20th IMEKO TC4 International Symposium and
18th International Workshop on ADC Modelling and Testing
Research on Electric and Electronic Measurement for the Economic Upturn
Benevento, Italy, September 15-17, 2014

An example of employment of static and dynamic monitoring to understand the behaviour of a flexible retaining wall

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***Abstract* – Analysis and design of flexible retaining walls under seismic actions is required in many applications, but is not fully established yet due to the involved complex soil-wall interaction mechanisms. Technology for advanced field measurements and integration with refined numerical analyses represent an attractive option to shade light on this relevant topic of seismic geotechnics. In the present paper, experimental measurements both dynamic and static recorded on a full-scale retaining wall are briefly discussed. Their relevance in view of the development of an effective soil-structure numerical model is pointed out. A multi-level approach to the optimisation of the FE model is discussed, pointing out the potentialities of the integration between field investigations and numerical modelling. The latter is aimed at providing reliable results concerning both the interpretation of the static response and the dynamic behaviour under seismic loads for the development of rational and effective performance based design and assessment procedures.**

I. INTRODUCTION

The behaviour of embedded retaining wall is strongly influenced by the interaction between the soil and the structure. Until now, a number of uncertainties exist in their computation methods, especially when seismic loading are concerned [1].

To shade some light on this topic, a full-scale flexible retaining wall was instrumented in order to provide both static and dynamic response data at the University of Molise, Italy. The retaining wall has a free height of about 6 meters, is composed by two rows of reinforced concrete (r.c.) piles having a diameter of 800 mm and has an overall length of 18 meters. No anchorages were used, while a reinforced concrete top-beam was built on top of the structures.

Further details on the retaining wall can be found elsewhere [2]. The monitoring system is made of 1) a set of six inclinometers pipes placed in two contiguous piles, allow measuring the time-history of the static deflection of the wall during and after the construction stages; and 2) an arrangement of accelerometers, allow detecting the main two fundamental frequencies of the soil-structure system using the Operational Modal Analysis [3]. Based on the data taken by the above described monitoring system, together with the available data of the existing conventional geotechnical characterization, an advanced FE model was developed and calibrated, providing some parameters by a back analysis procedure.

At the moment, the obtained model is correctly able to predict the displacements of the wall during the construction phases, under stating loading. The model is further capable to assess the dynamic response of the wall, subject to environmental excitation. Therefore, the model calibrated on measurements is able to support the study of the retaining wall response under seismic excitation through the adoption of more advanced soil constitutive models. The paper deals with the most relevant outcomes of the monitoring system, the interpretation of the data both for the geotechnical characterization and on the embedded wall and a discussion on the assumptions made and the results of the numerical analyses.

II. STATIC MEASUREMENTS

The deformed shape of the piles was periodically measured throughout the pile length by inserting the slope indicator device into the embedded casings aimed at providing data along the direction of maximum deflection during the erection phase. Step-by-step measurements have been carried out every 50 cm along the pile. A time interval of 10 minutes was adopted at each measurement location in order to compensate the hole and the probe temperatures.

A large amount of data was recorded over a five months period (from September, 2008 to February, 2009) during the construction and excavation phases of the wall and of the building foundations. Measures were stopped once the increments became negligible.

ISBN-14: 978-92-990073-2-7

Displacements of the piles were computed from inclination measurements assuming as a reference the end section of the piles and validated by means of a topographic measures. The horizontal displacement profiles perpendicular to the wall plane for the two monitored piles were very similar, demonstrating that the two rows of piles acted as a single structural component. Measurement results are shown in Figure 1 in terms of average displacements perpendicular to the wall plane.

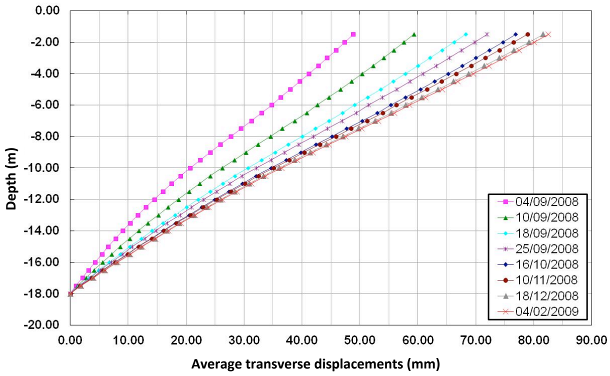

Fig. 1: Average displacements orthogonal to the wall.

A continuous increase in the horizontal displacements of the wall in time was observed, until it reached a final value of about 80 mm or even larger. It basically described a rigid rotation of the wall due to the excavation and erection of the building deep foundations. A comparative analysis of the measurements with others available in the literature is out of the scope of the present paper. Some specific aspects of the response are conversely analysed by means of a static as well dynamic modelling of the structure.

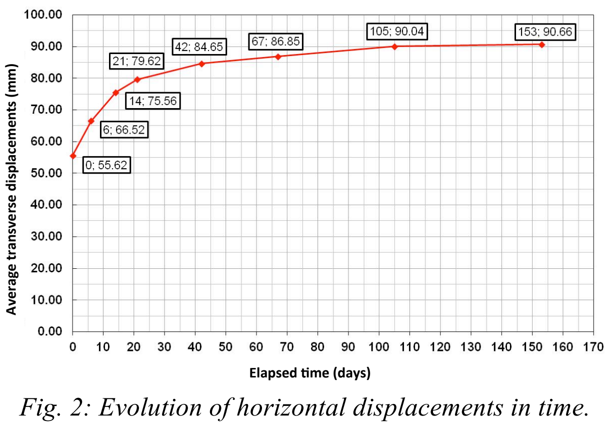

Fig. 2: Evolution of horizontal displacements in time.

III. DYNAMIC MEASUREMENTS

The main objective of the installed monitoring system was the experimental assessment of the main dynamic properties of the system in operation over time and, above all, of its response to earthquakes eventually occurring in the area. The number of embedded sensors installed in the piles, even if constrained by design and practical implementation issues, was certainly appropriate for these purposes [2].

However, due to the lack of literature reference experiences, a preliminary validation of the measures was carried out by means of an improved spatial resolution of the experimental mode shapes. This enabled a reliable evaluation of the correlation with the numerical results in view of the development of the optimization process.

In particular, the reference experimental estimates of the dynamic properties adopted in the refinement of the numerical model were obtained by increasing the number of measurement positions in the upper part of the wall (on the excavated side) through the external installation of additional sensors, Figure 3 and Figure 4.

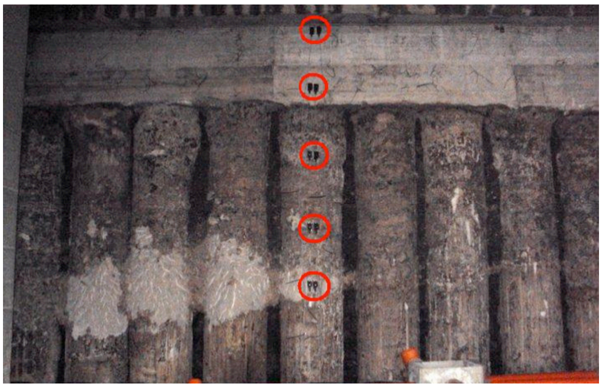

Fig. 3: External sensors on the wall for dynamic testing and embedded monitoring system validation.

Fig. 4: External sensors on the wall for dynamic testing and embedded monitoring system validation.

One hour long records of the dynamic response of the wall subjected only to ambient vibrations were recorded and analyzed in order to identify the fundamental dynamic properties of the wall.

Powerful and robust Operational Modal Analysis

(OMA) techniques, such as the Frequency Domain Decomposition (FDD), the Stochastic Subspace Identification (SSI) and the Second Order Blind Identification (SOBI), were adopted [3].

The low amplitude of ambient vibrations represented the main issue of the dynamic identification of the system; this circumstance however did not affect the relevance of the measures.

Mode	f_{exp} (Hz)	ξ_{exp} (%)
I	3.68	1.4
II	7.23	1.2

Tab. 1. Dynamic identification results

Furthermore, the selected high performance measurement chain provided successful measurements providing useful indications for similar applications involving embedded retaining walls.

A robust identification of the first two fundamental modes was achieved. Results of the dynamic identification process are summarized in Table 1.

Their reliability has been confirmed by cross checks in terms of both natural frequencies and mode shape estimates.

Very consistent results, characterized by negligible scatter, have been provided by the different methods, thus validating the achievement of the identification process.

IV. DYNAMIC MODEL OPTIMISATION

The dynamic module of Plaxis 2D v.8.4 [4] was used to develop a FE model of the retaining wall for the dynamic analysis (Figure 5).

The basic configuration of the design model was retained and reviewed in the light of the available results. However, a number of subtle changes were incorporated in order to enable proper functionality of the model. Such modifications can be addressed as the calibration of the model and they are briefly outlined in the following. The soil was modelled as a linear elastic material.

The very low amplitude of motion due to ambient vibrations supported such an assumption, since the vibrating soil did not experience the development of plastic zones in the region of interest. The elastic modulus of the soil was selected in agreement with the initial tangent modulus E_0 obtained from the Down-Hole tests. The multiple piles wall was modelled as a reinforced concrete member characterised by an equivalent stiffness.

A set of absorbent boundaries was used to reduce the increments of stresses on the boundaries caused by dynamic loading, which would otherwise be reflected inside the soil body. Rayleigh damping was used in the dynamic analysis in order to account for the material damping of the soil under the assumed plane strain conditions. Time integration was carried out according to the implicit Newmark scheme. It is well known that to obtain an unconditionally stable solution, the parameters have to satisfy the following conditions: $N_\beta \geq 0.5$, and $N_\alpha \geq 0.25(0.5+N_\beta)^2$. In the present study, the Newmark damping parameters were selected as follows: $N_\alpha=0.25$, $N_\beta=0.5$.

Based on the above settings, a set of dynamic analyses was carried out by applying a Gaussian white noise sampled at 100 Hz as external input. Then, the dynamic properties of the system were extracted by applying OMA techniques to the response simulated in positions corresponding to those of experimental measurements.

The optimisation procedure was based on the minimisation of two parameters: a. the first one was the frequencies scatter, defined by equation (1); b. the second one was th MAC index that provides the similarity of selected modal shapes, see Equation (2). The higher is the MAC index the more similar are the compared mode shapes [5].

$$\Delta f = \frac{f_{FEM} - f_{exp.}}{f_{exp}} \tag{1}$$

$$MAC\left(\{\psi_{FEM}\},\{\psi_{exp}\}\right) = \frac{\left|\{\psi_{FEM}\}^T \times \{\psi_{exp}\}\right|^2}{\left(\{\psi_{FEM}\}^T \times \{\psi_{FEM}\}\right)\left(\{\psi_{exp}\}^T \times \{\psi_{exp}\}\right)} \tag{2}$$

where f_{FEM} and f_{exp} are the numerical and experimental value of the natural frequency of the generic mode, respectively and $\{\Psi_{FEM}\}$ and $\{\Psi_{exp}\}$ are the numerical and experimental mode shape vectors for the generic mode, respectively.

Mode	f_{exp} (Hz)	f_{FEM} (Hz)	Scatter (%)	MAC
I	3.68	3.65	-0.81	0.994
II	7.23	7.32	1.24	0.997

Tab. 2. Dynamic identification results

Table 2 reports the results of the minimisation process corresponding to the refined model represented in Figure 5b and Figure 5c [6].

V. STATIC RESPONSE INTERPRETATION BY MEANS OF FE MODEL OPTIMISATION

The sub-soil optimised model provided by the dynamic characterisation of the system designated the first step for the interpretation of the data taken from static measurements.

Attention was mainly focussed on the assessment of the time-dependent response of the system as shown in Figure 2. In this context, the linear elastic assumption was not representative of the real conditions of the soil during the excavation process.

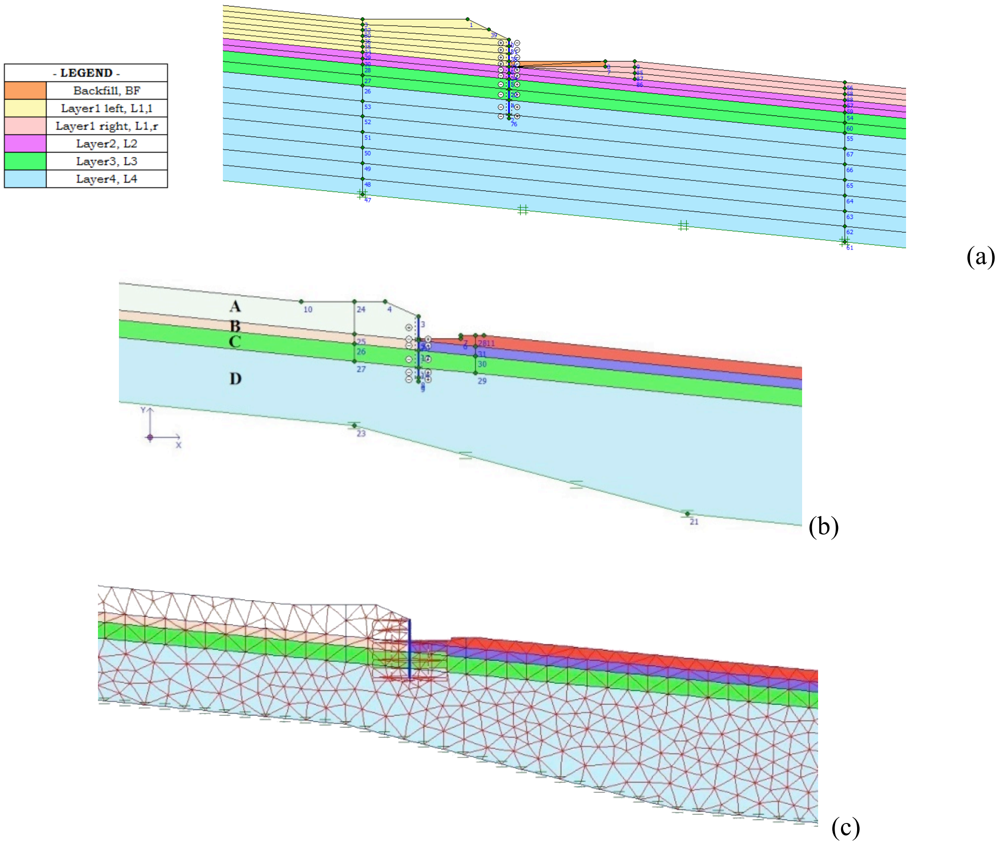

Fig. 5. Zoomed view of the basic geometry of the FEM model (a); optimised model (b) and correlated mesh (c).

The hardening soil (HS) model was then assumed as reference in the present study [7]. It is an elastic-plastic soil model capable to simulate the behaviour of both stiff and soft soils. In fact, in contrast to elastic-perfectly plastic constitutive model, double hardening (i.e. shear and volumetric hardening respectively dominant in OC and NC clays) can be accounted for, also in combination with a yield cap surface. Stress dependent stiffness moduli, according to a power law, and a Mohr-Coulomb failure criterion complete the basic characteristics of this model. HS model is based on hyperbolic relationship [8] between the vertical strains, ε_1, and the deviatoric stresses, q, in primary triaxial loading and, unlike to elastic models, it does not require a given relationship between drained triaxial stiffness and oedometric stiffness for one-dimensional compression.

A constant stiffness vs. depth relation was assumed for each soil layer (i.e. independent from stress-level). The basic soil parameters adopted in the FEM analyses are summarised in Table 3. In addition, it is worth noting the ν_{ur}, m and R_f parameters were assumed independent upon the layer and respectively equal to 0.2, 0.0 and 0.9. As the definition symbols is concerned, reference is made to background literature [7] for sake of brevity.

	γ_{unsat} [kN/m^3]	c' [kPa]	φ [°]	k_0=1-senφ	e_0	E_{50} [kPa]	E_{oed} [kPa]	E_{ur} =3E_{50} [kPa]
L1,l	18.0	22.0	22.9	0.611	0.86	3808	5126	11420
L1,r	18.0	29.0	18.0	0.691	0.86	3157	4250	9472
L2	19.05	22.3	21.3	0.636	0.90	4873	6559	14620
L3	19,5	26.0	20.0	0.648	0.81	15560	20900	46670
L4	20.0	19.7	23.9	0.595	0.69	27720	37320	83610

Tab. 3. HS parameters used for soil in static analysis

Linear elastic behaviour was confirmed for the piles that were analysed on the analogy with the procedure adopted for the dynamic analyses. The influence of the interface resistance (adhesion, c_a, and wall-soil friction angle, δ) has been also accounted for. The relevant parameter for the pile-soil interface R_{inter}=tgδ/tgφ was assessed by means of an effective interpretation of the available data.

In particular, adopting a back analysis procedure, that is typical in simulation response of geotechnical systems [9] the best value of R_{inter} was obtained by a preliminary optimisation process aimed at minimising the scatter between the FEM results and the first dataset measured in the field. A value of 0.683 was obtained; it fits really well many literature recommendations for this type of structures [10], i.e $\delta=2/3\varphi$ and $\delta=1/2\varphi$ respectively on active/passive soil wedge-wall contact. In addition the adhesion was introduced according to the relation $c_a=1/2c'$, limited to 15 kPa due to the softening of the ground during the execution of piles.

Once all the most relevant parameters were set, attention was paid to the time-dependent response of the system. A preliminary analysis of the results led to identify in the creep mechanisms the source of the increase of the displacements during time.

It is well known that soft soils, such as normally consolidated clays, clayey silts and peat, show high levels of compressibility and can undergo creep deformations under a constant effective stress state.

An appropriate understanding and prediction of this aspect was fundamental for a correct evaluation of the response of the most common applications in geotechnical engineering. However, the factors that influence the nature of this process are many and difficult to implement in numerical modeling of the soil behavior (soil composition and in particular the content of fine fraction, microstructure of the soil, the stress history, temperature changes and biochemical transformations, etc.).

The phenomenon related to the creep deformation of the solid skeleton of soils was taken into consideration by means of the soft soil creep model [4]; despite being very simple because it does not consider the soil anisotropy, it is easy to calibrate since it depends only a single parameter, namely C_α representing the slope of secondary consolidation curve.

Due to the lack of the direct representation of the creep behaviour of Vazzieri's soils, again an optimisation procedure was adopted. The aim was the definition of the optimal C_α and its validation by means of a reasoned comparison with literature available ranges [11, 12].

Creep was associated only to the most compressible layers of the soil, i.e. to the two shallowest strata L_1 and L_2. Results of this optimisation process, determining some changes of the values reported in Table 3 are given in the Table 4.

The comparison between the optimal C_α/C_c ratio and the available literature data led to recognise a good agreement with recommended values for the soil layers of interest. This circumstance confirmed the role of the viscous effects in the development of the observed response of the wall.

	C_c	C_s	C_α	C_α/C_c	ν_{ur}
L1,l	0.0835	0.0337	0.0025	0.03	0.29
L1,r	0.1007	0.0406	0.0030	0.03	0.29
L2	0.0666	0.0269	0.0018	0.027	0.29

Tab. 5: Optimised soft soil creep parameters.

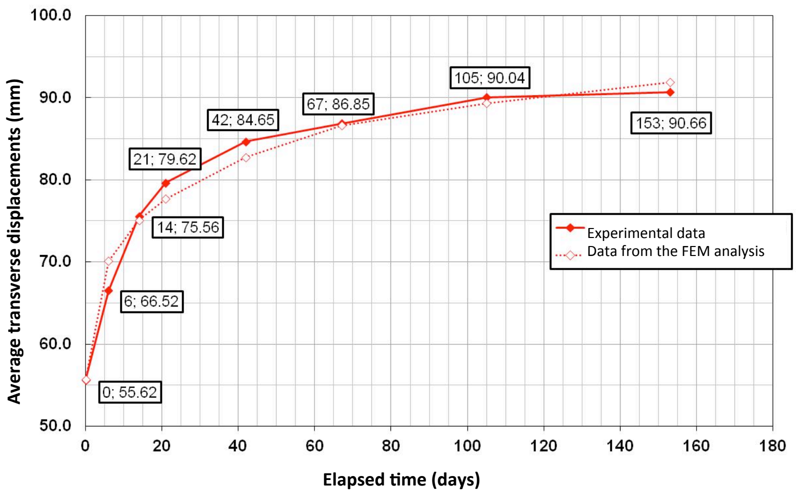

Fig. 6: Comparison between measured and optimal FEM wall deflection plotted versus time

Figure 6 and Figure 7 report the results of the optimised model. It is worth noting that the rational approach adopted for the model calibration appears to be effective. Modelling of the soil-piles response based on the dynamic identification appears to be able to provide a very effective support for the assessment of flexible wall retaining structures. The wall deflection versus time relationship provided by the calibrated numerical model showed a very good agreement with experimental data. Reliability of the model is associated in particular to a rational multi-level optimisation process that provided significant values of the parameters of interest, validated against available literature knowledge and real soil properties on site.

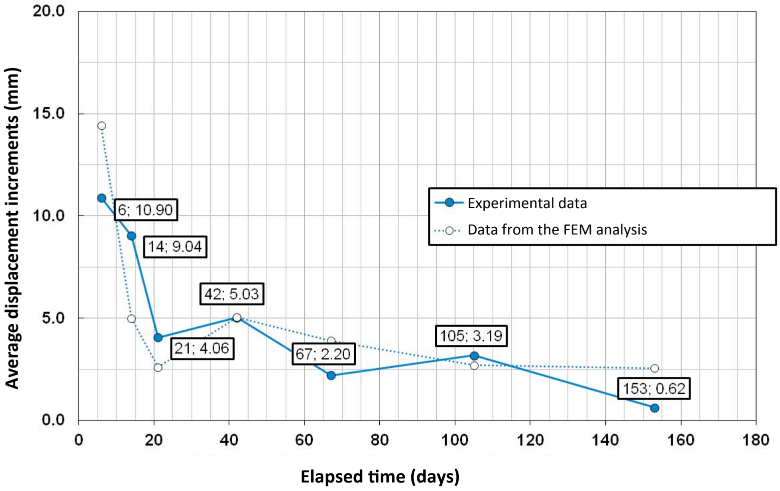

Fig. 7: Comparison between measured and optimal FEM wall incremental deflection plotted versus time

VI. CONCLUDING REMARKS

The presence of geotechnical structures in many urbanized areas exposed to seismic hazard represents a strong motivation for the knowledge enhancement in the field of the dynamic response of such complex constructions. In the present paper, attention has been paid to the flexible retaining walls, whose design techniques according to the seismic performance based design criteria are not fully established yet. In particular, the experience taken during the design, installation and validation of a static and dynamic monitoring system on a real structure was reported. The results obtained from vibration records under operational conditions were used to develop a rational approach to the calibration of a dynamic and static FE model able to shade light on the most relevant mechanisms affecting the response of such structures. Operational Modal Analysis techniques have been successfully applied to evaluate the fundamental dynamic properties of the wall. Correlation analyses provided a refinement of the model, so that it is now able to better reproduce the experimental results compared to its original design setting. Further developments of the research are however needed to confirm and extend such results as well as to refine tools for the analysis of complex soil-retaining wall interactions even under strong motion.

REFERENCES

[1] Visone C., Santucci de Magistris F. (2008). A review of design methods for retaining structures under seismic loadings. Strategies for reduction of the seismic risk (G. Fabbrocino & F. Santucci de Magistris eds.) pp. 51-64.

[2] Rainieri C., Fabbrocino G., Santucci De Magistris F. (2013). An Integrated Seismic Monitoring System for a Full-Scale Embedded Retaining Wall. Geotechnical Testing Journal, vol. 36.

[3] Rainieri C., Fabbrocino G. (2014), Operational Modal Analysis of Civil Engineering Structures An Introduction and Guide for Applications, Springer.

[4] Brinkgreve, R.B. (2006). Plaxis 2D Version 8.4: Reference, Scientific and Dynamics Manuals, Balkema, Lisse, The Netherlands.

[5] Allemang, R.J. and Brown, D.L. (1982). A correlation coefficient for modal vector analysis. Proceedings of the 1st IMAC, Orlando, FL, USA.

[6] Rainieri C., Dey A, Laorenza C, Fabbrocino G, Santucci de Magistris F (2011). Ambient vibration based modal identification of a flexible retaining wall. In: Civil Engineering Topics. vol. 4, p. 349- 356, Springer.

[7] Schanz T., Vermeer P.A., Bonnier P.G. (1999). The hardening soil model: formulation and verification. Beyond 2000 in computational geotechnics. 10 Years of Plaxis. Balkema.

[8] Duncan J. M. and Chang C. Y. (1970), Nonlinear analysis of stress and strain in soils, J. of Soil Mech. and Foundation Division, ASCE, 96 (SM5), pp. 1629-1653.

[9] Likar J., Vukadin V. (2003). Time-dependent back analysis of a multianchored pile retaining wall. J Geotech Geoenviron Eng ASCE, 129 (1).

[10] Clayton C.R.I., Milititsky J. and Woods R.I (1993), Earth Pressure and Earth-Retaining Structures, Blackie Academic & Professional.

[11] Mesri, G., and Castro, A. (1987). The Cα/Cc concept and Ko during secondary compression. Journal of Geotechnical Engineering Division 112(3), 230-247.

[12] Mesri G., Godlewski P.M. (1977) – Time and stress compressibility interrelationships . J. Geotech. Enging. Div., ASCE, 103(GT5), pp. 417-430.