	0	0	0	0

For the visualization of the results of the analytical calculation were plotted in Fig. 3

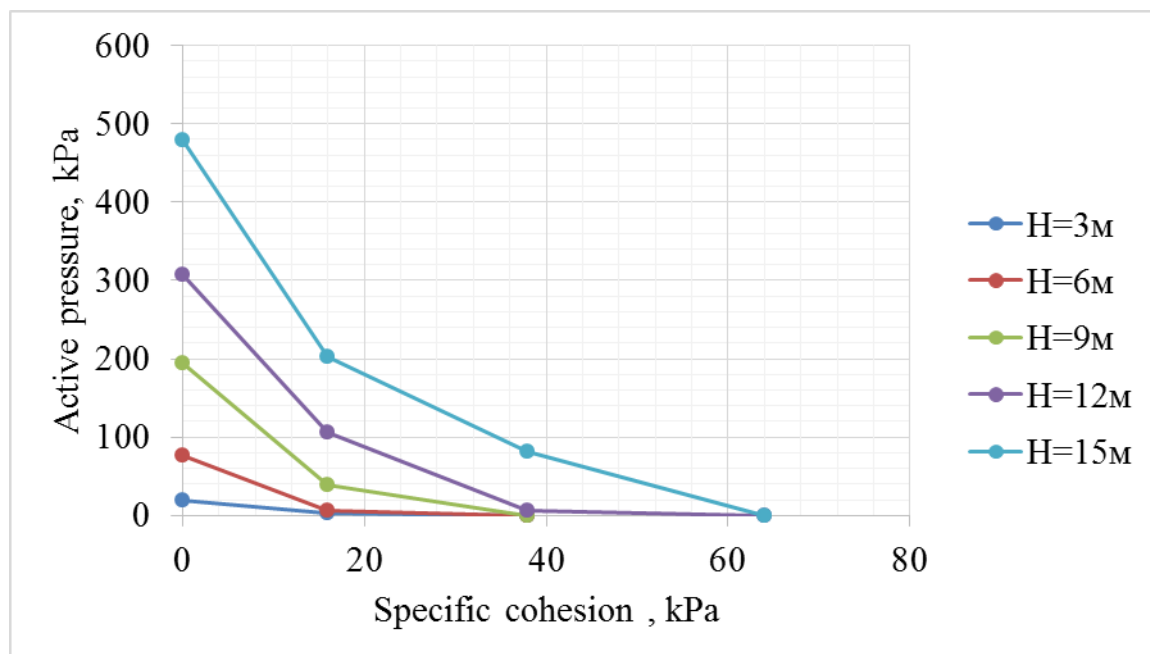


Fig.3 The graphs of active pressure from specific cohesion and the height of the retaining wall

The analysis of the graphs in Figure 3 shows that:

- for walls with a height from 3 to 15 meters when the content of fibers in an amount of 0.5% of the active pressure is reduced on average 5.5 times;
- when using sand with a content of 1% fibers, active pressure on retaining walls with a height from 3 to 6 meters is absent;
- for walls with a height of 12 meters when using sand with a content of 1% fibers active pressure is reduced to 51 times, and if you use sand with a content of 1.5% fibers active pressure is equal to zero;
- for walls with a height of 15 meters when using sand with a content of 1% fibers active pressure is reduced 6 times, and when using sand with a content of 1.5% fibers active pressure is equal to zero.

When using even relatively small amounts of polypropylene fibres (0,5% - 1,5%), the active pressure on a retaining wall is greatly reduced.

Analysis of the results of research allows to draw the following conclusions:

1. The magnitude of active pressure on a retaining wall depends significantly on the strength characteristics of the soil backfill and the height of retaining walls.
2. Joint work of polypropylene fibers and sand improves the strength characteristics of this material, resulting in a reduction of active pressure on retaining walls.
3. The use of fibre reinforced sand fibers as backfill of retaining walls allows you to build a more economical design.

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J11604-005

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ESTIMATE OF THE ERROR USING NON-DESTRUCTIVE METHOD OF DETERMINING THE STRENGTH OF CONCRETE IN VARIOUS TERMS OF CONCRETE HARDENING

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Introduction.

Nowadays concrete is one of the most popular building materials as in individual, such as in capital urban and industrial construction. Because of its wide distribution, quality control of monolithic structures plays a huge role in modern construction practice; in particular, it is determination of strength of concrete in compression. This article presents the results of experiments of strength of concrete in compression by different control methods: destructive and non-destructive. The comparison of methods and results was conducted and the conclusions on the research were made.

The development of large cities can not be imagined without better transport [1]. One of the main parts of the transport system are artificial structures – bridges, overpasses, flyovers, etc. The Main construction material which used in the construction of artificial structures, is reinforced concrete. In practice the problem of rapid determination of concrete strength often occurs. Currently the main method of operational control of concrete strength is the mechanical method of non-destructive control [2]. However, its practical application still has some problems containing the reliability of the obtained results.

Concrete is the basic and most frequently used material in construction during many years. It has many positive properties, such as reliability, durability, frost resistance. In addition, we should not forget about the economic efficiency of the use of this material.

Because of the simplicity of concrete production in our days, it is rapidly develop monolithic building. The main advantage of monolithic construction is the possibility of the erection of a building or structure of virtually any shape in a fairly short period of time. In addition, monolithic building has no seams, which significantly improves thermal insulation and sound insulation performance.

Despite all of the above, concrete is a complex material whose properties depend not only on its components, but also on manufacturing technology as the concrete, and the construction of it. In this regard, a huge role in the construction industry plays a quality control of monolithic structures.

The priority index of the hardened concrete is its compressive strength. To determine this indicator must be tested for strength by the method of destructive or non-destructive control.

The aim of this work was to test concrete for strength with destructive and non-destructive methods of control and to compare these readings.

Literature review.

Nowadays, there are some state standards, concerning the determination of strength of concrete and concrete structures, which are using on the territory of our



country. GOST 10180-2012 «Concretes. Methods for strength determination using reference specimens», GOST 28570-90 «Concretes. Methods of strength evaluation on cores drilled from structures», GOST 22690-88 «Concretes. Determination of strength by mechanical methods of nondestructive testing» are among them.

Input data and methods.

Data from the operational quality control of works during the construction of a bridge across the river Bym were used for the research. The tests were conducted on pre-fabricated monolithic cubes of nominal dimensions 100x100x100 mm, which were made in the manufacture of concrete works. Cubes were made of concrete class B25. All samples were made the same way, the solidification occurred under normal conditions. The tests were conducted at 7, 12 and 14th day after production. All were made on 12 samples.

The tests were conducted in the laboratory of the Department of Building Production and Geotechnics" of Perm National Research Polytechnic University.

The first stage of the series of experiments was a measurement of concrete strength by non-destructive methods, namely a method of shock pulse through measuring the strength of Onyx 2.5. The device is depicted in figure 1.

The strength meter is a device measuring the strength of the shock impulse and the rebound in the range of 1 to 100 MPa.



Fig.1. Measuring the strength «Onyx-2.5»

The second phase of testing was to measure the strength of these samples by the method of destructive testing with a hydraulic press PGM 500 MG4A, which is depicted in fig. 2.

The results.

The measurement by non-destructive method was carried out on 4 faces of each cube. The results are summarized in table 1. The results of destructive method are presented in tab 2.



Fig. 2. The hydraulic press PGM 500 MG4A

Table 1
The results of measurements by nondestructive control method

№	Age of concrete, days	The compressive strength of each face, MPa				The average compressive strength R_B , MPa	R_B with the factor of 0.92 on the error of the instrument ($\pm 8\%$), MPa	The average strength of series of samples, MPa
		Face 1	Face 2	Face 3	Face 4			
1	7	8,6	8,5	8,4	8,4	8,4	7,728	8,148
2	7	10,7	8,2	8,3	8,5	8,925	8,211	
3	7	8,9	10,0	9,1	9,3	9,325	8,579	
4	7	9,1	8,5	8,9	8,6	8,775	8,073	
5	12	17,8	13,9	15,0	22,1	17,2	15,824	14,932
6	12	15,3	14,5	13,9	14,6	14,575	13,409	
7	12	13,0	16,9	11,4	14,7	14,0	12,88	
8	12	17,4	19,8	22,9	16,5	19,15	17,618	
9	14	24,7	22,6	23,4	21,6	23,075	21,229	20,361
10	14	23,0	20,4	25,9	20,8	22,525	20,723	
11	14	20,5	22,0	21,9	19,9	21,075	19,389	
12	14	26,1	21,2	19,8	20,3	21,85	20,102	



Table 2

The results of measurements by destructive control method

№	Age of concrete, days	Breaking load, kN	The strength of concrete, reduced to the base size of the sample, MPa	The average strength of series of samples, MPa
1	7	60,66	5,82	5,489
2	7	60,82	5,836	
3	7	54,87	5,265	
4	7	51,43	5,035	
5	12	181,9	17,28	16,205
6	12	167,4	16,07	
7	12	161,1	15,62	
8	12	163,5	15,85	
9	14	236,2	22,90	21,543
10	14	223,9	21,48	
11	14	205,9	19,76	
12	14	227,3	22,03	

The discussions and analysis.

To estimate uncertainty methods it needs to calculate the difference of strength values for two of the above methods. The results of the calculation are presented in tab 3.

Table 3

Comparison of the experiment results

№	Age of concrete, days	The difference of the values of concrete strength by two methods, MPa	The difference of the average values, MPa
1	7	1,908	2,659
2	7	2,375	
3	7	3,314	
4	7	3,038	
5	12	-1,456	-1,273
6	12	-2,661	
7	12	-2,74	
8	12	1,768	
9	14	-1,671	-1,182
10	14	-0,757	
11	14	-0,371	
12	14	-1,928	



Conclusions.

The analysis of obtained results allows draw the following conclusions:

1. When assessing the strength of concrete at the curing period of 7 days, non-destructive test method gives overestimated values with an error of up to 48%.
2. At longer curing 14 days, and 21 days non-destructive test method gives conservative values with considerably smaller errors respectively of 8.5% and 5.8 %.

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J11604-006

Komarov D.A., Kleveko V.I.

**THE USE OF EXPRESS METHOD FOR DETERMINING OF THE BASE
DEFORMATIONS MODULE OF SOIL REINFORCED BY
GEOSYNTHETICS**

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Abstract. The paper presents the results of testing two types of soil foundation by dynamic densitometer DPG-1.2. Deformation characteristics of subgrade and base reinforced geosynthetics were investigated. The device DPG-1.2 is used to determine the static modulus of elasticity E_{st} and dynamic elasticity modulus E_d . Synthetic woven carcass mesh KS-16 production by JSC "KMMW" has been used as a reinforcing material. Conclusions on the effect of reinforcing the change E_{st} and E_d were made on the test results.

Key words: geosynthetic material, reinforced soil, dynamic densitometer, modulus of soil deformation.

Most of the territory of Perm and the Perm Kray has a complex geological conditions and is composed of soft soils, which do not allow the construction without the use of special events. One way to improve the deformation properties of the soil is different soil reinforcement geosynthetic materials [1-7]. Recently a large number of new geosynthetic materials appear on the market as a result of technological development in the chemical industry. This requires research aimed at understanding the work of the new geosynthetic materials in the construction of foundations and their interaction with soil. However, the use of standard field methods to determine the deformation characteristics of reinforced soil is very labor intensive and expensive.

One of the new methods for determining the deformation characteristics of soils is the use of dynamic densitometers [8, 9].

The results of the static modulus of elasticity E_{st} and dynamic elasticity modulus E_d soil base and base reinforced by geosynthetic using the device "Measuring the dynamic modulus of elasticity of soil DPG-1.2" are presented in this article.

The purpose of the test was:

1. Study the possibility of use of the instrument DPG-1.2 to determine the deformation characteristics of soils.
2. Determination of the influence of soil reinforcement geosynthetics to change the static modulus of elasticity E_{st} and dynamic elasticity modulus E_d .

Low humidity medium size sand was used in carrying out the test as a soil base. Its physical and mechanical characteristics were determined by laboratory method and were as follows, and they were as follows: specific gravity 16.5 kN/m³; humidity 4%; void ratio 0,65; internal friction angle 35 degrees, modulus deformation 13 MPa.

Two series of tests on three experiment in each one was carried out: first deformation characteristics not reinforced base were determined, and then these



characteristics were determined for a reinforced base. Reinforced base was arranged by laying on the ground reinforcing geosynthetic material and filling the top layer of soil thickness of 50 mm. The thickness of the backfill has been appointed on the basis of the diameter of the device stamp $d = 200$ mm. The depth of the reinforcing geosynthetic material equal $0,20-0,25d$ is optimal [10].

The device *DPG-1.2* is used for determining dynamic modulus elasticity E_d and static modulus elasticity E_{st} .

Carcass mesh KS-16 production by JSC "Krasnokamsk Metal Mesh Works" was used as the reinforcing geosynthetic material, it had the dimensions 1000x1000 mm and breaking load of 60 kN / m, at a relative elongation at break of 10%.

Measurements of elastic moduli was carried out at five points. Four points were located at the corners of the test area which has dimensions 1000x1000 mm and one point in the center. The scheme of the experimental area with the points of determining dynamic and static modulus of elasticity is shown in Fig. 1.

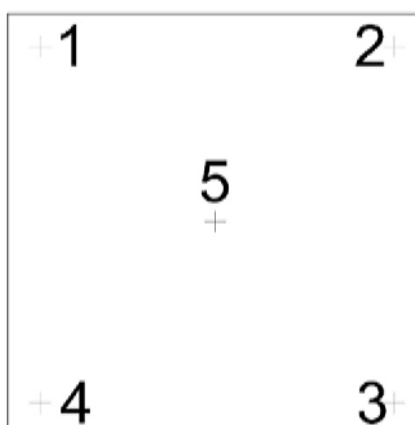


Fig. 1. The scheme of experimental area.

The experimental results obtained during the tests are shown in Table 1.

Table 1.

The results of tests

№ point	The elastic modulus not reinforced base, MPa		The elastic modulus reinforced base, MPa	
	Static, E_{st}	Dynamic, E_d	Static, E_{st}	Dynamic, E_d
1	10,7	10,6	10,8	10,4
2				
3				
4				
5			12,3	10,9
(c)				



The experimental results showed that the values of the elastic modulus obtained by express method using device DPG - 1.2 close in value to the data obtained by the laboratory method.

Static modulus of elasticity in point 5 (central) E_{st} essential increases for reinforced soil base. The corner points of the experimental area elastic modulus values have not changed, due to the lack of jamming geosynthetic material in the soil.

Conclusions:

1. The difference between the value of static modulus of elasticity of soil as determined by a dynamic densitometer DPG-1.2 and modulus deformation of soil determined by laboratory method was 17.7%.

2. Static soil modulus E_{st} increases by 13% due to the reinforcement geosynthetics soil and soil dynamic elasticity modulus E_d is increased by only 3%.

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