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Additional foundation settlements prediction considering the earthquakes repeatability

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ABSTRACT: Current practice of structural design in earthquake prone areas requires load-bearing foundation providing, and, for this reason, much attention is paid to soils subjected to liquefaction. As a rule, the foundation deformation - settlement or tilt, caused by seismic activity is continuous. The settlement resulted from the seismic impact is a secondary problem; the main goal is to avoid the structural collapse. However, the seismic events of different intensity may occur in seismic areas during the operational period of a building and result in accumulation of irreversible deformation. The total accumulated deformation from seismic impacts may exceed the static load settlement. The paper presents a method for predicting foundation deformation, taking into consideration the repeatability of earthquakes, based on expected number of impacts of different intensity. The method includes dynamic triaxial test for estimating changes occurred in soils density under seismic impact, and analytical proceeding of the data obtained. An experimental relationship between the degree of compaction and seismic load for fine sand has been received. The paper shows that the total accumulated seismic settlements of a foundation and static load settlement can be commensurable.

RÉSUMÉ: La pratique actuelle de la conception structurelle dans les zones sujettes aux tremblements de terre nécessite la fourniture de fondations porteuses et, pour cette raison, une grande attention est accordée aux sols soumis à la liquéfaction. En règle générale, la déformation de la fondation - tassement ou inclinaison, causée par l'activité sismique est continue. Le tassement résultant de l'impact sismique est un problème secondaire; l'objectif principal est d'éviter l'effondrement structurel. Cependant, les événements sismiques d'intensité différente peuvent survenir dans les zones sismiques pendant la période d'exploitation d'un bâtiment et entraîner une accumulation de déformations irréversibles. La déformation totale accumulée due aux impacts sismiques peut dépasser le règlement de charge statique. L'article présente une méthode pour prédire la déformation des fondations, en tenant compte de la répétabilité des tremblements de terre, basée sur le nombre attendu d'impacts d'intensité différente. La méthode comprend un test triaxial dynamique pour estimer les changements survenus dans la densité des sols sous l'impact sismique et une procédure analytique des données obtenues. Une relation expérimentale entre le degré de compactage et la charge sismique pour le sable fin a été reçue. L'article montre que le total des tassements sismiques accumulés d'une fondation et le tassement de charge statique peuvent être commensurables.

KEYWORDS: laboratory tests, sands, deformation, settlement, seismicity, liquefaction

1 INTRODUCTION

In comparison with other countries located in earthquake prone areas, the territory of the Russian Federation is generally characterized as moderately seismic. Pursuant to the General Seismic Zoning of the Russian Territory, 20,1% of it belongs to the zone of 7-degrees intensity, 6% falls into the zone of 8-degrees intensity, and 2% may suffer severe shaking of 9- or 10-degrees intensity within 50 years (Ulomov, 2012).

Civil and industrial earthquake resistant foundations are computed by the bearing capacity criteria only. The goal of the earthquake engineering–is to prevent structures from damage or collapse and to preserve human health and lives. Nevertheless, due to the seismic impacts, a structure may suffer such a damage, that restoration or repair of it may turn to be very expensive and impractical. One of such irreversible damages is an additional seismic deformation (seismic settlement, seismic tilt).

Foundations constructed for seismic regions are exposed to combined static and dynamic loading. If the dynamic seismic load acts repeatedly, the resulting deformation may be the same or even greater than deformation under a static load. This happens owing to accumulation of irreversible deformations caused by seismic impact of the various intensity and repeatability. The situation worsens for sand, which is much more susceptible to repeating loads than clay. According to (Seed & Lee 1966; Seed & Idriss 1971; Ishihara 1996), accumulation of irreversible deformations caused by relatively weak but more likely frequent impacts should not be neglected.

Codes of Practice do not provide any guidelines for assessing seismic settlement. The major reason for this is the lack of knowledge, uniqueness of the test procedures, and the lack of reliable methods for seismic settlement evaluation.

Recent development of the test equipment has improved seismic settlement prediction technique. Some laboratories have an advanced test equipment at their disposal for determining dynamic soil properties. The most relevant are resonant column and dynamic triaxial tests.

2 DESCRIPTION OF SEISMIC IMPACTS ON THE SOIL FOUNDATION

The principal measures of the earthquake's strength are the earthquake intensity and magnitude.

The intensity I is a degree of ground shaking estimated with a macroseismic scale at an observational point. Various scales estimating the intensity of earthquakes are currently available: MSK-64 (Medvedev-Sponheuer-Karnik scale, version of 1964), CS (Mercalli, Kankani and Ziberg scale), MM (Modified Mercalli scale), EMS-98 (European Macro seismic scale, version 1998), and ESI-2007 (Environmental Seismic Intensity Scale). All the enumerated scales show nearly the same meanings within the accuracy of measurements.

The Earthquake intensity characterizes seismic effect that reflects the reaction of people and their environment and a structural response. Three following characteristics sufficiently designate the seismic effect – amplitude level, frequency and duration of the oscillations.

Earthquake design takes into consideration peak ground acceleration *PGA*, different values of which depend on the earthquake intensity. Thus, 7-, 8- and 9-degrees earthquakes measured by MSK-64 scale respectively correspond to PGA value of 0.1g, 0.2g and 0.4g.

Magnitude is a quantitative measure, which characterizes the energy released during seismic wave propagation. There are a number of Magnitude scales available for measuring earthquake strength nowadays (Voznesenski, 1998):

-The Local Magnitude M_L describes earthquakes with epicentral distances less than about 600 km and with the focal depth less than 30 km. This scale is rather useful when S-waves prevail among the all-round spectrum of seismic loads. The local magnitude M_L is a logarithm of the maximum trace amplitude A_{max} recorded on a short-period seismograph: $M_L=\log A_{max}$ (Richter1935);

- The Surface Wave Magnitude M_S with a period of waves of about 18-22 sec most commonly describes the size of distant (farther than 2000 km) earthquakes (Gutenberg and Richter, 1956);

-The Body Wave Magnitude M_B describes deep-focus earthquakes at a distance of 600 to 2000 km from epicenter (Gutenberg and Richter, 1956);

-The Moment Magnitude M_W describes ground shaking, basing on seismic moment.

Magnitudes M_s and M_W can be estimated from recorded low-frequency seismic signals. On the contrary, magnitudes M_L and M_B can be derived from recorded high-frequency seismic signals. Consequently, different meanings of the magnitude may characterize the same earthquake (Nuttli & Shieh, 1985). The moment magnitude M_W has been recognized as the most reliable and more accurate measure for large earthquakes.

Gutenberg & Richter (1956) proposed the equation associating the intensity *I* with magnitude *M*:

$$M = \frac{2}{3}I + 1 \tag{2.1}$$

Shear stress emerging in a soil massive is often expressed through cyclic stress ratio CSR (Seed &Idriss 1971) as:

$$CSR = \frac{\tau_d}{\sigma'_v} \tag{2.2}$$

where τ_d is a shear stress emerging during an earthquake; σ'_v is a vertical effective stress in a soil mass.

The appropriate number of stress cycles depends on the duration of ground shaking and thus on the magnitude of earthquake (Seed &Idriss 1971) (Table 1).

Table 1. The quantity of cycles in relation to a magnitude M_W, assumed for dynamic triaxial compression test

M_W	5.25	6.00	6.75	7.50	8.50
N	2-3	5	10	15	26

3 CURRENT METHODS FOR FOUNDATION SETTLEMENT PREDICTION UNDER SEISMIC IMPACT

The soil mass settlement resulted from liquefaction generated by seismic impact can be determined with the following equation (GOST R 56353-2015):

$$S_d = R_S(e_0 - e_{min})\frac{H}{1 + e_i} \tag{3.1}$$

where e_i is an initial void ratio; e_{min} is a void ratio at a maximum density; H is a soil thickness; R_S is a compaction factor at liquefaction, which is considered as:

$$R_s = \frac{\Delta e}{e_i - e_{min}} \tag{3.2}$$

where Δe is a void ratio increment after liquefaction, estimated at the consolidation stage in the end of dynamic loading under effective stress corresponding to in-situ conditions.

However, liquefaction susceptible soils are not authorized for using them as a basement (Dashti et al. 2010, Iwasaki et al. 1978, Chaloulos Y.K. et al. 2020). Therefore, this technique is unsuitable for predicting seismic settlements of foundations.

Stavnitser has proposed a notably practical layer-by-layer summation approach (Stavnitser, 2010).

The seismic settlement $S_{d\mu}$ can be predicted with respect to the earthquake repeatability as follows (Figure 2):

$$S_{d\mu} = \sum_{j=I_*}^{I}\sum_{i=1}^{n}\mu_{jt}\,S_{dij}\frac{h_i}{h_0} \tag{3.3}$$

where μ_{jt} is an expected number of earthquakes within the entire operational period of the structure t_c;

h_i is a thickness of the i-layer;

h_0 is a thickness of the layer in the testing device;

S_{dij} is an experimentally received additional settlement induced by the seismic impact.

The expected number of earthquakes μ_{jt} can be estimated from the equation:

$$\mu_{jt} = 1 + \lceil \mu_j t_c \rceil \tag{3.4}$$

μ_{jt} is an annual average frequency of earthquakes with intensity $j \leq 1$.

The parameters S_{dij} and h_0 can be obtained from in-situ and laboratory test results: dynamic plate test or shaking table test, and others. These are unique tests, and they are not commonly used in a routine engineering survey. The prediction technique of the seismic settlement without liquefaction has not been developed yet.

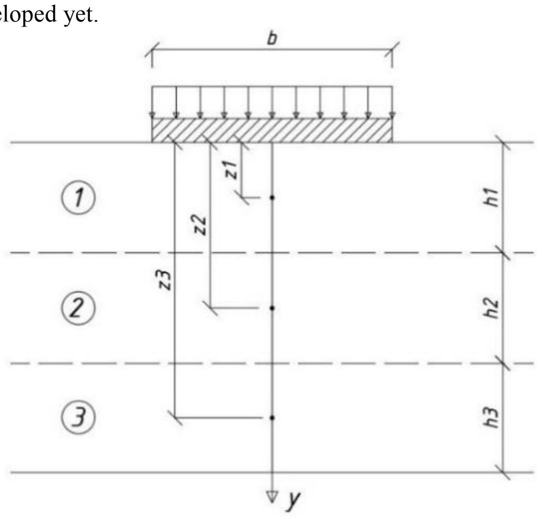

Figure 2. Designing scheme of layer-by-layer summation method for predicting seismic settlements

4 METHODS AND RESULTS

4.1 Experimental technique

To develop a method for predicting settlement with respect to the frequency of earthquakes, sand specimens have been subjected to cyclic triaxial compression test performed with a GDS ELDYN test equipment (fig. 3).

The specimens were prepared from disturbed sand samples collected at three different construction sites in Russia. Preparation of the reconstitute sand specimens included pluvial compaction in the air and subsequent water saturation with using a back-pressure increment method. The initial specimen size was 50 mm in diameter by 100 mm in height.

Samples collected for testing comprised Quaternary alluvial, fluvioglacial and marine medium to fine sand (fig. 4). Mineralogical composition of alluvial and fluvioglacial sand comprised quartz and feldspar. The sand of marine origin

comprised crushed seashells. The initial void ratio was assumed as $e_i = 0.60$ for all tests.
The following experimental technique was proposed.
At the first stage, after the specimen had been reconstituted and saturated, an anisotropic load (consolidation) was applied until the conditional deformation steadiness appeared.
At the second stage, the specimen experienced cyclic loading under consolidated undrained (CU) conditions.
At the third stage, static anisotropic loading (consolidation) was immediately applied after the cyclic loading had ceased, and it had been maintained until the conditional deformation steadiness occurred.
The stages two and three were repeated then. The cyclic load was applied at a frequency of 0.5 Hz. The number of cycles had been initially set as four cycles, and was then increased by 2 to 3 times at every step.
Volumetric and axial strain not exceeding 0.05% per 30 minutes was assumed as a conditional stability criteria. For measuring the volumetric strain, a back-pressure system was applied. A series of test was performed for *CSR* = 0.039…0.381. As a result, the void ratio Δe changed with increasing number of cycles.
Such an approach has been justified by the fact that each new earthquake induces shear deformations in a soil foundation and, consequently, the soil compaction (especially for sand).

Figure 3. Triaxial compression test instrument GDS ELDYN at loading

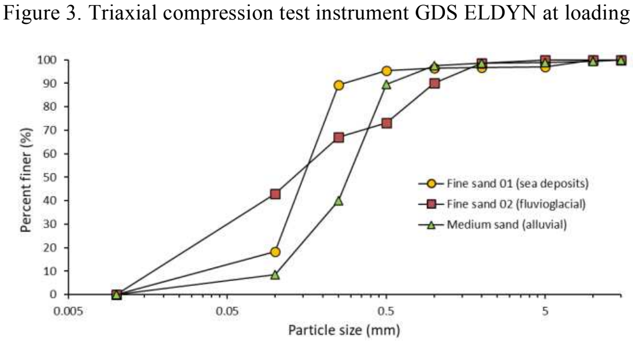

Figure 4.Grain-size distribution curves of tested sands

5 RESULTS

The performed series of test has provided the following conclusions.

5.1 Deformations at cyclic loading.

Figure 5 presents axial deformation versus a number of cycles at cyclic loading *CSR*=0.237…0.260 at the first stage of loading. It can be observed that Fine sand 01 (marine) produces the largest resistance to a cyclic loading: its maximum deformation has not exceeded 0.72% at four cycles. This results from the size and roundness of the sand particles represented by crushed seashells. Clinging to each other, the seashells' pieces resist cyclic loading. At the same time, Fine sand 02 (fluvioglacial) shows 6.1% of axial deformation; the specimen has suffered liquefaction at *N*=3. Such a low resistance to cyclic loading is associated with large amount of clay particles (figure 4).
Although Medium sand (alluvial) has not experienced liquefaction, its axial deformation has been recognized as 2,65%, which can be considered significant.
Since liquefaction of the soil foundation is undesirable during the construction in seismic areas, the specimens have been analyzed under the load not exceeding their dynamic strength.

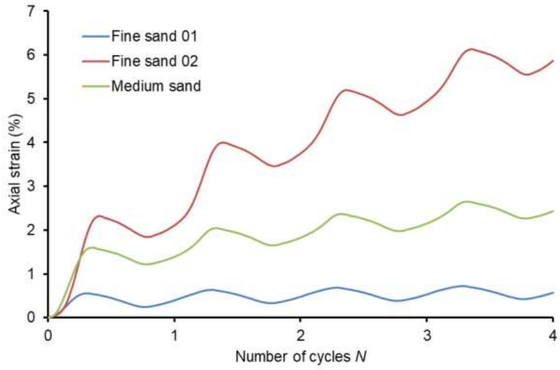

Figure 5. Specimens' deformation at cyclic loading CSR=0.237…0.260

5.2 Anisotropic consolidation

The soil specimen behavior under isotropic loading follows the compression law.
However, specimens subjected to axisymmetric triaxial compression under anisotropic consolidated conditions may exhibit dilative behavior (dilatancy) due to the following factors:
- Development of shear planes under shear stresses. As a result, the specimen shows the onset of rheological behavior (Vyalov,1978) (Fig. 6b). In particular, after the load has been initially applied, the specimen may start to contract and then dilate.
- The influence of particle size and roundness. The roundness of sand particles produces a significant effect on their contractive behavior. For example, marine fine sand largely represented by angular crushed seashells contracts being anisotropically consolidated (Fig. 6a). The sharp edges of these particles finely cling to each other forming a mass resistant to shear stresses. In this case, dilatancy has not been observed. At the same time, alluvial and fluvioglacial fine sand largely containing rounded particles of quartz and feldspar or clay particles has exhibited much more dilative behavior. Rounded particles and clay particles are less likely to cling to each other. Medium sand has shown the same behavior.

Besides, anisotropic consolidation is affected by *CSR* preceding it. Thus, specimens have tended toward contraction at *CSR*>0.125, and have shown more dilative behavior at *CSR*<0.125. This is probably due to the fact that large loads induces more intensive repacking of the particles, especially in zones subjected to intensive shear stresses.

The behavior of soils under anisotropic consolidation is a topic of the day, and it obviously requires an additional study. A value of the soil contraction is a most significant for calculation of the foundation settlements. Triaxial test can be performed under both isotropic and anisotropic consolidation conditions.

The quality of saturated specimens deserves a special attention. When shear stress acts on the soil specimen, the tensile stress develops urging soil pores to open and release trapped gas bubbles. This reduces the accuracy of the deformation measurements. To minimize the influence of the trapped gas, the initial pore pressure in the specimen should be maintained at least 300 kPa. At the same time, some authors (Head 1998) suggested that the complete release of the trapped gas could be achieved by applying the pressure of 700 kPa or more.

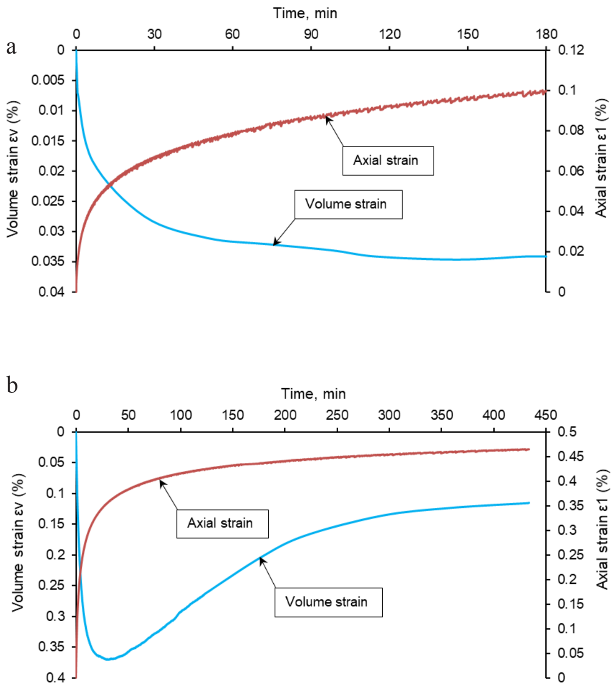

Figure 6. Deformation of the specimen at anisotropic consolidation:(a) – marine fine sand (01), (b) – fluvioglacial fine sand (02)

5.3Contraction at cycling loading

Figure 7 presents the results of tests on three types of sand with e_i=0.6. Considering the results analyzed, the following conclusions can be drawn out.

Fine sand (01) and medium sand have experienced liquefaction at *CSR*=0.353…0.381 and *N*<8. Fine sand (02) has experienced liquefaction at *CSR*=0.245 and *N*=3. After liquefaction, the void ratio insignificantly changes at the increasing number of the loading cycles *N*. The volumetric strain shows 1.4-3% after liquefaction and predominantly depends on the soil type.

Depending on *CSR*, the volumetric strain ε_v can achieve 1.3% (0.25% on the average) without soil liquefaction. At a low load *CSR*≤0,1 ε_v does not exceed 0.23%. The obtained results are in compliance with those received from shaking table test (Ueng et al.2010). In that test, the value of ε_v does not exceed 0.2% at *PGA*=0.03-0.1g (corresponding to CSR<0.1), and *N*<40.

Sand contraction at cyclic loading is subjected to a power law and can be described with a following power function:

$$\Delta e = aN^b \tag{5.1}$$

where - *a* and *b* are empirical coefficients depending on CSR and sand type respectively.

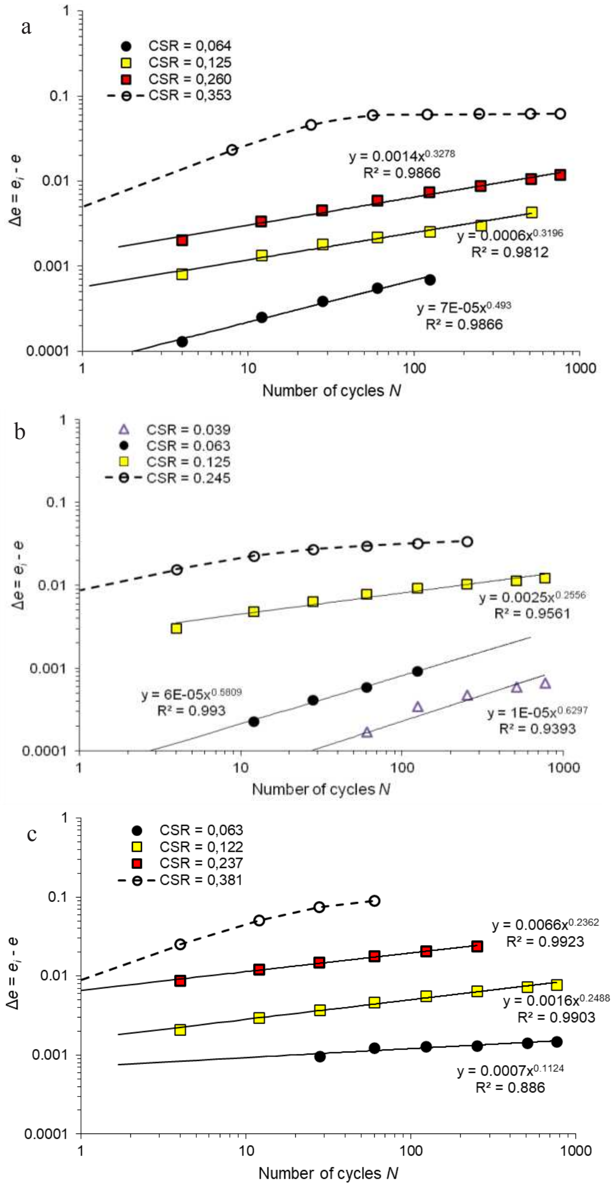

Figure 7 – Void ratio versus number of loading cycles: a) fine sand 01 (marine), b) fine sand 02 (fluvioglacial). c) medium sand (alluvial)

The empirical coefficients for different types of sand are shown in table 2. It has been recognized, that *a* increases with increasing *CSR* and the dependence is governed by the power law (figure 8). Thus, for fine sand subjected to low seismic loads, *a* increases by 6 times as *CSR* increases from 0.039 to 0.063, and with increasing *CSR* from 0.063 to 0.260, *a* increases by 20…42 times. For medium sand, *a* increases by 9 times with *CSR* increasing by 3.8 times. The parameter *b* scarcely depends on *CSR*. However, it depends upon a sand type and assumes the value of 0.380-0.412 for fine sand and 0.2 for medium sand. For the same type of sand *b* largely depends on the qualitative composition of sand.

Considering the experiments performed and analyzing the empirical coefficients *a* and *b*, the following function has been proposed to describe sand contraction:

$$\Delta e = a_1 CSR^{a_2} N^{a_3} + a_4 \tag{5.2}$$

where a_1, a_2, a_3 and a_4 are empirical coefficients obtained from regression analysis of the tests results.

Table 3 presents the particular values of these empirical coefficients.

Table 2 – Empirical coefficients for the formulae (5.1)

Soil description	*CSR*	*a*	*b*
Fine 01 (marine)	0.064	7.0E-05	0.493
	0.125	6.0E-04	0.320
	0.260	1.4E-03	0.328
Fine 02 (fluvioglacial)	0.039	1.0E-05	0.630
	0.063	6.0E-05	0.581
	0.125	2.5E-03	0.256
Medium (alluvial)	0.063	7.0E-04	0.112
	0.122	1.6E-03	0.249
	0.237	6.6E-03	0.236

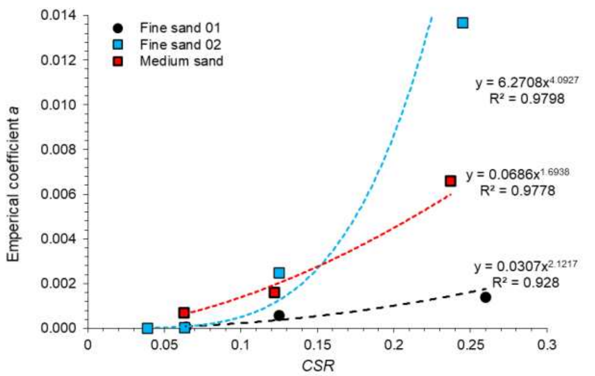

Figure 8 – An empirical coefficient *a* versus *CSR* for sands used in this study

Table 3 – Empirical coefficients for (5.2)

Soil description	Empirical coefficients				R^2
	a_1	a_2	a_3	a_4	
Fine sand 01	0.01	1.18	0.27	-0.001	0.997
Fine sand 02	0.19	1.77	0.16	-0.001	0.992
Medium sand	0.12	1.95	0.22	-0.0004	0.998

6 ADDITIONAL FOUNDATION SETTLEMENT PREDICTIONS CONSIDERING THE EARTHQUAKES REPEATABILITY

The following approach using the data received from dynamic triaxial test can be proposed to compute additional seismic settlement of foundation:

$$S_{d\mu} = \sum_{j=1} \sum_{i=1}^{n} \frac{a_1 CSR_j^{a_2} \left(N_j \cdot \mu_{j,t}\right)^{a_3} + a_4}{1 + e_i} h_i \qquad (3.2.1)$$

where e_i is an initial void ratio.

The example below provides calculation of the additional seismic settlement of the building model. A structure erected on a foundation slab bearing a mean pressure of 300 kPa along the bottom has been taken as a building model. The operational period of the structure is 50 years. The model is located in Petropavlovsk-Kamchatsky (Russia). The soil foundation consists of sand with static deformation moduli E=28-32MPa. The designed settlement at static loading is about 120 mm.

According to the average values of annual seismic impacts in the Petropavlovsk-Kamchatsky region (Stavnitser, 2010), μ_j=0,00333 at I=9; μ_j=0,0218 at I=8; μ_j=0,0713 at I=7. Earthquake intensity I of 5 and 6 degrees has not been taken into consideration. The Intensity has been defined by MSK-64.

Computation of $S_{d\mu}$ has been performed with respect to distribution of the pressure and CSR in depth. A number of cycles produced by seismic impact are given in the table 1.

Table 4 presents the computation results. It has been revealed that additional settlement depends on soil properties and can achieve from 55 to 175 per cent for a 50-year long operational period. The computed settlements are significant for structures. At the same time, the magnitude values are rather close to those available from the world experience.

For example, Kramer, (1996) described several examples of earthquake settlements. In particular, the San-Fernando earthquake in 1971 (M_S=6.5) produced 85.6 mm settlement of the 15 m thick unsaturated sand without any additional loading (Tokimatsu & Seed, 1987). The value is close to those presented in the table 4. In his study, Stavnitser (2010) has recognized that seismic settlement is 2.4 times greater than settlement caused by static load.

It is worth noting that such significant settlement may not cause the destruction of structures, however, it can affect its serviceability. If a foundation is inhomogeneous and with thinning layers, a substantial tilt of the structure may appear.

In the future, it is necessary to compare the results obtained with monitoring data.

It worth to be mentioned, that values of additional settlement directly depend upon the specimen behavior at cyclic loading, specifically, large settlement values are typical to soils less resistant to cyclic impacts.

It is reasonable to consider the current approaches to seismic foundation design in more detail, as well as to evaluate additional seismic settlement of especially important facilities throughout their entire operational period.

Since a large number of assumptions have been made, the presented calculations may be inaccurate (against the background of static calculations of foundation settlement). However, as it has been shown above, the results are commensurable with other experiments and observations.

7 CONCLUSIONS

In accordance to the current practice, it is an ultimate capacity of foundation that is mostly required to be ensured by earthquake engineering. For this reason, a significant attention has been paid to the resistance of soils to liquefaction. As a rule, foundation deformations induced by a seismic impact (seismic settlement and seismic tilt) may continue. However, the earthquake may cause such a damage, that restoration or repairing work may turn to be very expensive or impractical.

Current approaches deal with prediction of settlement after liquefaction, and besides, they demand a unique instrumentation for soil properties determination.

This paper presents a method for predicting foundation settlement with respect to the earthquake repeatability, based on expected number of impacts of different intensity.

Since the soil parameters are defined from the triaxial test results, the elaborated approach can be introduced for practical application.

In accordance with the proposed approach, the additional settlement has been defined as 55-175 per cent for a 50-year long

Table 4. The results of the additional seismic settlement calculation with account for earthquake repeatability

I	*PGA*	μ_j	μ_{50}	M_W	*N*	$N \cdot \mu_{50}$	*CSR*	Description of soil under the foundation slab bottom		
								Fine sand 01	Fine sand 02	Medium sand
9	0.4*g*	0.00333	2	7.0	12	24	0.187-0.249	47.6	–*	165.7
8	0.2*g*	0.0218	3	6.3	7	21	0.093-0.125	15.6	63.0	37.1
7	0.1*g*	0.0713	5	5.7	5	25	0.047-0.062	2.9	4.1	5.6
Additional settlement resulted from an earthquakes $S_{d\mu}$								66.1	67.1	208.4

* -Liquefaction has been registered; the soil is not supposed to serve as foundation

operational period. Roundness and particle size have a significant effect on a seismic settlement value.
Inhomogeneous foundation with thinning layers may suffer considerable tilt due to a seismic settlement.
Perhaps, seismic settlement does not cause a complete destruction of a structure; however, it may affect its serviceability.
It should be recommended to accumulate test results, compare them with the data of geotechnical monitoring, and develop the proposed approach.

8 ACKNOWLEDGEMENTS

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