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BEHAVIOUR OF A LARGE ANCHORED BASEMENT IN DENSE SANDS AND GRAVELS

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ABSTRACT

Prior to the excavation of half a million cubic meters of dense gravelly material within the city centre of Milan, a 24 to 27m long diaphragm wall was built to provide a 17m retained height to a four-level basement. Temporary support was offered by two to four rows of ground anchors with the exception of a portion adjacent to an existing multi-level cark park where a post-tensioned capping beam was installed to protect the car park and prevent it from being subject to sway. Other design challenges were posed by the proximity of a buried channel and of an urban rail tunnel.

An extensive monitoring system was set up including inclinometers, load cells and topographical survey points to reveal lateral movements in the order of 10mm or less, negligible variation of anchor loads as the excavation progressed and heave behind the wall. In response to a general lack of case histories and design guidance on deep excavations in dense coarse material, especially in the region, a backanalysis of diaphragm wall monitoring data was carried out and its main results are commented on in the present paper. Lateral movements are best reproduced with pseudo-FE or full FE software if a soil stiffness compatible with the relevant shear strain level is adopted.

INTRODUCTION

Underground works such as excavations in an urban environment may induce settlements in existing buildings and adjacent infrastructure. A reliable estimate of the retaining structure performance is therefore essential to mitigate the risks associated with construction. This paper presents the observed performance of a 24m to 27m long diaphragm wall (DW) during the excavation of a 330m x 95m box for a four storey basement. The site is in central Milan, forming part of a redevelopment project called "Varesine" located in a former railway station area. The paper also describes the geotechnical context of the site, in response to a general lack of publications on soil characterization in the area.

GROUND CONDITIONS

The site is situated within the Padana Plain in northern Italy, underlain by a 100m thick deposit of Quaternary alluvial granular material, the "Padana Plain main formation". This consists of an Upper Pleistocene coarse sand and gravel unit up to 30 meters below ground level which governs the design of the works. At the site a layer of made ground up to 7m thick is present over the sand and gravel unit.

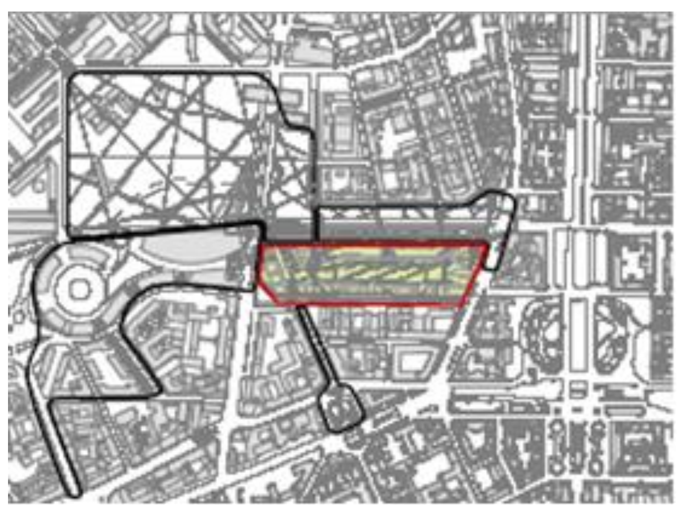

Fig. 1. The Varesine site is located in Milan's city centre.

The typical stratigraphy is shown in Fig. 2. A 2m thick clayey silt layer is present 30m below ground level between 93m and 90m above sea level (asl).

The site investigation included continuous coring of 50m deep boreholes with associated collection of undisturbed samples and laboratory testing (PSD analyses, Atterberg limits, oedometer tests and CU triaxial tests on samples collected within the clayey silt unit). SPT tests and Lefranc (falling head) permeability tests were also carried out.

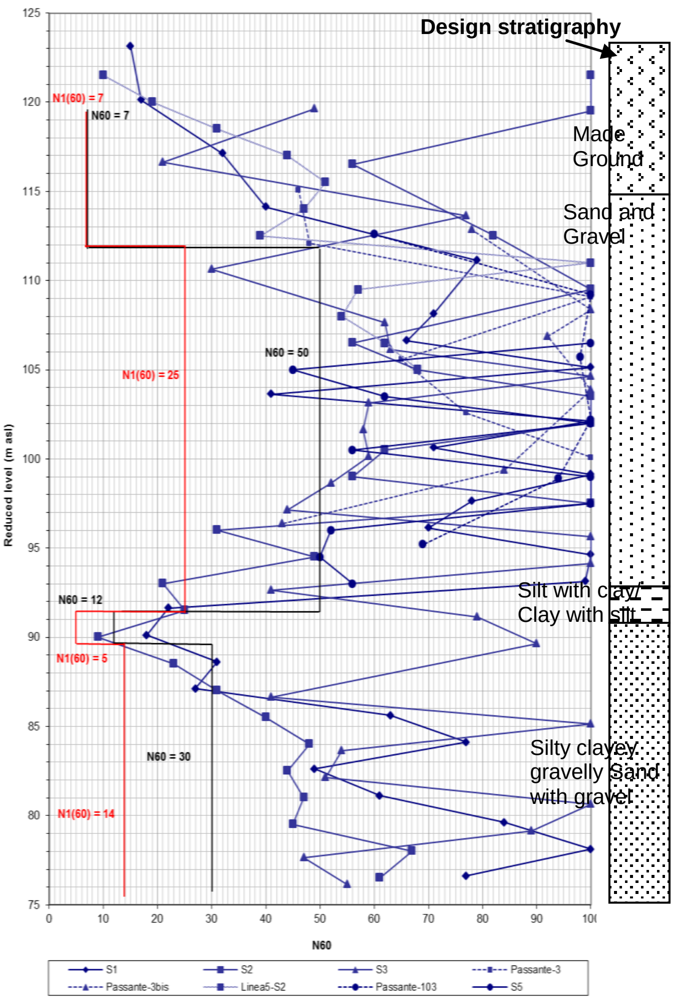

Fig. 2. SPT results (N value recorded on site). Design N_{60} and $(N_1)_{60}$ profiles are shown in black and red.

Fig. 3 shows the results of the PSD analyses from which four main families of grading are evident for the made ground, the upper sandy and gravelly layer, the cohesive layer and the deeper sandy layer, respectively.
The plasticity chart is depicted in Fig. 4.

The groundwater table was detected at an average depth of 20m below ground level (reduced level 104m asl) within four standpipes, corresponding to an unconfined aquifer, although perched aquifers have also been detected in the area where lenses of cohesive material are encountered within the sand and gravel layer.

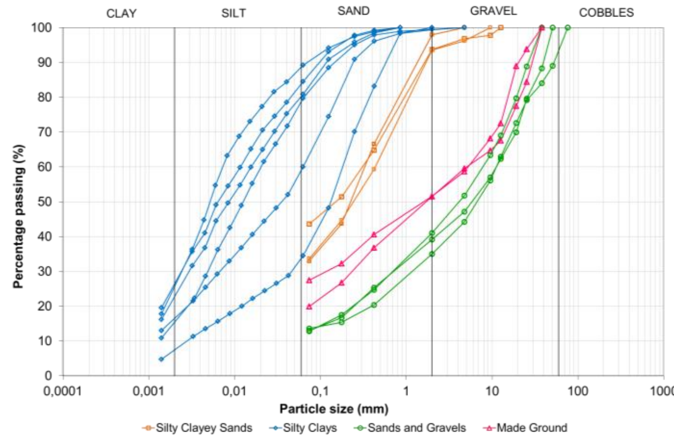

Fig. 3. Particle size distribution.

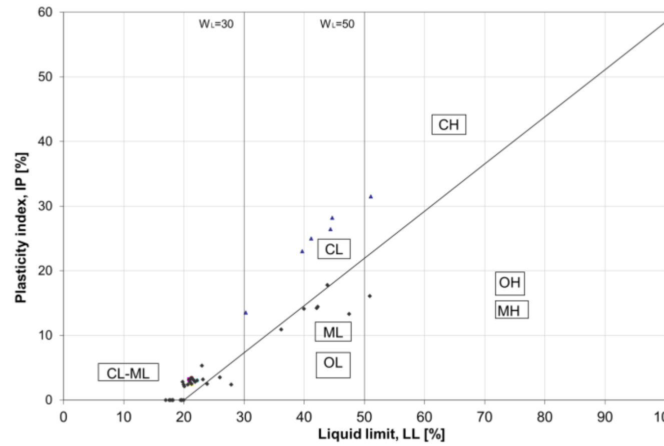

Fig. 4. Plasticity Chart.

Table 1. Geotechnical parameters.

Parameter	Made Ground	Sand and Gravel	Silt with Clay/ Clay with Silt	Silty/ clayey Sand
γ (kN/m3)	20	21	21	21
PI (%)	0	0	15	3
w (%)	N/A	N/A	23	N/A
N_{60}	7	50	12	30
$(N_1)_{60}$	7	25	5	14
ϕ' (°)	31	38	25	35
c' (kPa)	0	0	0	0
ϕ'_{cv} (°)	30	35	N/A	~33
D_r (%)	30	80	N/A	60
c_u (kPa)	N/A	N/A	100	N/A
K_0	0,48	0,38	0,58	0,43
E' (MPa)	10,5	75	N/A	N/A
ν' (ν_u)	0,2	0,2	0,2 (0,5)	0,2

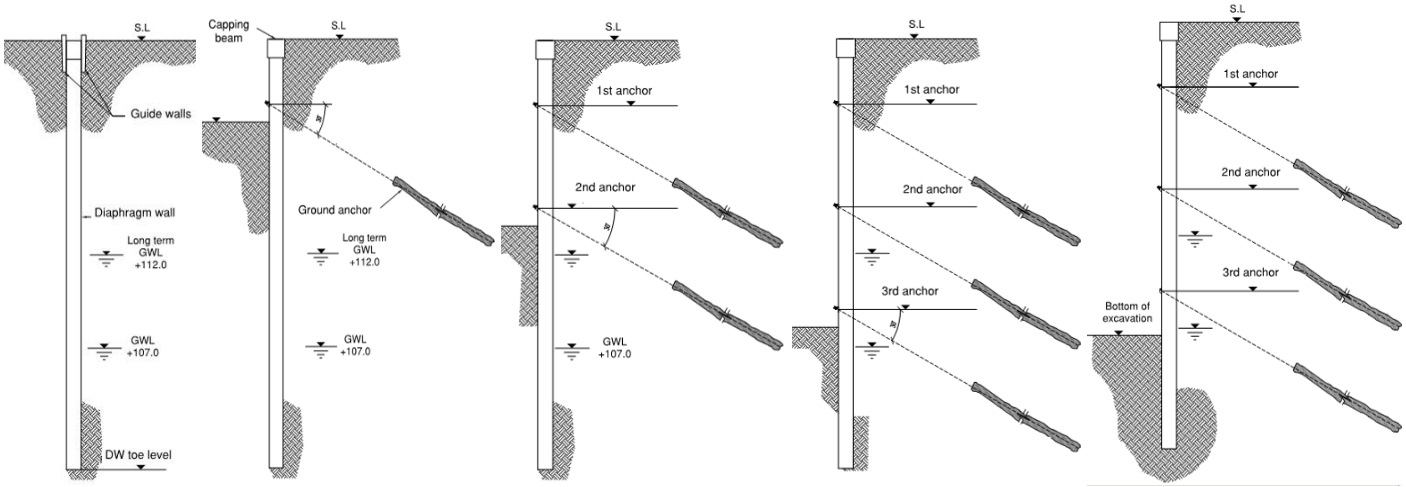

Fig. 5. Anchored diaphragm wall construction sequence.

Soil strength and stiffness parameters where inferred from published empirical correlations (Stroud, 1989) assuming an OCR of 1 based on the geological history, and are summarized in Tab. 1.

RETAINING WALL DESIGN AND PERFORMANCE

Diaphragm wall design

A 0.60m to 1.20m thick, 24m to 27m long diaphragm wall was designed to retain an up to 17m deep excavation. The support system of the wall consists of two to four rows of temporary ground anchors in the construction stage, and four reinforced concrete basement slabs in the permanent condition.

Wall friction on the active side was disregarded due to the friction caused vertical loads exerted by the ground anchors, while on the passive side a δ/ϕ ratio was assumed equal to 0.8.

A 25°C temperature variation was assumed and the corresponding thermal force acting on the basement slabs was included in structural design checks.

Eurocode 8 and Italian OPCM 3274/2003 were applied in the seismic design, with dynamic earth pressure on the wall calculated using the Wood theory. A short return period (T_r=10 years) design earthquake was considered as an accidental seismic event during the construction period. Nevertheless, the seismic case did not govern the design, as the site falls within an area with low seismicity.

Building damage assessments were performed and the maximum damage category (after Burland, 1995) for the adjacent (~10m) buildings was found to be equal to 2, which corresponds to “slight”.

The design analyses of the embedded retaining wall were carried out using Oasys FREW. The FREW program analyses soil-structure of a flexible retaining wall; it allows rapid analysis using elasto-plastic soil behaviour and stiffness matrices derived from finite element results. Three stiffness matrices relating nodal forces to displacements are developed: one represents the wall in bending and the others represent the soil on each side of the wall. The soil behaviour is modelled as an elastic continuum relying on the Mindlin method, with the soil stiffness based on the integrated form of the Mindlin equations and plastic limits defined by EC7 earth pressure coefficients. Full details of the analytical model can be found in Pappin et al. (1986).
The geotechnical design standard used was Eurocode 7, adopting Design Approach 1 for the ultimate limit state condition with partial factors applied to soil parameters, resistances and actions.

Monitoring system

The monitoring system installed on and around the site included the following devices:

- 28 inclinometers installed within diaphragm wall panels.
- 50 load cells installed on temporary anchors.
- 12 arrays perpendicular to the diaphragm wall comprising 5 topographical target points each for measuring vertical settlements and horizontal displacements of the ground on the active side of the retaining wall.
- 4 piezometers for groundwater level measurements.
- Optical prisms and crack width measuring devices on adjacent existing buildings.

The instrumented sections which have been back-analysed are shown in Fig. 6 as “Section 2 – DW 1C – IN7” and “Section 3 – DW 11B – IN9”.

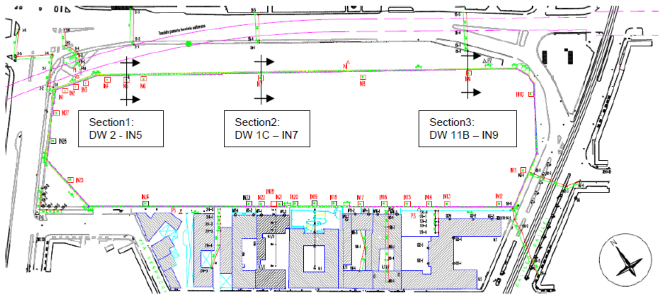

Fig. 6. Monitoring plan. The red and light green markers show the position of inclinometers and load cells.

Measurements

For all the 28 inclinometers, the observed horizontal displacements may be summarised as less than 8mm towards the excavation, which corresponds to less than 0.04% of a retained height of ~17m. This falls within the lower bound of the data presented by Clough and O'Rourke (1990), and by Long (2001) suggesting good behaviour of retaining structures embedded in Milan sands and gravels, see fig. 8.

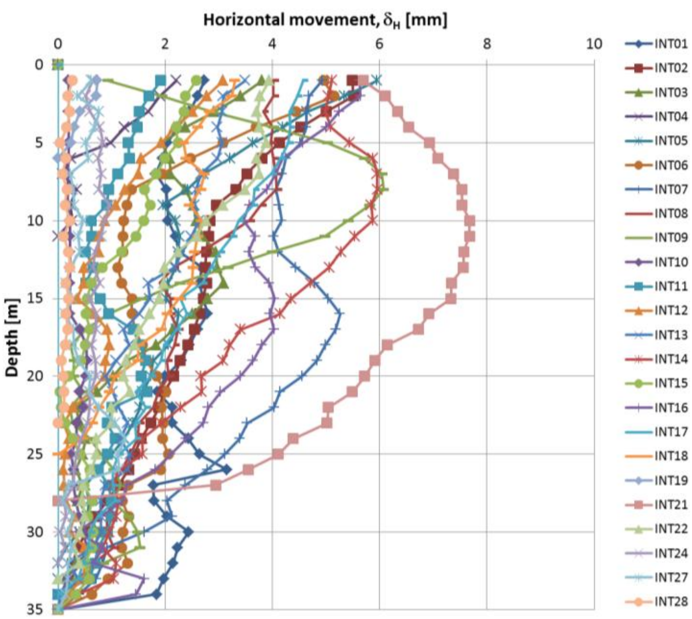

Fig. 7. DW horizontal movements from inclinometer readings.

The observed vertical movements may be summarised as a heave of 5 to 8mm close to the walls, see fig. 9. It is considered that this may be due to monitoring having started only after wall installation and/or to installation and stressing of the ground anchors, taking into account that pressure grouting was not adopted during the installation of the anchors.

A generalised heave due to the excavation of the large construction site for the redevelopment of the Garibaldi-Repubblica area has been also reported in the area. This aspect is not included in the present study and merits further investigation.

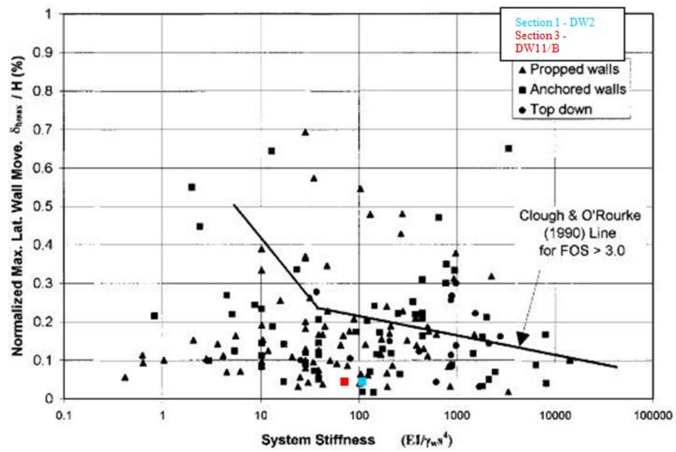

Fig. 8. Normalised lateral movement (after Long, 2001).

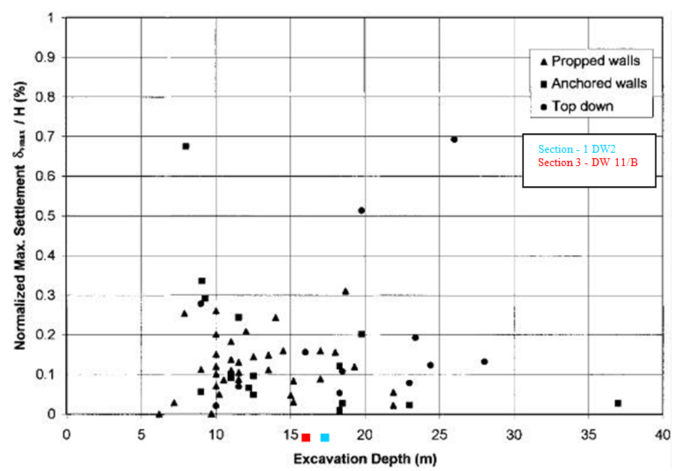

Fig. 9. Normalised vertical movement (after Long, 2001).

The observed movements are consistent with another Arup project in Milan (unpublished), for which heave due to anchor installation is understood to also have occurred, although quantitative data for this site are not available. Heave caused by the installation of seven rows of ground anchors in the Repubblica station for the Passante line is also documented by Amagliani *et al* (1991) for unspecified ground conditions, but believed similar to those at the Varesine site.

Barla *et al* (1986 and 1989) in publications relating to construction of the Milan Metro system provide information on ground parameters (N=20-50 at 10m depth, N~50 from 20 to 30m depth) and report similar wall horizontal displacements (convergence from opposite walls between 4 and 10mm), although a jet grouted curtain of unknown thickness was executed behind one retaining wall and no settlement trough behind the wall is presented.

BACK-ANALYSIS OF GEOTECHNICAL MONITORING DATA

Monitoring data from the instrumented sections were selected for back-analysis purposes and cover the whole construction stage until the maximum excavated depth was reached. Inclinometer readings and ground monuments surveys provide wall horizontal movements and vertical movements behind the walls for the section studied.

A summary of the diaphragm wall and anchor design properties at the backanalysed sections as well at section 1 is shown in Tab.2 and Tab.3.

Table 2. Diaphragm wall panel geometry

Section	DW type	Inclinometer	DW top level (m asl)	DW toe level (m asl)	DW thickness (m)	Retained height (m)	DW length (m)
2	1C	IN7	123.15	105. 5	0.8	17.15	23
1	2	IN5	123.55	97.5	1.2	17.55	26
3	11B	IN9	120.8	99.5	0.6	14.80	21.3

Table 3. Ground anchor properties

DW type		1C	2	11B
1st row	Inclination [°]	25	17	20
	Pre-load [kN/m]	200	306	160
	Stiffness [kN/m/m]	2727	2986	2245
2nd row	Inclination [°]	25	13/8	20
	Pre-load [kN/m]	340	306	255
	Stiffness [kN/m/m]	3816	6543	3180
3rd row	Inclination [°]	25	-	20
	Pre-load [kN/m]	340	-	297
	Stiffness [kN/m/m]	4200	-	3515

Use of pseudo-FE software

2D pseudo finite element analyses were carried out for the back-analysis, using Oasys FREW to model the observed construction sequence. A set of sensitivity analyses were performed to assess the influence of structural parameters, soil strength parameters and soil stiffness profile: a good agreement with measured data was obtained for all the backanalysed sections in terms of horizontal deflection. The comparison between measured and computed horizontal movements for DW 1C is shown in Fig. 10.

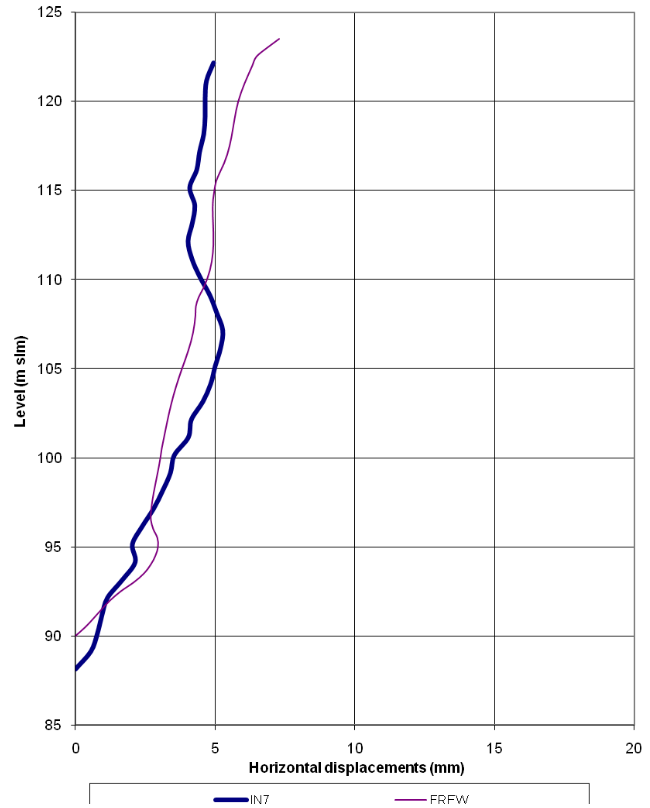

Fig. 10. Computed versus measured horizontal movement of DW 1C (inclinometer IN7).

Use of FE software

2D FE back-analyses were also performed with PLAXIS, using the geometry and the material strength sets selected in the design stage.

In the analyses, the Hardening Soil model was adopted as the soil constitutive model because of its ability to reproduce the increase of soil stiffness with depth (i.e. mean effective stress) in granular materials together with a higher soil stiffness profile in unloading conditions. The unloading-reloading stiffness parameter E_{ur} was derived from unloading considerations based on construction sequence and an E_{ur}/E_{50} ratio equal to 2 was conservatively applied, while the oedometer stiffness E_{oed} was assumed to be equal to E_{50}.

On the basis of the results of a specific sensitivity analysis, the dilatancy was set to zero: its negligible effect may be due to the system behaviour being far from failure conditions.

Table 4. Back-analysed ground parameter set

Parameter	Made Ground	Sand and Gravel	Silty/Clayey Sand and Gravel
γ_{unsat} [kN/m^3]	20	20	20
γ_{unsat} [kN/m^3]	20	20	20
E_{50ref} [MPa]	100	200	170
E_{oedref} [MPa]	100	200	170
E_{URref} [MPa]	200	400	340
ϕ [°]	31	38	38
c' [kPa]	0	0	0
δ/ϕ [-]	0.67	0.67	0.67
ν_{ur} [-]	0.2	0.2	0.2
p_{ref} [kPa]	100	100	100
m [-]	0.5	0.5	0.5
K_{0NC}	0.50	0.40	0.40

Discussion

The comparison below between active, passive and at rest lateral earth pressures from the various analyses in fig. 11 below shows the predicted earth pressures on the active side to be equal to the at rest conditions where ground anchors are present and full passive resistance to be mobilised within the first 3 metres below the base of the excavation on the passive side.

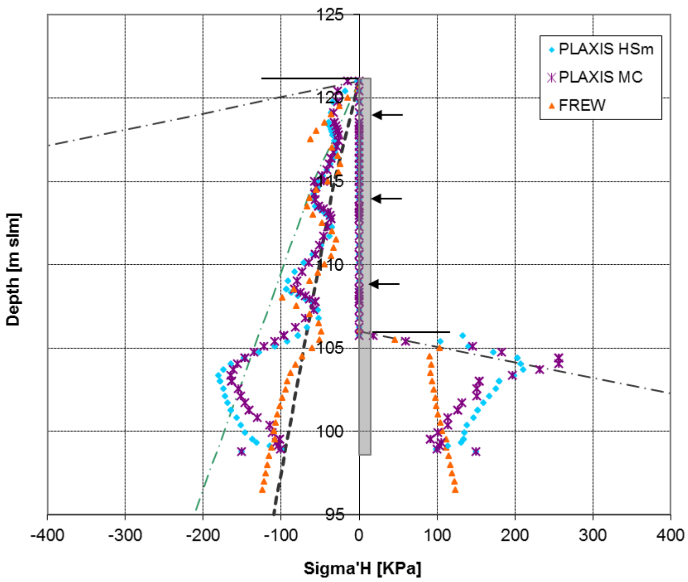

Fig. 11. Computed versus empirical horizontal earth pressures on DW 11B, section 3.

The observed wall behaviour was found to be best reproduced in the numerical analyses when an increased stiffness with respect to that chosen during the design stage is adopted: more specifically, the input values for the numerical analyses were derived after Seed and Idriss (1970) stiffness degradation curves assuming an average shear strain equal to 0.02%, consistent with measured wall movements.

The predicted average strain level is lower than that assumed at the design stage where a Young's modulus of 75 MPa for the Sand and Gravel layer was adopted from the Stroud correlation. These lower strains are consistent with the higher back-analysed stiffness of the upper sand and gravel layer of approximately E_{50} = 200 MPa at a reference stress of 100 kPa.

Lateral wall movements were thus well reproduced with the Hardening Soil model parameter set used in the FE analyses; this constitutive model is particularly suitable for a staged construction sequence as it correctly reproduces the load history and the unloading process. The lateral movements from the inclinometer readings match the computed wall horizontal movements, whilst the FE analyses results are less consistent with measured data in terms of vertical movements behind the wall.

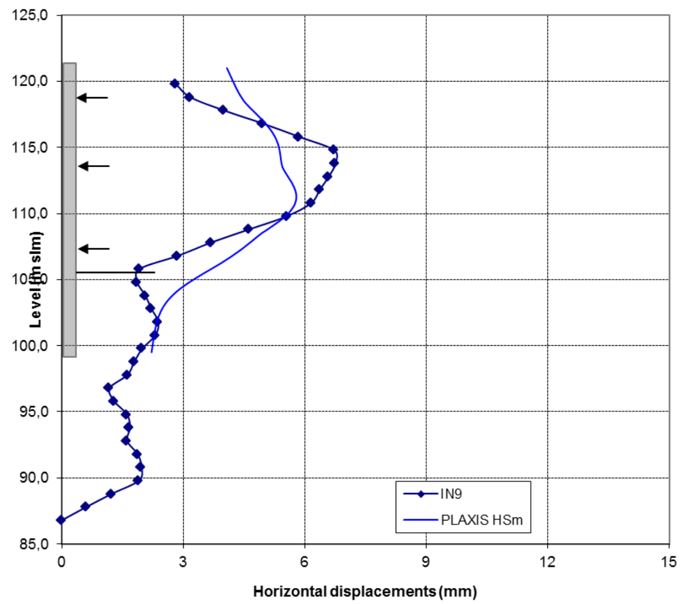

Fig. 12. Computed versus measured horizontal movements of DW 11B (inclinometer IN9), section 3.

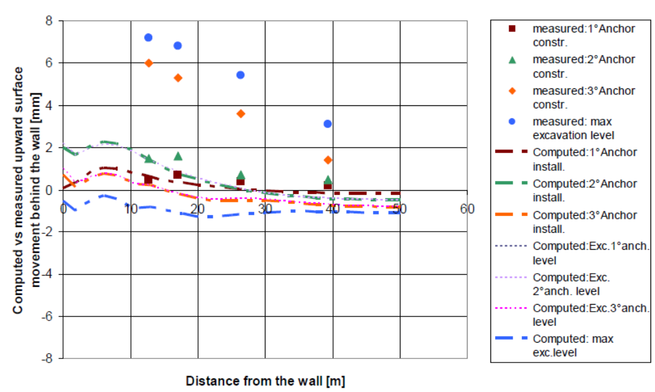

Fig. 13. Computed versus measured vertical movement behind DW 11B.

The computed upward movements of the first two excavation stages (i.e. after installing the second row of anchors) satisfactorily matches the observed movements. As the excavation progresses, the predicted upward movement reduces in the back-analyses: this is substantially different from what was measured on site, as the ground monument data show a progressive increase in the soil upward movement. It is also worth noting that the upward movement continued to increase up to a value in excess of 7mm after the maximum excavation depth was reached (March 2008), albeit at a much lower rate. Heave behind the wall is believed to be due to a combination of anchor grouting and pre-stressing, as observed in another Arup project in the Milan area, although the effect of grouting has not been included in the numerical model.

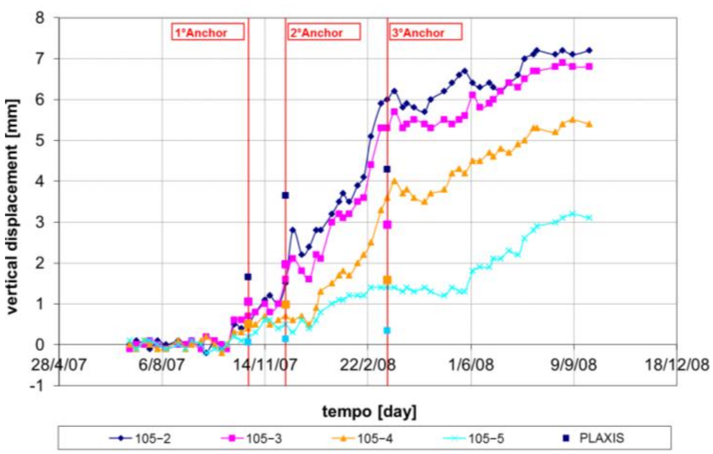

Fig. 14. Elapsed computed versus measured vertical movement behind the DW 11B.

It was also observed that the anchor loads remained substantially constant throughout the various construction stages. This result was also obtained from the FREW and PLAXIS models as shown in Fig. 15 which compares the anchor load measured during nine months of construction at DW 11B with the corresponding computed values.

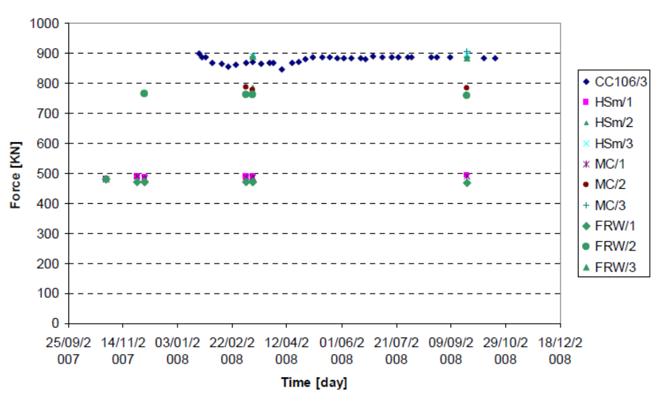

Fig. 15. Comparison between measured anchor loads at DW 11B and computed loads with FREW and PLAXIS.

CONCLUSIONS

The present study was driven by the lack of case histories in the Milan area on the behaviour of embedded multi-anchored retaining walls and by the object of determining the soil parameter(s) affecting the lateral displacements of the wall in similar ground conditions, as the observed movements at the Varesine site were lower than estimated at design stage.

The diaphragm wall thickness varies between 0.6m and 1.2m across the site. The wall has a typical retained height of 17m and an embedment of 5m or more. In the temporary situation, its lateral stability was provided by two to four rows of ground anchors which were later progressively destressed after the basement slabs were constructed.

The monitoring system included inclinometers and topographical survey points located at ground level at the back of the wall as well as load cells to monitor the evolution of anchor loads.

Two wall sections were back-analysed with FREW and PLAXIS and studied by means of a sensitivity analysis.

Out of all the design parameters considered in the sensitivity analysis, the soil stiffness appeared to have the most relevant effect on the wall behaviour. The wall lateral displacements were best reproduced by using Seed and Idriss (1970) stiffness degradation curves to derive the soil stiffness at the relevant shear strain level as well as PLAXIS Hardening Soil model.

The original design stiffness for the sands and gravels layer was derived from Stroud (1989) (E'=75MPa) whereas that obtained from the Seed and Idriss correlation and a shear strain of 0,02% is 200MPa at mid-height.

Heave behind the wall could not be fully reproduced in a PLAXIS model with a staged excavation and ground anchor pre-stressing, especially in the final stages of the excavation; this might be partly related to grouting during ground anchor installation but there currently are not enough data to further investigate this postulation.

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