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Full scale shaking table tests on a reinforced soil wall with high tenacity polyester geostrips

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ABSTRACT: Reinforced soil walls (RSW) have been commonly used as retaining structures since 1960s. Throughout the years, this technology has been deeply investigated and a large database of full-scale walls was utilized to develop new design methods and to improve the systems performance, mainly under static conditions. In the past 20 years, examination of post-earthquake RSW structure conditions have also indicated that reinforced soil systems were capable to withstand seismic loading even when the actual ground accelerations highly exceeded the design assumptions. Many researchers focused on the dynamic behavior of retaining walls with metallic bars or polymeric geogrids reinforcement but very few studies have been conducted on geostrip reinforcement. For this reason, a full scaleconcrete panel faced retaining structure with geostrips reinforcement has been built and tested in Russia under intense dynamic loads, equivalent to earthquake effects of 7, 8 and 9 points on Russian MSK-64 scale. Full scale physical tests were carried out on a 3.75 m high model wall using a Russian SGD-75 type shaking table. A FEM numerical model was developed for simulating and analyzing the test results. The paper presents the program and results of the shaking table tests, and the description and results of the FEM model.

1 INTRODUCTION

Reinforced soil walls (RSW) are retaining structures where the soil is internally reinforced with tensile resistant inclusions. Soils have usually high compressive strength but limited resistance to tensile and shear stresses. Soils can be self stable only up to their angle of repose. In order to allow the construction of steeper slopes and wall, reinforcing elements can be placed in the soil mass: the composite structure soil – reinforcement affords higher mechanical properties than the single components.

Nowadays, reinforcedsoil walls are widely used worldwide, thanks to their static and seismic performance, and costeffectiveness. Several empirical researches onreinforced soil walls have been developed in recent years but, among these, only very few studies have been conducted on structure reinforced with geostrips in seismic conditions.

The research described in the present paper has been carried out on the RSW system commercially known as Maccaferri MacRes® system. This RSW, with prefabricated concrete facing panels connected to high strength geostrips, placed in the backfill in successive layers, is fast and easy to build; moreover, a wide range of geostrips, with different tensile strengths (up to 100 kN/strip), is available. These features afford anefficient design and allow the construction of tall structures bearing very high loads.

Maccaferri ParaWeb® geostrips are made of bundles of high tenacity polyester fibres protected by heavy-duty linear low-density polyethylene (LLDPE) coating. Polyester, with its minimal tensile creep, is the tensile resistant element, whereas polyethylene coating provides the protection of the fibres against aggressive environment (with pH up to 13) and severe installation conditions. The geostrips have a rough surface which develops excellent soil - reinforcement interaction. The design life of the geostrips has been certified up to 120 years.

Thanks to these important characteristics, the analyses of walls in post-earthquake conditions have indicated that these structures can withstand very high seismic loads, even higher than the design values.

The aim of this paper is to present the results of full-scale shaking table tests performed on the above described RSW reinforced with geostrips, subjected to high seismic loads. A FEM numerical model was developed for simulating and better analyzing the test results.

2 SHAKING TABLE TESTS

Shaking table dynamic tests for assessing the seismic stability of the RSW were carried out in 2011 on a full scale reinforced soil retaining wall by Stroy-Dinamika Scientific and Engineering Company (Belyayev 2011).

The tests were performed following the program specifications developed by Stroy-Dinamika in accordance with the Russian GOST Standards (GOST 30546.1-98, GOST R 22.0.03-95,SP 14.13330.2011, GOST 17516.1-90). The purpose of the tests was to simulate a dynamic impulse sequence comparable to the ones experienced under earthquakes of 7 to 9 points on the Russian MSK-64 scale.

The SGD-75 shaking table used for the seismic tests is a parallelepiped metal structure of 11.3 m x 5.0 m x 0.5 m dimensions, mounted with clearance inside a hard box-type enclosure on cushion type pneumatic elements. A 9.5 m x 2.6 m x 4.0 m soil tray is attached to the shaking table (Fig. 1).

The movement of the table was provided by a bench-top actuator, which consists of a number of pneumatic and hydro-pneumatic actuators applying impulse load, a vibration machine and a software to control their operation (Belyayev 2011), as shown in Fig. 1. The shaking table could have horizontal, vertical and multidirectional motion, including rotation.

During the tests, the tensile force in the geostrips was continuously monitored through strain gages mounted on the geostrips (Fig. 2a), which calibration curve directly allowed to get the tensile forces in the geostrips; 30 sensors were used to record the face movement, the soil pressure behind the facing (Fig. 2b), and the acceleration of the shaking table with the panels relative displacements. The layout in Fig. 3 identifies the positions of the instruments, where "A" stands for accelerometer, "P" for displacement gage, "D" for soil pressure cell, and "U" for the strain gages on the geostrips.

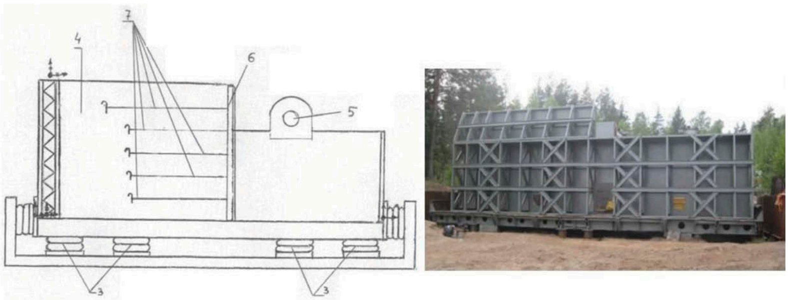

Figure 1. Scheme (a) and actual view (b) of the SGD-75 shaking table: 3. cushion type pneumatic actuators; 4. tray for soil; 5. vibration machine;6.concretepanels facing of the reinforced soil wall under test; 7. geostrip reinforcement.

Figure 2. Pictures of the strain gages mounted on a geostrip (a) and of the soil pressure cells (b).

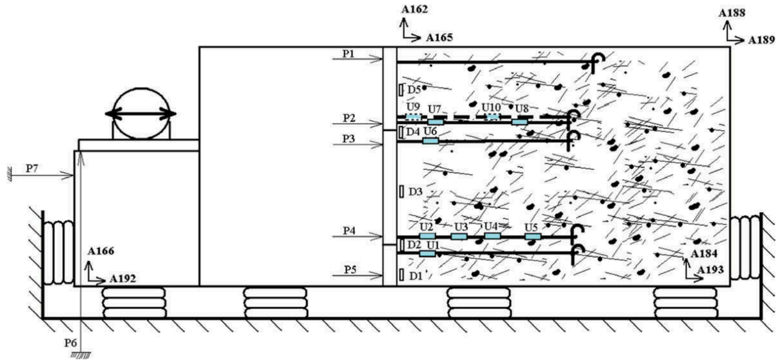

Figure 3. Layout of sensors in the SGD-75 shaking table: “A” stands for accelerometer, “P” for displacement gage, “D” for soil pressure cell, and “U” for the strain gages on the geostrips.

The facing of the tested retaining wall was made of prefabricated concrete panels, 140 mm thick (Kupriyenko & Belyayev 2011). The wall was 4.0 m high and 2.6 m wide, with the facia consisting of two standard square concrete panels (1.5 m x 1.5 m x 0.14 m) and seven cut panels which completed the cross section of the shaking table soil tray. Fig. 4a shows the front view of the model wall with the panels configuration; Fig. 4b shows the cross-section of the model wall with the layout of geostrips. In Fig. 4a the “letter” denomination describes the panel type while the number refers to the number of connections to geostrips (e.g. A4 stays for A type panel with 4 connections to geostrips). Polymeric bearing pads were placed in the horizontal joints to avoid the concrete-to-concrete contact; the lower panels were embedded in a 0.15 m thick sand layer.

The fill was a dense compacted sand (Fig. 5). Soil compaction was performed to obtain a dry density of 96.5 – 97.0 % maximum Standard Proctor density according to GOST 22733-2016 standard. Frequent compaction controls were carried out for each compaction lift (Belyayev 2011). The soil density was checked usinga hydrostatic densimeter, and CBR tests were performed on the compacted soil.

The sand fill was reinforced with five layers of geostrips: the top geostrips (type 2D30) featured 30 kN tensile strength and 83 mm width, while all other geostrips (type MD36) featured 36 kN tensile strength and 47 mm width. Fig. 2b shows the reinforcement layout.

After the compaction of each soil lift (Fig. 5a), the geostrips were installed passing through the loops on the rear side of the panels (Fig. 5b) and were fixed using special pins to anchor the strips at the rear end (Kupriyenko & Belyayev 2011). The geostrips were then stretched, removing slack, and instrumented with load cells to record the tension behaviour during the tests (see Fig. 2a). The instrumentation was protected with specific casings to avoid damage during construction of the wall.

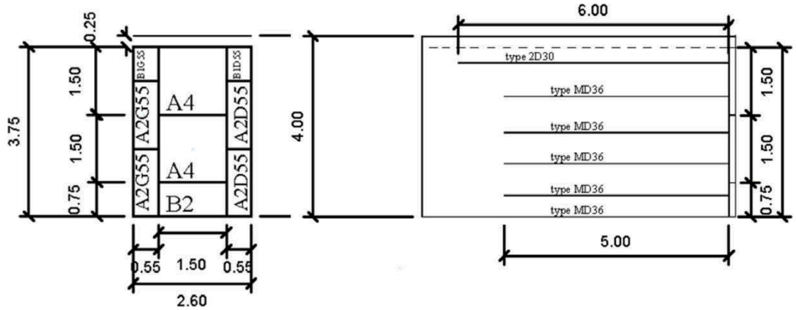

Figure 4. Front view of the test wall (a) showing the concrete facing panels and cross section (b) showing the layout of geostrips.

Figure 5. Compaction of soil (a) and connection of geostrips to concrete facing panels (b).

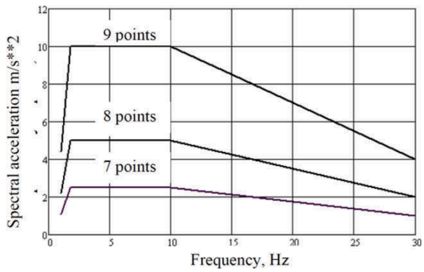

Figure 5. Standard response spectra of horizontal acceleration for 7, 8 and 9 points earthquakes based on the MSK-64 scale.

3 RUSSIAN SEISMIC REGULATION

The shaking table tests were performed in Russia, thereforethe seismic inputs referred to the macroseismic scale used in the Country. The scale is called MSK-64, where the acronym stands for the three researchers, Medvedev, Sponheuer and Karnik, who in 1964 proposed this scale to rate earthquakes.

The MSK-64 scale (Medvedev et al 1963) derives from the old Mercalli scale (1887-1910), the Mercalli - Cancani - Sieberg scale (1930) and the Modified Mercalli scale (1956), and it was reviewed several times up to 1981. The present MSK-64 version, together with the European Macroseismic Scale (EMS 1998), is one of the macroseismic scales commonly used in Europe.

Table 1. Earthquakes scales comparison

MSK-64	Mercalli	Richter
1.0-3.0	1.0-3.0	I
3.0-3.9	3.0-3.9	II-III
4.0-4.9	4.0-4.9	IV-V
5.0-5.9	5.0-5.9	VI-VII
6.0-6.9	6.0-6.9	VII-IX
7.0 and higher	7.0 and higher	VIII and higher

The MSK-64 scale is based on the observation of earthquake effects on people, environment and structures. Structures are divided in three categories (Medvedev et al 1963): masonry buildings (class A), bricks buildings (class B) and concrete skeleton building (class C). The intensity level is rated from I (slight) up to XII (landscape changes).

To provide a better perception of the intensity levels described in the Russian scale, a comparison with other scales used worldwide is given in Table 1. The comparison between the MSK-64, the Mercalli and the Richter scalesis elaborated referring to a USGS study (United States Geological Survey 2018) and to relevant literature (Musson et al 2010).

The shaking table tests were performed by reproducing the effects of 7, 8 and 9 points earthquakes based on the MSK-64 scale. The retaining wall was assumed to be installed on foundation soils similar to 7, 8, 9 point earthquakes typical site conditions (Belyayev 2011).

According to Russian Standards, the seismic horizontal speed for foundation can vary in the range of 130 - 250 mm/s for a 7 point earthquake, 260 - 500 mm/s for an 8 point earthquake, and 510 - 1000 mm/s for a 9 point earthquake. The forecasted horizontal ground displacements ranges were 70 - 120 mm, 130 - 180 mm, 190 - 270 mm (Belyayev 2011).

The standard response spectra of horizontal acceleration for 7, 8 and 9 points earthquakes based on the MSK-64 scale are shown in Fig. 5.

4 EXPERIMENTAL RESULTS

Three simulations are presented below: “Experiment 1”, “Experiment 2” and “Experiment 3”, simulating the 7 point, 8 point and 9 point earthquakes of MSK-64 scale, respectively.

In Experiment 1 (see Fig. 6a) the shaking table was set to simulate the standard response spectra (spectral accelerations over frequencies) for 7 point earthquake on the MSK-64 scale (SP 14.13330.2011). The maximum horizontal acceleration values were $+3.5 \div -2$ m/s^2.

The peak pressures in the fill applied to the rear framing surface were recorded at the level of approx. 2.5 m on the base, showing a relatively low value of 5.6 kPa. The maximum tensile force in the geostrips was recorded in the fourth layer on the base, and was equal to 365 N.

Figure 6b shows that the central panel displacement considerably grows along the height of the wall. During the shaking test, the maximum displacement, recorded in the upper part of the wall facing, was equal to 1.4 mm, while the maximum residual displacements after the simulated earthquake (~ 0.4 mm) were observed at the junction of the middle and upper A4 panels shown in Fig. 4a. These facing deformations do not represent a problem for the wall stability.

The inspection of the structure after the first experiment confirmed the stability of the wall and the possibility of further seismic loadings (Belyayev 2011; Kupriyenko & Belyayev 2011).

In Experiment 2, an 8 point MSK-64 earthquake was simulated (Fig. 7a). The maximum values of horizontal acceleration in the impulse phase were $+7.5 \div -4$ m/s^2.

The peak pressures in the fill applied to the rear framing surface were recorded at the level of approx. 2.5 m on the base, showing a value of 16 kPa. The maximum tensile force in the geostrips, recorded in the fourth layer on the base, was equal to1.53 kN.

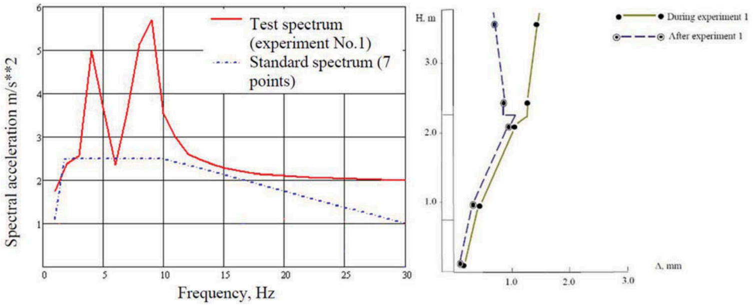

Figure 6. Test and standard response spectra of horizontal acceleration for Experiment 1 (a)andmaximum central panels displacements (b).

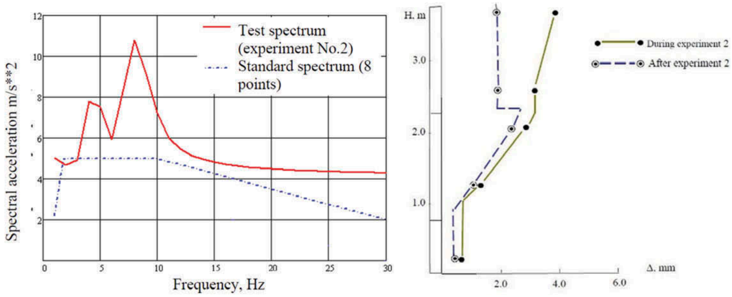

Figure 7. Test and standard response spectra of horizontal acceleration for Experiment 2 (a) and maximum central panels displacements (b).

Fig. 7b shows the maximum relative panel displacements along the height of the facing central section referred to Experiment 2 only (that is, not considering the displacements already produced during Experiment 1). The upper facing panel reached a maximum displacement of 4.0 mm, while 3.0 mm post earthquake residual displacements were observed. Once again, an inspection was done, confirming the wall stability and the possibility of carrying on further tests (Belyayev 2011; Kupriyenko & Belyayev 2011).

During Experiment 3, a 9 point MSK-64 earthquake was simulated (Fig. 8a). The maximum values of horizontal acceleration in the impulse phase were in the range of +12.8 ÷ -8.5 m/s^2.

The peak pressures in the fill applied to the rear framing surface were recorded at the level of approx. 2.5 m on the base, showing a value of 22 kPa. The maximum tensile force in the geostrips, recorded in the fourth layer on the base, was equal to 1.80 kN, as shown in Fig. 9.

Fig. 8b shows the distribution of maximum central panel displacements along the height of the wall and the post earthquake residual displacements, referred to Experiment 3 only (that is, not considering the displacements already produced during Experiment 1 and Experiment 2). The top facing panel reached a maximum displacement of approximately 5.5 mm, with 4 mm residual displacement (Belyayev 2011; Kupriyenko & Belyayev 2011).

Fig. 10a shows the accumulated maximum central panel displacements along the height of the wall after Experiments 1, 2, 3: it is evident that the response of the wall is proportional to the peak seismic acceleration. It is also evident that the reinforced soil structure was able to withstand three consecutive earthquakes without any sign of failure.

The picture in Fig. 10b shows the post earthquake residual displacement of the top central facing panel after Experiment 3, during which a peak horizontal acceleration in excess of 13 m/s^2 (corresponding to over 1.3 g) was applied to the wall (Fig. 8a): considering that the wall

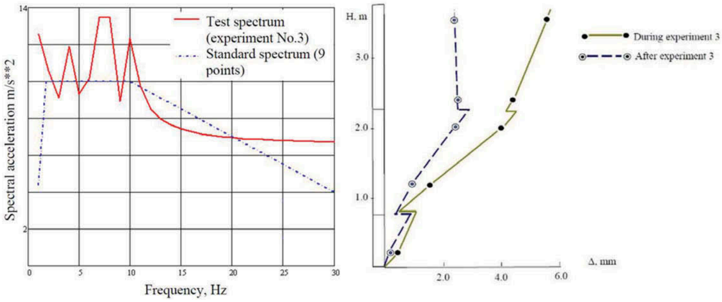

Figure 8. Test and standard response spectra of horizontal acceleration for Experiment 3 (a) and maximum central panels displacements (b).

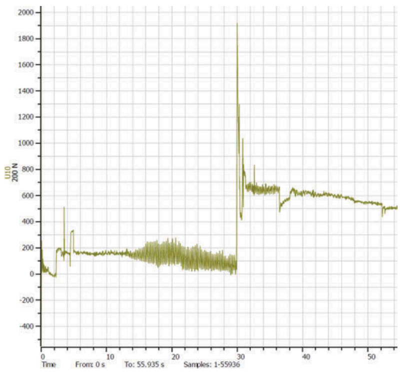

Figure 9. Time history of the tensile force in the geostrip in the fourth layer on the base during Experiment 3.

sustained three consecutive earthquakes of growing intensity, it is evident that the reinforced soil wall can resist very strong earthquakes with minimal deformations and displacements.

Moreover, it has to be noted that the tensile forces recorded in the geostrips are far lower than their available tensile strength; as shown in Fig. 9, the geostrip response to impulsive loads is impulsive as well; under impulsive loads the geostrip resists with its tensile strength, not affected by tensile creep; hence for MD36 geostrips the available tensile strength under impulsive loads can be assumed to be equal to 87.5 % of their ultimate strength (that is 0.875 · 36 kN = 31.5 kN); therefore it results that even during the very strong earthquake simulated in Experiment 3 the maximum recorded tensile strength of geostrips (equal to 1.80 kN, see Fig. 9) was just a fraction of the available tensile strength. This is one of the reasons of the high seismic resistance of reinforced soil structures.

5 NUMERICAL MODEL

A numerical calculation was carried out for better analyzing the results ofthe full scale shaking table tests.The Finite Element Method (FEM) numerical model was implemented using the ANSYS software code (Ansys Inc. 2011). The concrete panels were modeled using solid eight node elements and four-node shell element, included in the ANSYS libraries (Ansys Inc. 2011). The geostrips were modeled using 4-node flat elements, while 8-node elements were used for modeling the soil. The Drucker - Prager model was used to characterize the soil behavior,taking into account adhesion, dilatancy ratio and internal friction angle. Table 2 lists

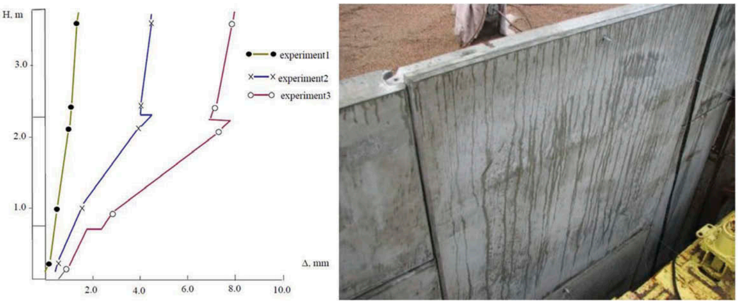

Figure 10. Accumulated residual displacementsof central panels for Experiments 1, 2, 3 (a) and picture showing the post earthquake residual displacement of the top central facing panel afterExperiment 3 (b).

Table 2. Values of material properties used in the numerical model

	Elastic modulus, MPa	Poisson ratio	Density, kg/m^3	Adhesion factor, MPa	Angle of internal friction, degrees	Dilatancy ratio
Concrete Panels	1,500	0.3	1,444	------	------	------
Soil (sand)	20	0.3	1,800	0.005	30	0.05
Geostrips	370	0.3	31.87	------	------	------

the values of the material parameters used in the numerical model. These values were calibrated and fine tuned through a trial-and-error process until a satisfactory matching of numerical results and physical model results was achieved. The three central concrete panels (see Fig. 4a) were modeled, as shown in Fig. 11a; contact elements were used to model the panels interaction, characterized by a friction factor equal to tan $\delta = 0.4$. Vertical and horizontal displacements of the lower soil surface and of retaining wall foundation were inhibited, and the symmetry condition on the right and left longitudinal planes were set as boundary conditions.

The seismic load was introduced using the same time histories of horizontal and vertical acceleration applied by the shaking table during the tests; Fig. 11b shows the time history of horizontal acceleration used for the simulation of Experiment 1.

The numerical calculation afforded the following results: the horizontal displacements of the facing panels shows the pattern in Fig. 13a, with a maximum of approx. 10 mm; the soil pressure against the wall facing has a waving pattern with a maximum of approx. 10 kPa (Fig. 13b); the tensile force in the geostrips shows a maximum value of approx. 3.5 kN at the connection with the concrete panels and a rapid decrease away from facing (Fig. 14).

The following considerations can be drawn from the analysis of these results:

- The geostrips afford excellent interaction with the soil: in fact the tensile force, which is maximum at the connection with the wall facing panels, is quickly transferred to the soil through shear and pullout stresses;
- Moreover, the geostrips have a confining effect on the surrounding soil: in fact the soil pressure is higher in the middle between two layers of geostrips;
- The geostrips provide excellent anchorage of the facing panels, thus minimizing the horizontal displacements.

The comparison between the horizontal movements measured in the full-scale test and calculated with the FEM model is shown in Fig. 15: results appear to be in good agreement, and FEM results seems even conservative in respect of the full-scale test results.

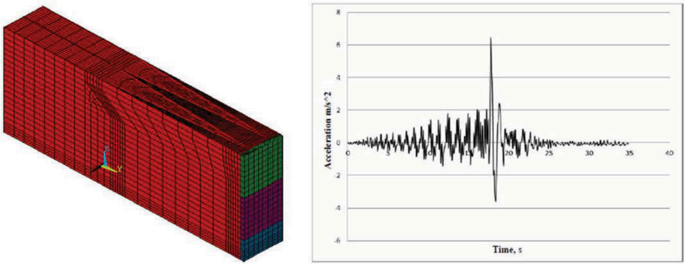

Figure 11. Discretization of the FEM model (a) and time history of horizontal acceleration for the simulation of Experiment1 (b).

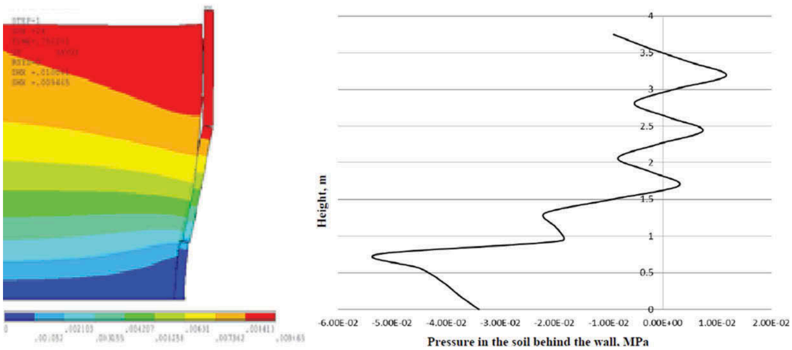

Figure 13. Horizontal displacements of the wall (a) and soil pressure against the wall facing (b).

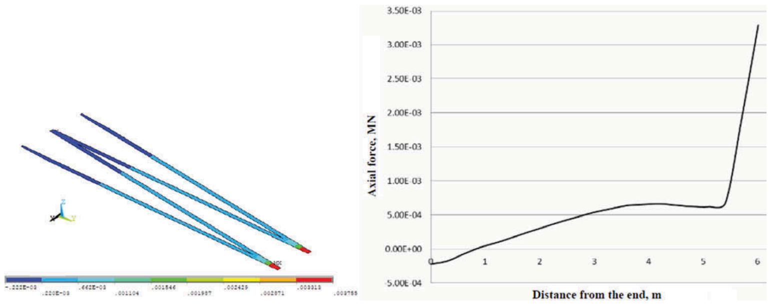

Figure 14. Tensile strength in the top geostrips: spatial distribution (a) and axial distribution (b).

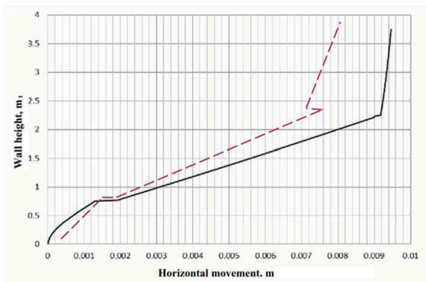

Figure 15. Horizontal movement in the full-scale test (dashed line) and FEM model (continuous line).

Hence the FEM numerical model can be safely used for further parametric analyses of the behavior of this specific RSW under different static and seismic loading conditions.

6 CONCLUSIONS

A full scale RSW system with geostrips reinforcement and concrete panels facingwas tested on a SGD-75 shaking table in Russia. The structure was 4m high with 5 layers of geostrips, having ultimate tensile strength in the range of 27÷36 kN. The shaking table was set to simulate 7, 8 and 9 point earthquakes effects on MSK-64 seismic scale.

All three experiments presented minimum deformation of the reinforced soil wall with maximum 5.5 mm horizontal displacement during the third test. However, the accumulated residual displacement after three consecutive simulated earthquakes was limited to approx. 8 mm.

A FEM numerical model was implemented using the Ansys code, allowing better analyses of the seismic response of the wall in the shaking table tests, and the confirmation of the excellent behavior of the reinforced soil system with concrete panel facing and geostrip reinforcement.

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