Geotechnical Characterization for the Magneti Marelli Factory in Crevalcore (Bologna): DMT, CPTU and Laboratory Tests Comparison

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ABSTRACT: The present paper concerns the geotechnical aspects regarding the reconstruction interventions of the Magneti Marelli factory, located in the Municipality of Crevalcore (BO). Great relevance will be reserved to the interpretation of investigations and the definition of the vertical profile of the most important geotechnical parameters, focusing on the comparison between DMT, CPTu and laboratory tests results in the definition of geotechnical model. The seismic ground response analyses, that were conducted employing 1D EERA and 2D Plaxis Dynamic calculations, will be summarized in the second part of the paper.

1 INTRODUCTION

The 2012 Emilia-Romagna earthquakes caused several damages to the structures of the Magneti Marelli factory, located in the Municipality of Crevalcore (Bologna). Indeed it was necessary to design strengthening interventions and seismic improvements of the structures.

The whole geotechnical and structural design of the interventions was committed to Teleios Srl engineering company.

Those earthquakes showed relevant site effects, so geotechnical aspects had a primary importance for the design.

As part of the assignment before mentioned, a wide campaign of surveys was planned for the geological and geotechnical characterization of the intervention area.

During a preliminary evaluation it was found the need to employ deep foundations for the new structures because of the characteristics of soil and actions that those new earthquake resistant structures discharge at the base.

In addition, a seismic ground response analysis was conducted to obtain site specific design tools, such as response spectra and accelerograms, that replace the use of the simplified methods proposed in the Italian technical code NTC (2008).

From the geotechnical point of view the work provided: design and execution of the soundings, geological characterization, assessment of liquefaction susceptibility of soils, geotechnical characterization for seismic ground response analysis and design of deep foundations, execution of the analysis and project of the new foundations.

The present paper will describe the surveys and their interpretation, the geotechnical model and the results from site effect analysis.

Particular attention will be provided to the use of the flat dilatometer (DMT) for the geotechnical characterization proposing a comparison between results obtained from DMT, CPTu and laboratory tests.

2 GEOTECHNICAL SURVEYS

The surveys were chosen, in typology, number and location, by the geotechnical designer considering the parameters necessary for the analysis.

For the seismic ground response analysis an accurate geological characterization was conducted in addition to geophysical soundings for the estimation of shear waves velocity Vs and cyclic laboratory tests for the measurement of G-γ and D-γ curves (where G is the shear modulus, D is the damping ratio and γ the shear strain).

In order to design deep foundations for the new earthquake-resistant structures, the homogeneous soil layers were investigated in terms of strength and deformability parameters. Thus a 31.0 m deep borehole with soil sampling and stratigraphy, two 40.0 m deep CPTu and a 20.0 m deep DMT were performed for soil characterization from lithological, strength and stiffness point of view (see Fig. 1 and Fig. 2).

Considering geophysical tests, three surface wave tests (MASW), three passive seismic tests (HVSR) and one Down-Hole test were conducted while one direct shear test, two oedometric tests, two consolidated drained (TX CD) triaxial tests, two consolidated undrained (TX CIU) triaxial tests and two resonant column (RC) tests were performed in laboratory.

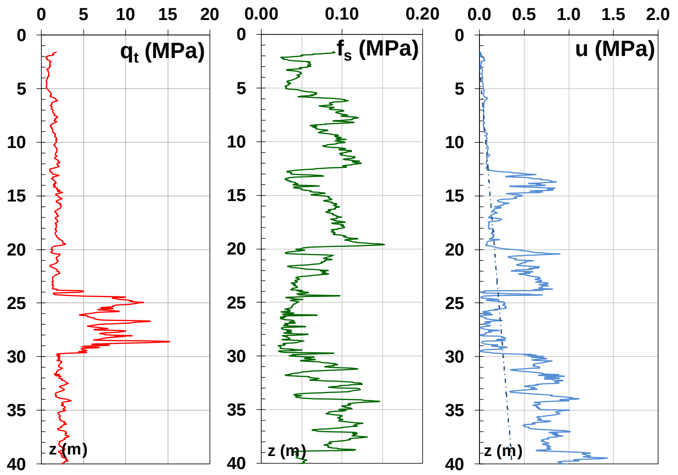

Fig. 1. Data from CPTu 2: from left, cone resistance q_t, sleeve friction f_s and pore pressure u_2.

In Fig. 2 the intermediate parameters from DMT are shown, as defined by Marchetti (1980), Eq. (1):

$$I_D = \frac{p_1 - p_0}{p_0 - u_0}; K_D = \frac{p_0 - u_0}{\sigma'_{v0}}; E_D = 34.7 \cdot (p_1 - p_0) \quad (1)$$

In Eq. (1) p_0 and p_1 are the corrected readings from flat dilatometer, u_0 is the in situ equilibrium pore pressure and σ'_{v0} is the effective overburden stress prior to blade insertion.

3 GEOTECHNICAL CHARACTERIZATION

The geotechnical parameters were defined interpreting the results from soundings; the whole interpretation was curated by geotechnical designers.

A geotechnical model is a set of parameters that, together with a constitutive model, allows to describe mathematically the mechanical response of the soil. Indeed considering the calculations that have to be performed, the constitutive laws can change and, consequently, the values of the parameters.

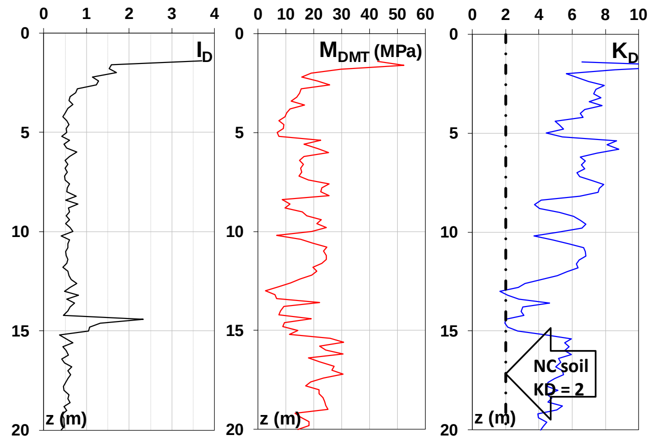

Fig. 2. DMT data: from left, material index I_D, constrained modulus M_{DMT}, horizontal stress index K_D.

In this paragraph the principal geotechnical parameters will be treated while in the following one the geotechnical model employed in the site effect analysis will be shown. The model concerning the design of deep foundations will not be described.

The soil, differently from the building materials, is characterized by a high heterogeneity and variability of its characteristics. For this reason the principal parameters, obtained from geotechnical and geophysical soundings, will be compared to evaluate the reliability of each test to investigate a specific parameter.

In particular, the comparisons will be focused on DMT and CPTu tests.

3.1 *Litho-stratigraphic profile*

It is very important to define an accurate stratigraphic profile in order to identify soil layers that can be considered homogeneous for the mechanical response.

This profile was directly obtained from the borehole and it was integrated applying correlations with CPTu and DMT data, that are a very useful support.

For the CPTu tests, three correlations for lithological interpretation were applied: the ones by Robertson & Cabal (2010), Schneider et al. (2008) and Fellenius (2009). The results are shown in Fig. 3 and Fig. 4. Instead, for the DMT sounding the correlation proposed by Marchetti (1980) was used (Fig. 5).

In Fig. 5 I_c is Soil Behavior Type index (Robertson & Cabal 2010, Eq. (2)), while I_D is the material index (Marchetti 1980, Eq. (1)).

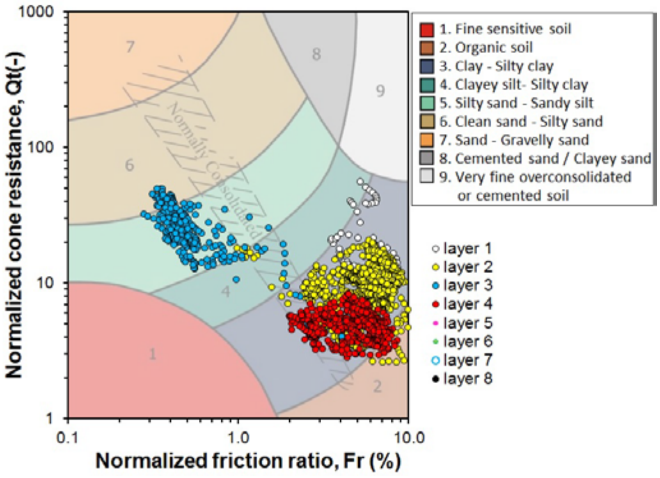

Fig. 3. Robertson (2010) profiling chart.

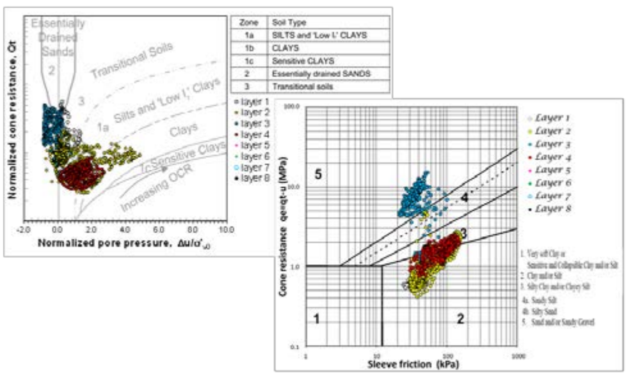

Fig. 4. Schneider et al (2008) and Fellenius (2009) profiling chart.

$$I_C = \left(\left(3.47 - \log Q_t \right)^2 + \left(\log F_r + 1.22 \right)^2 \right)^{0.5} \qquad (2)$$

Where Q_t is the normalized cone resistance Eq. (3) and F_r is the normalized friction ratio Eq. (4):

$$Q_t = \left(q_t - \sigma_{v0} \right) / \sigma'_{v0} \qquad (3)$$

$$F_r = \left(f_s / \left(q_t - \sigma_{v0} \right) \right) \cdot 100\% \qquad (4)$$

As shown in Fig. 5, the feedback from lithological characterization via DMT and CPTu is good.

Please note that a sandy layer is situated between 24.0 m and 30.0 m from the surface. The liquefaction assessment, that is not included in this paper, has shown that this is a non liquefiable layer.

Table 1 summarizes the stratigraphic profile.

3.2 *Undrained shear strength s_u*

The undrained shear strength s_u describes fine grained soil from the resistance point of view in short term condition and it is very important for the evaluation of axial capacity of piles. Hence it is interesting to examine this parameter because of the important presence of fine grained soils on site.

The undrained shear strength s_u was measured in the TX CIU laboratory tests and also obtained via correlations from DMT and CPTu. The comparison of those results, shown in Fig. 6, is useful to evaluate how the different surveys are able to investigate this parameter.

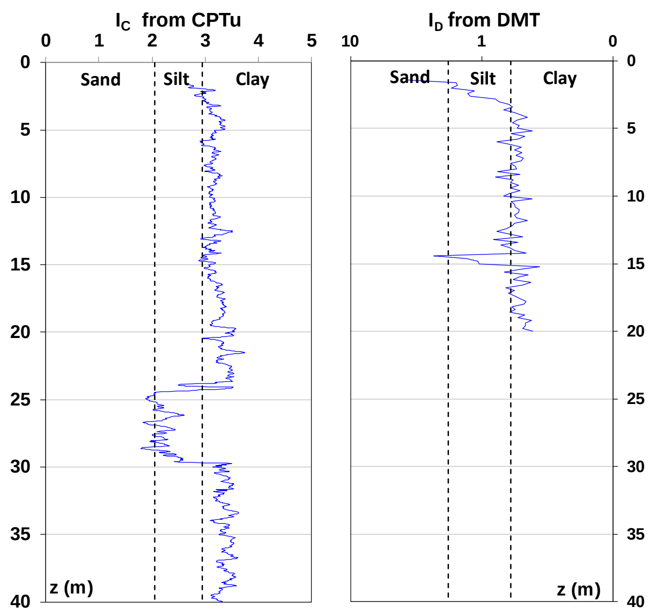

Fig. 5. Comparison between stratigraphy from CPTu and DMT.

Table 1. Stratigraphy resume.

Layer 1	From ground level to 3.0 m from g.l. SUPERFICIAL ALTERED LAYER
Layer 2	From 3.0 m from g.l. to 24.0 m from g.l. CLAY – SILTY CLAY
Layer 3	From 24.0 m from g.l. to 30.0 m from g.l. SILTY SAND
Layer 4	From 30.0 m from g.l. to 40.0 m from g.l. CLAY – SILTY CLAY

As shown in Fig. 6, the agreement between the different test is reasonably good. s_u from DMT, that was derived applying Marchetti (1980) correlation (Eq. (5)), fit quite well the data obtained from TX CIU tests.

$$S_u = 0.22 \cdot \left(0.5 \cdot K_D \right)^{1.25} \cdot \sigma'_{v0} \qquad (5)$$

For CPTu, it was looked for the correlations that best fit the s_u from DMT and TX CIU. Best results were obtained employing the formulation reported in Eq. (6).

$$S_u = \frac{q_c - \sigma_v}{N_k} \qquad (6)$$

The s_u profile, shown in green in Fig. 6, was obtained using N_k = 16. This is a site specific value of the parameter N_k, calibrated according to the DMT and TX CIU data. Instead the s_u profile, shown in blue in Fig. 6, was calculated applying the factor N_k according to Robertson (2012, see Eq. (7)):

$$N_k = 10.5 + 7 \cdot \log(F_r) \tag{7}$$

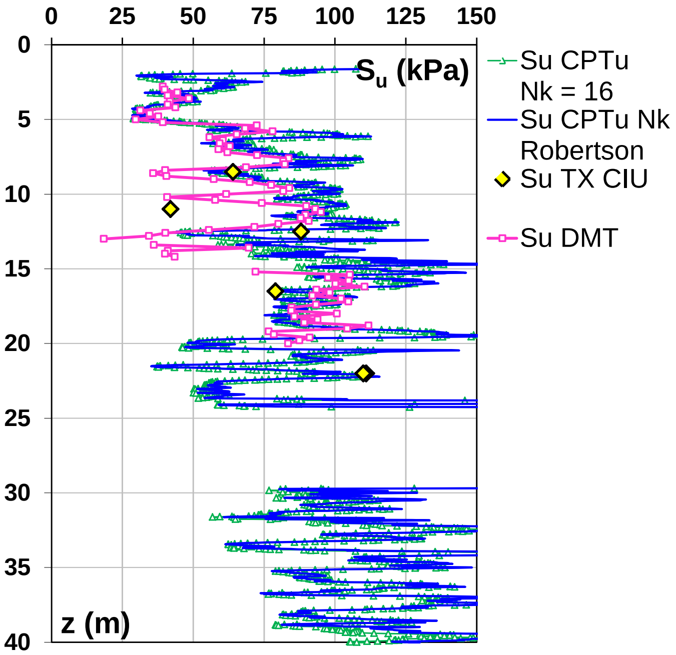

Fig. 6. Undrained shear strength s_u of cohesive soils from laboratory tests, DMT and CPTu.

3.3 *Oedometric modulus E_{oed}*

Among the geotechnical parameters, stiffness parameters are of great importance but are also delicate and complex to be investigated.

It is known that in a borehole the soil sampling causes a disturbance to the specimen that influences the laboratory results in terms of deformation moduli. Indeed it is preferred to rely as much as possible on in situ tests, that are able to investigate the natural condition of the soil.

However, not all in situ tests are characterized by the same reliability. For example, in a CPTu the cone causes the rupture of the soil and so there is less accuracy about stiffness parameters. Instead the DMT was designed to investigate soil stiffness and its reliability was widely demonstrated (Baligh & Scott 1975).

Laboratory data are available from oedometric and TX CD tests that were carried out on two samples taken from the silty-clay layer (from 3.0 m to 24.0 m from surface). In addition there are correlations with CPTu and DMT.

From CPTu and from oedometric lab tests oedometric moduli (E_{oed}) for cohesive soils were derived. Then a comparison with the constrained modulus M_{DMT}, obtained from DMT, was proposed. (Please note that M_{DMT} is equal to E_{oed}).

In the PLAXIS HS Small constitutive model it is very important the E_{50} modulus, i.e. the stiffness that soil shows at 50% of the yielding stress. This modulus can be directly derived from TX CD, and it was also proposed in the comparison in order to compare with the oedometric ones.

Fig. 7 illustrates the correlations of Mitchell & Gardner (1975, Eq. (8)), Kulhawy & Mayne (1990, Eq. (9)) and Tonni & Gottardi (2012, Eq. (10)).

$$E_{oed} = 2.5 \cdot q_c \tag{8}$$

$$E_{oed} = 8.25 \cdot (q_c - \sigma_{v0}) \tag{9}$$

$$E_{oed} = 1.35 \cdot Ic \cdot (q_c - \sigma_{v0}) \tag{10}$$

Considering the M_{DMT} profile, it can be observed (Fig. 7) that CPTu correlations highly underestimate the moduli. The results from oedometric tests are aligned with CPTu correlations and this is compatible with what mentioned about disturbance caused to the samples.

In order to establish a better correlation for CPTu data, a site specific coefficient $\alpha = 12$ was defined for Eq. (11), using DMT data.

$$E_{oed} = \alpha \cdot q_c \tag{11}$$

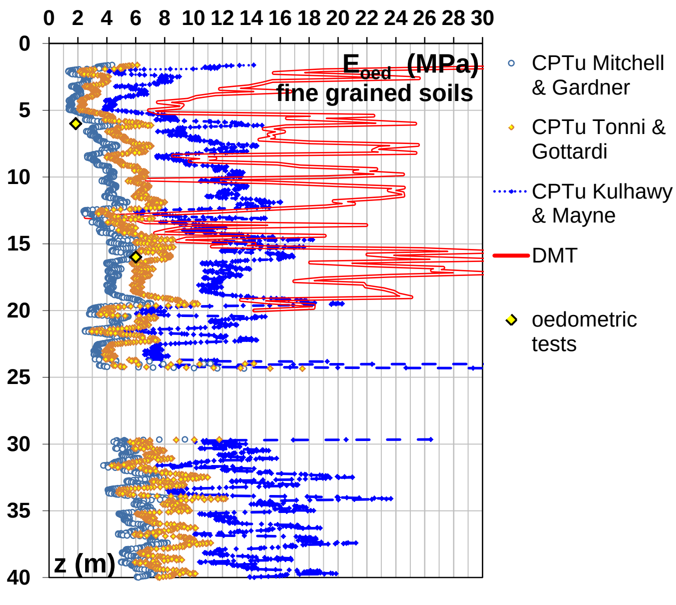

Fig. 7. Constrained modulus E_{oed} of cohesive soils from DMT, CPTu and lab tests.

In Fig. 8 the lab data are referred to E_{50} moduli obtained from TX CD. At least in the present case M_{DMT} measured with flat dilatometer is similar to E_{50} values. However, considering that TX CD tests are also influenced by disturbance caused to the samples, this is not sufficient to state that M_{DMT}

correspond to E_{50} in HS Small model instead of E_{oed}.

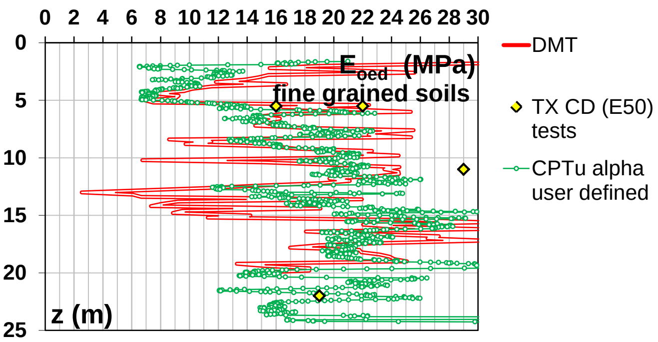

Fig. 8. E_{oed} from CPTu: site specific correlation.

3.4 *Shear waves velocity Vs*

MASW, HVSR and a Down Hole have been performed to evaluate the shear waves velocity vertical profile.

Those investigations were useful not only for seismic ground response analysis but also for the design of deep foundations because, via the small strain stiffness G_0, they were used for example to estimate the load-settlement curves and in numerical BEM codes.

The Vs profile was very important for the ground response analysis, as for one-dimentional approaches Vs is a direct input, while in Plaxis bi-dimensional finite element analysis Vs are used to obtain G_0 values, one of the input parameters in HS Small model.

HS Small differs from Hardening Soil model because it is able to account for the soil stiffness at small strain, thanks to two additional input parameters, the tangent modulus G_0 and the value of shear strain $\gamma_{0,7}$ at which G is decreased to 70% of its initial value, used to define the stiffness and damping vs. shear strain curves (Benz 2007).

It is possible to obtain a Vs profile also via correlation with DMT and CPTu. The correlations that are best able to predict Vs from CPTu, in the present case study, are those of Rix & Stokoe (1991, Eq. (12)) and Hegazy & Mayne (1995, Eq. (13)). Colombi et al. (2007) have defined A, α and β coefficients for Ferrara area.

$$Vs = A \cdot \left(\frac{q_c}{p_a} \right)^{\alpha} \tag{12}$$

$$Vs = A \cdot \left(\frac{q_c}{p_a} \right)^{\alpha} \cdot \left(\frac{f_s}{p_a} \right)^{\beta} \tag{13}$$

For DMT Marchetti formulation (Marchetti et al. 2008) was used to estimate G_0, from which Vs can be easily derived (see Eq. (14)).

The comparisons between Vs measured and estimated from CPTu and DMT are shown in Fig. 9.

The CPTu-Vs correlations provide good results in the first 15.0 m but below that depth they are not able to predict the stiffness change shown by geophysical investigations.

$$\begin{aligned} I_D < 0.6 &\Rightarrow G_0/M_{DMT} = 26.177 \cdot K_D^{-1.066} \\ 0.6 < I_D < 1.8 &\Rightarrow G_0/M_{DMT} = 15.686 \cdot K_D^{-0.921} \\ I_D > 1.8 &\Rightarrow G_0/M_{DMT} = 4.5613 \cdot K_D^{-0.7967} \end{aligned} \tag{14}$$

The DMT-Vs correlation is able to predict correctly the geophysical results, and in particular the stiffness increase at the depth of 15.0 m from ground level. The good agreement between geophysical tests, CPTu and DMT gives to Vs profile, that was very important for the aim of the work, a great reliability.

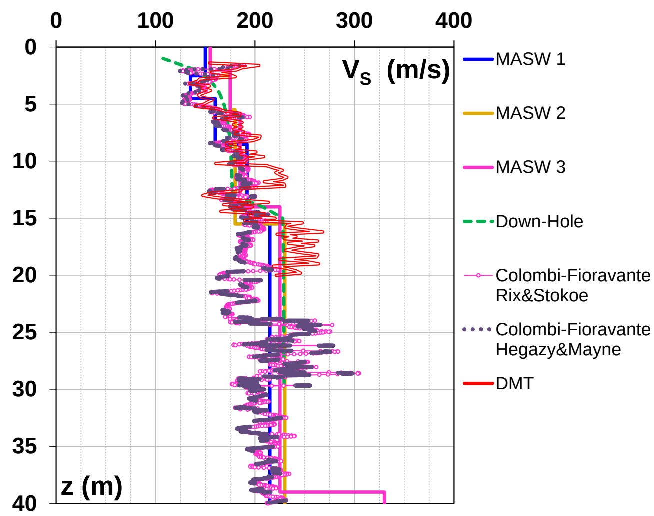

Fig. 9. Vs profiles from geophysical tests, CPTu and DMT.

3.5 *G-γ decay curves*

The stiffness decay curves G-γ play a primary role in seismic ground response analyses. In 1D EERA analyses they are a direct input while in 2D analyses, that were performed using Plaxis Dynamic, the HS Small constitutive model includes laws that, starting from input parameters, allows to generate the corresponding G-γ curves (in terms of stiffness) and D-γ (in terms of damping ratio). To directly investigate the decay curves two cyclic laboratory tests in resonant column were performed. Fig. 10 shows one of the measured G-γ curves.

Recently researchers worked on the relationship between the G-γ decay curves and the flat dilatometer (Amoroso et al. 2014, Amoroso et al. 2012, Marchetti et al. 2008). The basic concept is that, performing a SDMT, or a standard DMT and knowing the Vs profile via other geophysical soundings, two points of the curve are known: the initial one G_0, with the initial tangent value of the stiffness linked to Vs, and a second point G_{DMT} in which the initial value of the stiffness is decreased to the one obtained by flat dilatometer. Hence a shape of the curve that is coherent with those two points could be defined to obtain the complete G-γ curve. Using this approach, the strain value corresponding to G_{DMT}, i.e. γ_{DMT}, is defined through a range.

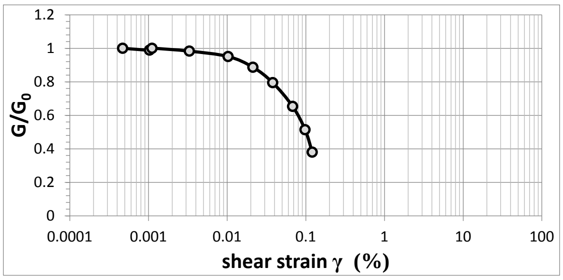

Fig. 10. G-γ curve measured in one of the RC tests. Clayey sample taken at a depth of 11.0 m.

In particular, two shapes of the curves were considered. The first was taken from Maugeri & Carrubba (1988, Eq. (15)) while the second from Amoroso et al. (2014, Eq. (16)).

$$\frac{G}{G_0} = \frac{1}{1+\alpha\cdot\gamma(\%)^{\beta}} \tag{15}$$

$$\frac{G}{G_0} = \frac{1}{1+\left(G_0/G_{DMT} - 1\right)\cdot \gamma/\gamma_{DMT}} \tag{16}$$

Comparing data from DMT at the same depths at which the RC tests were performed and decay curves from RC tests it was possible to calibrate the parameters of Eq. (15), i.e. α and β, and the parameter γ_{DMT} of Eq. (16) to obtain the curves that best fit those from RC. Then the obtained parameters can be applied to evaluate the curves also at other depth where lab tests have not been performed but in which DMT data are known. γ_{DMT} was found in a range between 0.75% and 1.7%, in agreement with bibliographic data, reported in the background of Fig. 11.

The great advantage of this approach is that, while lab testing are rarely performed and are however punctual, G-γ curves from DMT provide continuous information with depth.

The decay of stiffness was also studied employing, once again, the flat dilatometer and data from Down-Hole to evaluate, in function of the different lithology and with depth, how much the stiffness decreases with respect to its initial tangent value. In Fig. 12 the yellow points represent the G_0/M_{DMT} ratio at various depth: it can be seen how they are aligned with the bibliographic data (white points from Marchetti et al. 2008).

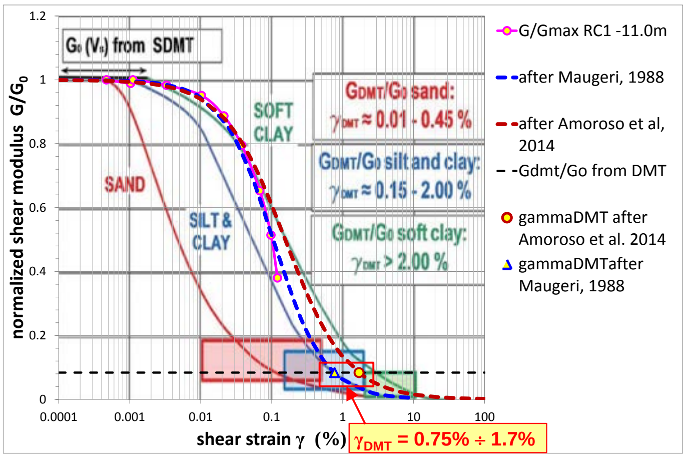

Fig. 11. Comparison of G-γ curves from RC and DMT.

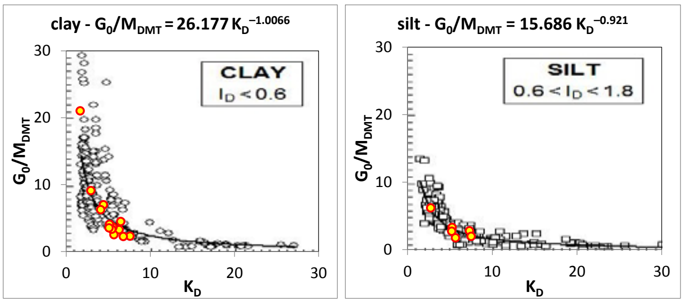

Fig. 12. G_0/M_{DMT} ratio for clayey and silty soils.

4 SEISMIC GROUND RESPONSE ANALYSIS

The aim of the seismic ground response analysis is to evaluate how a seismic signal changes while crosses soil layers.

The analyses were performed using EERA (Equivalent-linear Earthquake site Response Analyses of layered soil deposits, Bardet et al. 2000), a one-dimensional 1D equivalent-linear approach, and Plaxis Dynamic (Plaxis 2014), a 2D non linear FEM software.

All the calculations are based on 10 accelerograms that are compatible with the parameters defining the seismic hazard of the site with respect to Italian building code (NTC 2008).

The 1D calculations need the definition of the litho-stratigraphic profile, the Vs vertical profile and the G-γ and D-γ decay curves.

The 2D Plaxis analyses were performed employing the HS Small model. Table 2 summarizes the principal parameters of the geotechnical model. Only those concerning the first 40.0 m of soil are reported because are those that were directly investigated by the surveys. In the analysis the data were extrapolated with depth, referring also to geological characterization and to other deep soundings performed in the Po valley to reach the bedrock depth considered in the calculations.

Table 2. HS Small model parameters.

Layer: C1A			
From: 0 m		To:	-16.0 m
Parameter		U.M.	Value
Unit weight	γ	kN/m^3	18
Cohesion	c'	kPa	7
Friction angle	Φ'	°	16
Secant stiffness	E_{50}^{ref}	MPa	24
Oedometric modulus	E_{oed}^{ref}	MPa	16
Unloading-reloading modulus	E_{ur}^{ref}	MPa	80
Power function	m	-	0.4
Small strain shear modulus	G_0^{ref}	MPa	65
Shear strain	$\gamma_{0.7}$	-	4E-4
Layer: C1B			
From: -16.0 m		To:	-24.0 m
Parameter		U.M.	Value
Unit weight	γ	kN/m^3	18
Cohesion	c'	kPa	7
Friction angle	Φ'	°	16
Secant stiffness	E_{50}^{ref}	MPa	30
Oedometric modulus	E_{oed}^{ref}	MPa	20
Unloading-reloading modulus	E_{ur}^{ref}	MPa	100
Power function	m	-	0.5
Small strain shear modulus	G_0^{ref}	MPa	90
Shear strain	$\gamma_{0.7}$	-	4E-4
Layer: S1			
From: -24.0 m		To:	-30.0 m
Parameter		U.M.	Value
Unit weight	γ	kN/m^3	19
Cohesion	c'	kPa	1
Friction angle	Φ'	°	29
Secant stiffness	E_{50}^{ref}	MPa	20
Oedometric modulus	E_{oed}^{ref}	MPa	20
Unloading-reloading modulus	E_{ur}^{ref}	MPa	60
Power function	m	-	0.5
Small strain shear modulus	G_0^{ref}	MPa	80
Shear strain	$\gamma_{0.7}$	-	2E-4
Layer: C2			
From: -30.0 m		To:	-40.0 m
Parameter		U.M.	Value
Unit weight	γ	kN/m^3	18
Cohesion	c'	kPa	7
Friction angle	Φ'	°	21
Secant stiffness	E_{50}^{ref}	MPa	36
Oedometric modulus	E_{oed}^{ref}	MPa	24
Unloading-reloading modulus	E_{ur}^{ref}	MPa	120
Power function	m	-	0.5
Small strain shear modulus	G_0^{ref}	MPa	70
Shear strain	$\gamma_{0.7}$	-	4E-4

From 1D and 2D analysis response spectra and accelerograms were calculated to be used in the design of reinforcement interventions and seismic improvements of the structures required after the earthquakes of 2012.

Among the results, response spectra have a particular interest because not only they are more frequently applied in the design but they also allow to evaluate changes induced in the input signal by the soil in function of the vibration period. Results from 10 accelerograms have been statistically treated to obtain an average spectrum and a confidence interval, as can be observed in Fig. 13.

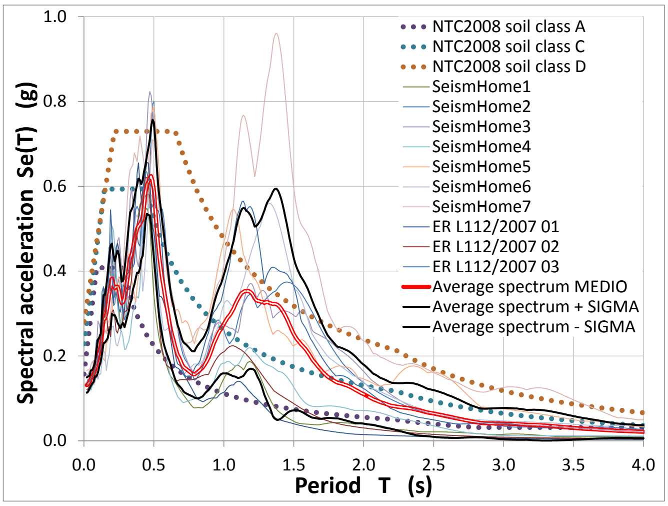

Fig. 13. Response spectra from Plaxis 2D FEM analysis.

With the simplified method proposed by technical code NTC 2008, the effects induced by stratigraphic amplification can be taken into account by defining the soil type in terms of $V_{s,30}$, that represents a weighted value of Vs in the first 30.0 m of depth.

Geophysical tests show values of $V_{s,30}$ close to the boundary between class C and D, since $V_{s,30} \cong 180$ m/s. As illustrated in Fig. 14 there are significant differences between the two corresponding spectra. The choice depends on the designer's choice.

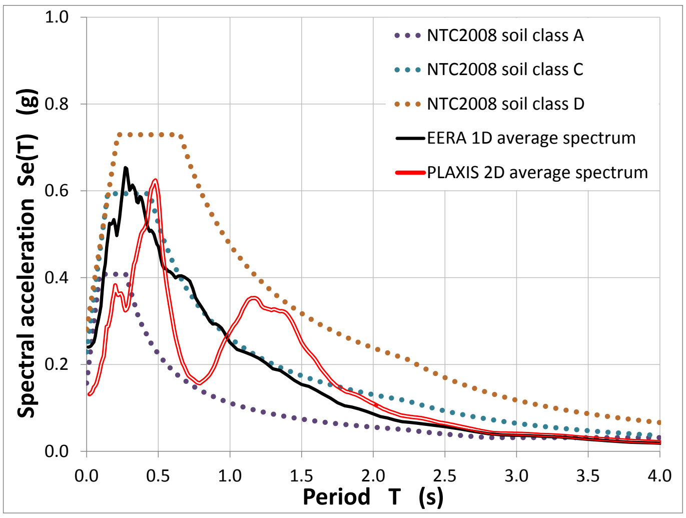

Fig. 14. Average response spectra from 1D EERA analyses and Plaxis 2D Dynamic FEM analyses.

Ground response analyses (see Fig. 14) show how the spectrum for C soil class is not able to predict the maximum spectral acceleration, that is underestimated of about 10%, neither the relevant spectral amplifications that the analysis highlights for periods ranging between 1.0 s and 1.5 s. The spectrum for D soil class is instead able to envelope the average response spectra obtained from the analyses.

5 CONCLUSIONS

The present paper concerns the geotechnical aspects regarding the reconstruction interventions of the Magneti Marelli factory, located in the Municipality of Crevalcore (Bologna), designed by Teleios Srl engineering company.

Those aspects are generally of basic importance but they become essential after the 2012 Emilia-Romagna earthquakes. The present paper provided a description of the geotechnical campaign and of the use of the geotechnical parameters in the design. The superstructures need deep foundations, so part of the soundings were finalized to the construction of a geotechnical model for pile capacity and settlement evaluation. Another important aspect for the design of interventions on damaged buildings is the ground response analysis to estimate spectra able to take into account the influence of the soil on the seismic input. The geotechnical and geophysical campaign allowed to construct a reliable geotechnical model.

In this article the most relevant part is dedicated to the comparison of results obtained from various type of surveys, to evaluate how each one is able to investigate a certain parameter (s_u, E_{oed}, Vs, etc.) paying particular attention to DMT and CPTu correlations. DMT results show a stiffer response of soils with respect to what expected from laboratory tests and CPTu correlations. So it was possible to define a site-specific correlation based on CPTu for the evaluation of constrained modulus E_{oed}.

Finally, a HS Small model was defined to perform the seismic ground response 2D analysis. In addition 1D calculations with EERA code were performed to allow a comparison of the results.

The results obtained from site effect analyses were very important because from one side a decrease in the peak pseudo-acceleration for low periods was highlighted but, on the other side, a relevant increase in pseudo-acceleration for higher periods (between 1.0 s and 1.5 s) was found. This is not provided in the spectra defined in the NTC 2008 technical code but is very important in the design of steel structures, both new structures and reinforcing frames, that can vibrate with periods in this range.

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