BEHAVIOUR OF A FULL SCALE INSTRUMENTED FREE FLEXIBLE RETAINING WALL UNDER STATIC AND DYNAMIC CONDITIONS

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Abstract

The monitoring system on a full scale free flexible retaining wall and their first results are presented here. The embedded retaining wall has a free height of about 6 m with an overall length of 18 m. It is composed of two rows of reinforced concrete r.c. piles, 800 mm in diameter. Two piles have been instrumented with specially designed embedded piezoelectric accelerometers and conventional inclinometer cases. A FE model of the system was developed and implemented using PLAXIS code. Simulations under static and dynamics conditions were performed.

Introduction

A research activity is in progress at the University of Molise to shade some light on the behaviour of flexible retaining walls, in the framework of the performance-based design methods (Visone & Santucci de Magistris, 2009). A key element in this activity is the availability of a full scale instrumented wall to check and test design procedures and models (Fabbrocino et al., 2008).

Comparing to the static cases, a limited number of full-scale dynamic measurement systems are currently applied to geotechnical systems. The knowledge on their behaviour under seismic loading is mainly based on post-earthquake permanent deformations or upon the interpretations of physical or numerical models. Some case-histories on seismically monitored earth dams are reported by Sica (2003); soil-foundation-structure interaction under seismic loading is also studied (Steidl et al., 2004; Pitilakis et al., 2005).

The full-scale flexible retaining wall was equipped with monitoring instruments able to describe its static, dynamic and seismic response. Based on the collected experimental data and their comparison with the corresponding results provided by numerical analyses, a calibration of a numerical model of the wall has been started. The adopted values for the mechanical properties of soil have been obtained from conventional geotechnical investigations. Material models able to reproduce the actual soil behavior were employed in the analyses. A number of parameters have been calibrated in order to improve the agreement between the experimental and numerical results.

Site Characterization and Description of the Structure

The monitored wall is located at the Vazzieri site in Campobasso. Medium to high seismic hazard is related to the area. Some remarks on this topic and on the disaggregation of seismic hazard are reported elsewhere (Caccavale et al., 2010). The geology of the area includes deposits of varicolored scaly stiff clays, with alternation of thick levels of limestone, calcareous and sandy materials, covered by a layer of remolded man-made debris cover Two

boreholes were carried out in the area of pile wall construction: for each borehole, the investigations included stratigraphic columns, standard penetration tests SPT, laboratory tests on undisturbed samples and Down-Hole tests. Rainieri et al. (2010) described the results of the soil investigation, making a comparison between the values obtained in the different tests.
Based on the laboratory and in situ investigations, a simplified geotechnical model was adopted, whose main characteristics are reported in Table 1.

Table 1. Soil parameters.

Soil layer	E_{50} (kPa)	E_{oed} (kPa)	c' (kPa)	ϕ (°)	γ (kN/m^3)	γ_{sat} (kN/m^3)
S1	4700	6350	22	23	19.2	19.4
S2	15900	20220	28	17	19.5	19.8
S3	28900	38900	20.5	23	19.8	20.3
BR	35200	47400	19	24	20.1	20.5

No water table is present.
The structure is a free sheet wall, composed by a two alignment of adjacent but not contiguous piles (Figure 1). Each pile has diameter d=80 cm and spacing between the pile axis i=90 cm (Figure 2). The wall had total height of L=18 m (the wall free height is 6.4 m in the construction phases and 4.2 m in the exercise phases).The connection between the piles was obtained through a top beam (h= 1.5 m).
Reliable values of the system piles-beam stiffness were obtained (Fabbrocino et al., 2008), considering a 1 m band of the wall, in which about two piles were included.

Figure 1. View of the northern part of the wall

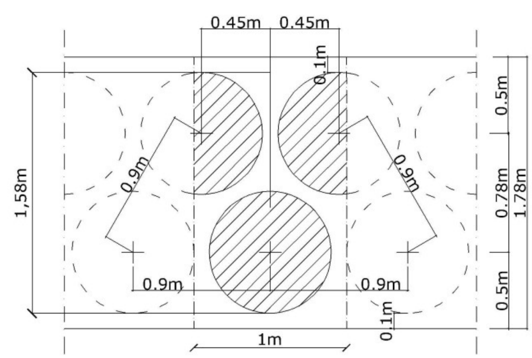

Figure 2. Layout of sheet pile wall

Monitoring systems and measurements

Two piles located in the central part of the wall, to avoid as much as possible boundary effects, were equipped with monitoring instruments.

Static

Measurements of horizontal displacements of the wall were carried out using inclinometers. Six inclinometer casing were used, three for each piles.
The experimental values for the inclinometers in the different piles gives very similar values (Rainieri et al., 2010): for this reason average values of the measurements were plotted against depth in Figures 3, including the timing of the wall deflection measurement. To

analyze data, the bottom of the inclinometer hole was assumed as a fix point. Data reported in this figure are re-oriented normal to the wall plane (where the highest movements are expected). In the lacking of absolute measures of orientation of the inclinometer casing, possible only at top of the tube, the azimuth of the inclinometer tube was identified assuming that the first measurement (on September 4th, 2008) has nil components in the plan of the wall. In figure 4, the value of displacement at the top of the wall is plotted versus time.

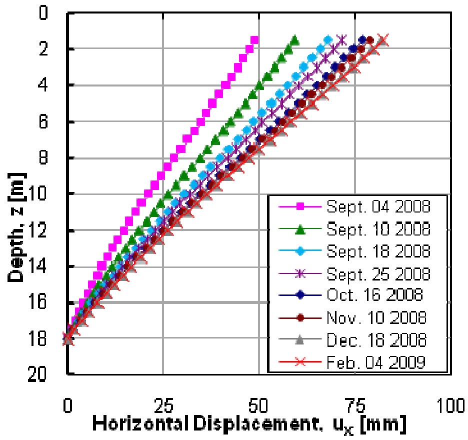

Figure 3. Average horizontal displacement normal to the wall plane

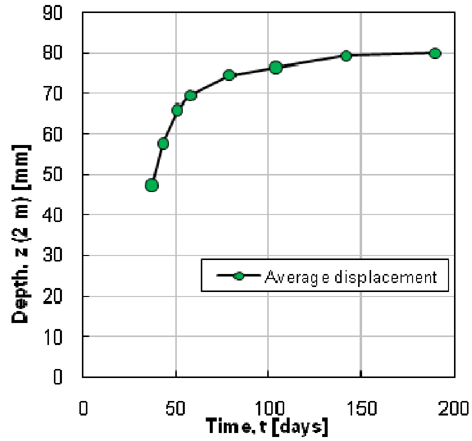

Figure 4. Evolution of the horizontal displacement with time (top wall)

The wall deformation is very similar to a rigid rotation while the gradient of the displacement with time tends towards zero. Absolute values of the displacement appear to be very large if compared with a collection of case-histories by Clough and O'Rourke (1990), Long (2001) and L'Amante (2009) as indicated in Figure 5.

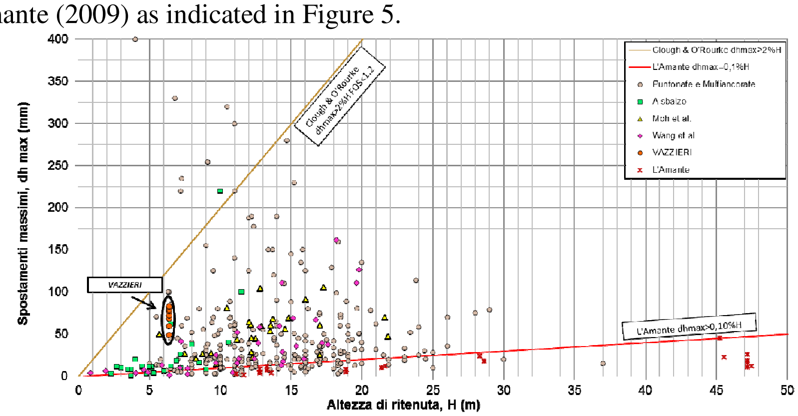

Figure 5. A collection of case-hisories of maximum displacement versus the wall free height (after Long 2001).

Dynamic

Some embedded accelerometers have been installed in the two monitored piles (Fabbrocino et al., 2008). Accelerometers are placed in sensor moduli consisting of two seismic, high sensitivity (10 V/g) ceramic shears integrated circuit-piezoelectric, ICP, accelerometers, placed in two orthogonal directions and encapsulated in a stainless steel enclosure which assures impermeability and protection against concrete pressure. Three sensor moduli have been placed at different depth in each pile. Additional reinforcement close to each sensor modulus has been designed to obtain piles characterized by similar strength and flexural

stiffness to those ones of the adjacent piles. Two additional sensors have been placed in the top beam.
Vibration records in operational conditions are regularly carried out. Operational Modal Analysis techniques (Rainieri, 2008) are being applied to records of the dynamic response of the wall due to ambient vibrations to evaluate its fundamental dynamic properties. Even in presence of very low levels of ambient vibration, the first two fundamental frequencies of the wall, at 3.68 Hz and 7.23 Hz respectively, have been identified with a certain degree of accuracy.

Numerical analyses

The numerical analyses have been carried out through the Plaxis 2D v.8.4 code (Brinkgreve, 2002). Static measurements are analyzed first and then the model was updated using information coming out from dynamic measures.

Static

The domain used in the analyses is shown in Figure 6. It reproduces the site conditions before the excavation. Details are provided in Dey et al., 2011.

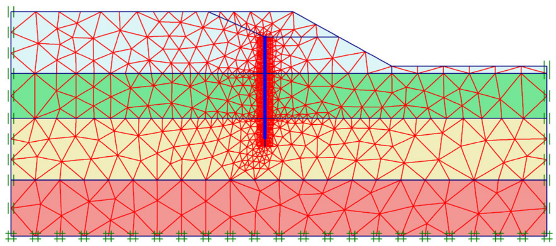

Figure 6. Mesh used for the static analyses

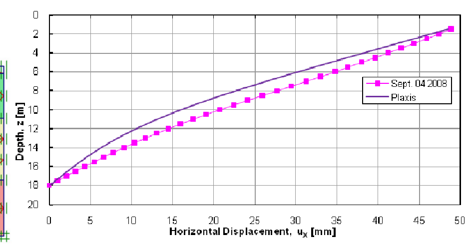

Figure 7. Comparison between the first measurement and the numerical analyses results

The soil behavior has been modeled by the Hardening Soil model (Schanz et al., 2000). The structure has been modeled as a plate, with an equivalent thickness calculated from the values of the bending (EI) and axial stiffness (EA) by considering the Poisson ratio to be zero.
The interface between soil and structure is assumed to be rigid, because the ratio (δ/ϕ) between the soil-structure roughness angle and the friction angle of the soil can be considered equal to 1 when the structure is made of concrete.
The experimental data have been compared to the results of the numerical analyses, as depicted in Figure 7 corresponding to the first on-site measurement (Sept. 4th, 2008). The plot shows a reasonable agreement between experimental and numerical data.

Dynamic

The basic configuration of the preliminary static model has been substantially modified to account for the typical problems in handling dynamic cases through finite element approaches (Dey et al., 2011) and to account for the results of the dynamic experimental measurements.
In dynamic numerical analyses, several key aspects have been taken into consideration, including:
(a) Material Models and Material Properties: the soil is modeled as a linear elastic material. The elastic modulus of the soil has been chosen to be conforming to the initial tangent modulus, E_0, as obtained from the Down-Hole tests.
(b) Domain Width: a larger domain is chosen to minimize interferences between waves

and boundaries of the model. A domain aspect ratio (ratio of total domain width to the average height) greater than 40 is chosen.

(c) Meshing: A trade-off is required to obtain a nearly accurate solution (fine meshing) in reasonable less time (coarse meshing). In this study, the global meshing is achieved with a medium mesh, the central part of the model is then refined.

(d) Input Excitation: A horizontal prescribed displacement of 0.1 m is used along the bedrock level, which is subsequently excited by a Gaussian white noise of 1 h length. Impulse loading of 0.01 s was also adopted.

(e) Rayleigh Damping Parameters: For the model used in the present study, the Rayleigh damping parameters are R_{α}=0.293, R_{β}=0293 both for the soil and the structure.

(f) Newmark Parameter: An undamped Newmark scheme, also known as average acceleration scheme, has been considered (so that the predicted frequency spectra suffer minimal effect from numerical damping) with the following parameters: N_{α}=0.25 and N_{β}=0.5.

(g) Absorbent boundaries: Although the model vertical boundaries are chosen to be present far away from the region of interest, standard viscous adsorbent boundaries having C_1= 1 and C_2 = 0.25 are chosen for the relaxation coefficients in order to obtain the highest wave absorption and minimize the possible reflection of the waves.

A series of model refinements, described in detail elsewhere, were employed so that the results of the numerical model could match both natural frequencies and modal shapes experimentally measured. This approach derives from structural engineering and was successfully employed here for a geotechnical structure.

This process brings us to modify:

1) the inclination of the soil strata from horizontal to an approximate slope (*1V:10H*);
2) the depth and the profile of the bedrock; and,
3) the elastic properties of the two upper layers on the left-hand side of the wall.

Material properties for the refined models are in Table 2; geometry and meshing of the adopted numerical model are in Figure 8; modal identification results and correlation with the FE model are in Table 3.

Soil Layer	Material Type	E_{0L} (10^5 kPa)	E_{0R} (10^5 kPa)	N	γ (kN/m^3)	H_L (m)	H_R (m)
A	LE	1.565	3.188	0.432	18.00	8	3
B	LE	3.482	4.086	0.426	19.03	3	3
C	LE	14.86	14.51	0.438	19.47	5	5
D	LE	26.49	26.49	0.433	19.98	19	35

Table 2. Material properties adopted in the FE model (L= left side of the wall), (R= right side of the wall)

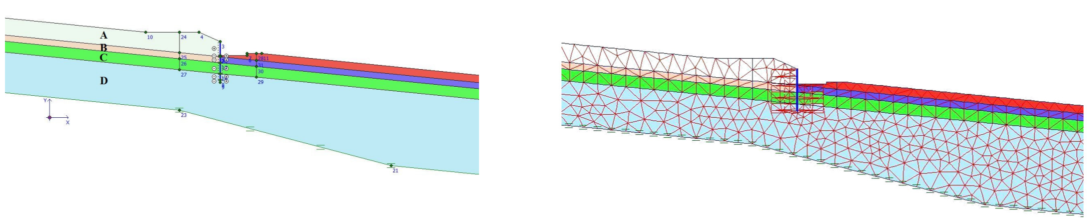

Figure 8. Geometry and meshing of the adopted numerical model.

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

Table 5. Modal identification results and correlation with the FE model.

With the assumption here indicated, the matching between experimental and numerical analysis is very satisfying, thus confirming the precious added value given by a proper designed monitoring system in calibrating an enhanced dynamic FE model.

Conclusion

A FE model of an embedded wall was calibrated, based on the results of a proper designed monitoring system, allowing detecting the performance of a real geotechnical structure under static and dynamic loading. The model was refined based on the dynamic measurement under operative conditions showing very good performances. The same model would be employed in the near future to: 1. check again the performance of the system under static loading; and, 2. investigate the behaviour of the structure under earthquake loadings.

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