A local seismic response studies based on the pressuremeter results

Une étude de réponse sismique locale basée sur les résultats du pressiomètre

Cavallaro A.[1], *Grasso S.*[2], and *Sammito M. S.*[3]

[1]*CNR-ISPC, Palazzo Ingrassia, Via Biblioteca n. 4, 95124 Catania, Italy*
[2]*University of Catania, DICAr, Viale A. Doria 6, 95125 Catania, Italy*
[3]*University "Kore" of Enna, Piazza dell'Università, 94100, Enna, Italy*
[1]*Corresponding author: antoniomariafrancesco.cavallaro@cnr.it*

ABSTRACT

The city of Augusta is located in the province of Syracuse, on the eastern coast of Sicily, which is one of the most seismically active areas of Italy. In order to study the geotechnical dynamic characteristics of the foundation soil of the Augusta Hangar, in-situ investigations and laboratory tests were carried out. Recorded accelerograms were employed as seismic input in linear-equivalent codes operating in the frequency domain. Processing of these data also allowed the ground response analysis at the surface, in terms of time history and response spectra. This paper tries to summarize this information in a comprehensive way in order to provide a representative geotechnical model of the site where an important historical building is located.

RESUME

La ville d'Augusta est située dans la province de Syracuse, sur la côte est de la Sicile, l'une des régions les plus sismiques d'Italie. Afin d'étudier les caractéristiques géotechniques dynamiques du sol de fondation du hangar d'Augusta, des investigations in situ et des essais en laboratoire ont été réalisés. Les accélérogrammes enregistrés ont été utilisés comme données d'entrée sismique dans des codes linéaires équivalents fonctionnant dans le domaine fréquentiel. Le traitement de ces données a également permis d'analyser la réponse du sol en surface, en termes d'historique temporel et de spectres de réponse. Cet article tente de synthétiser ces informations de manière exhaustive afin de fournir un modèle géotechnique représentatif du site où se trouve un important bâtiment historique.

Keywords: dynamic; shear modulus; damping; response analysis.

1. Introduction

The Augusta old town is an island connected to the mainland by two bridges. As Augusta is located in a seismic area that was struck in the past by several intense earthquakes, seismic loadings were taken into account in the design of the reinforcement and retrofitting. Since 1169, the city of Augusta was struck by three disastrous earthquakes with an MKS intensity ranging from IX to XI. The seismic event that occurred on December 13, 1990, with the epicenter close to Augusta and maximum intensity of VII-VIII MCS caused also heavy structural damage to some historical buildings.

To evaluate the geotechnical characteristics, the following in-situ and laboratory tests were performed in the foundation soil located in the area of Augusta Hangar (Figure 1): boreholes; borehole permeability tests; Standard Penetration test (SPT); Field Vane tests (FVT); Ménard Pressuremeter tests (MPMT); Down-Hole tests (DHT); oedometer tests; direct shear tests (DST); Triaxial CU tests (TxCU); Triaxial UU tests (TxUU); cyclic loading torsional shear tests (CLTST); resonant column tests (RCT).

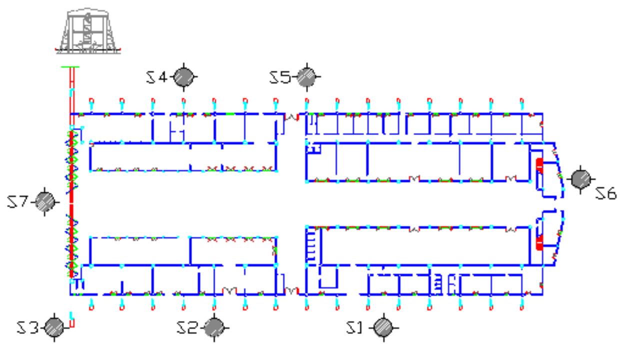

Figure 1. Layout of Augusta Hangar site and geotechnical boreholes S1, S2, S3, S4, S5, S6 and S7.

Recorded accelerograms were employed for the ground response analysis and results were obtained at the surface in terms of time history and response spectra using a 1-D non-linear model. The codes implement a one-dimensional simplified, hysteretic model for the non-linear soil response. The analysis provides the time-history response in terms of displacements, velocity and acceleration at the surface. Using this time history, response spectra for the investigated site were successfully deduced.

Similar geotechnical studies were successfully performed for significant historical test sites (Cavallaro et al., 1999, 2003, 2013) with the aim of preserving the cultural and artistic heritage from possible future earthquakes.

2. The pressuremeter test

In order to perform the pressuremeter test, a cylindrical expandable probe is inserted into the ground at the appropriate depth, gradually expanded, and the applied pressure and the corresponding deformation are recorded (Figure 2).

The probe is generally installed vertically and connected by pipes or cables to the surface where a control and measurement unit operates. The membrane is made to expand against the surrounding ground by means of the pressure of a fluid (water, gas or oil) introduced by the control unit through special tubes (Baguelin et al. 1972).

The deformation of the ground is detected by measuring the volume of fluid injected into the probe. In pressuremeters that use pressurized gas, the radial deformation of the ground is directly detected by a caliper or by instrumented arms.

The pressuremeter was created by Menard in 1954 as a device installed in a pre-formed hole (MPMT). The test was widely used in France, thanks also to the research carried out by the LPC (Laboratoire Central des Ponts et Chaussées), a state organization in the field of public works, which was among the first users. LPC published an official standard for the execution of the test in 1971 and issued a recommendation for the use of this test in the design of foundations in 1985 (LPC 1971, 1985).

The research activity of the Menard group, which distributed the pressuremeter in various countries according to the formula of the "exclusive concession" and that of the LPC, contributed to extending the knowledge of the test, to improving the evaluation of its potential and its limits and to defining an empirical method for the use of the results in the design.

Consequently, the instrument and the installation methodology were gradually improved and strengthened (Tornaghi., 1983). The most interesting feature of the pressure test is that it provides direct information on both the resistance and the deformability of the soil, under defined and at least partially controlled boundary conditions.

In France, during the period of great development of the MPM pressuremeter test (Menard), the approach of directly incorporating the results into the geotechnical calculations using empirical or semi-empirical procedures prevailed. The direct use of the results obtained from tests carried out according to the French standard (Baguelin et al. 1986) can be considered as a valid and tested method in the geotechnical design. Moreover, it was recently refined especially regarding pile foundations subjected to both axial and horizontal loads.

Through the use of in situ dynamic tests and the ability to obtain the deformability parameters of the soils through the MPM tests, it is possible to address the study of the local seismic response in a coordinated manner.

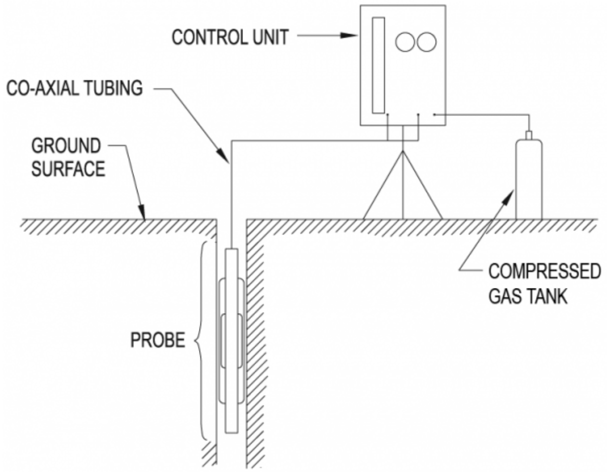

Figure 2. Pressuremeter Menard test.

3. Basic geotechnical soil properties

The site investigation, performed within the area of the hangar, and reached a maximum depth of 50 m.

The Augusta Hangar area mainly consists of grey clay in the lower layer, an alternation of clayey silt and silty clay layers, and silty sand with gravel lenses in the upper layers.

Using information made available from in situ boreholes, soil profiles of the ground beneath the Augusta Hangar werw developed.

Based on laboratory test results, the Augusta Hangar deposit mainly consist of over-consolidated clayey silty soil. The preconsolidation pressure σ'_p and the overconsolidation ratio $OCR = \sigma'_p/\sigma'_{vo}$ were evaluated from the compression curves of Incremental Loading (IL) oedometer tests. OCR values range from 2 to 7.

The value of the natural moisture content, w_n, prevalently ranges from 22 to 35%. Characteristic values for the Atterberg limits are: w_L = 49-67% and w_P = 21-33%, with a plasticity index of PI = 21-40%. The specific gravity, G_s, ranges from 2.57 to 2.79. The soil deposits can be classified as inorganic silt and clay of medium to high plasticity [CH-MH].

The water level ranges between 2.25 and 2.7 m from the ground surface for boreholes 1, 2, 4, 5, 6. For boreholes 3 and 7, the water level is located at 4.85 m from the ground surface.

As regards strength parameters of the deposits mainly encountered in this area, c' ranges between 19 and 28 kPa, while φ' ranges between 19° and 23°. The permeability values obtained by in situ Lefranc tests range between 10^{-8} and 10^{-9} cm/s.

4. In situ soil tests

Small strain shear modulus G_o (γ less than 0.001 %) can be evaluated by different empirical correlations and in-situ tests, such as the MPM tests, the DH tests and SPT tests.

A Menard Pressuremeter (MPM) device is used to in situ assess the stiffness of inorganic silt and clay of medium to high plasticity soil. The Menard Pressuremeter (MPM) is generally inserted in a pre-bored hole. This device was developed in the early sixties in France and is widely used in the case of direct interpretation methods (Baguelin et al. 1978).

Four pressuremeter tests were performed in the Hangar area of Augusta. All the results acquired were used for scientific research. The data relating to the borehole S6 are reported in the following figures because they are considered more representative.

A pressuremeter test consists of the expansion of a cylindrical cavity which has a finite length L and diameter D. During the test, the applied cavity pressure (p) and the corresponding cavity volume V are measured.

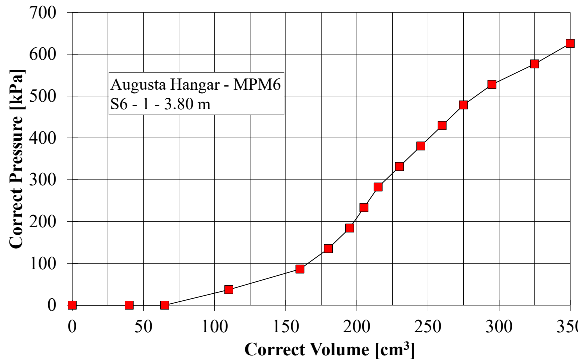

Figure 3. Expansion curve from Menard pressuremeter tests.

The test yields an expansion curve of the type shown in Figure 3 which allows, at least in principle, the direct determination of the small strain shear modulus G_o from the initial slope of the expansion curve.

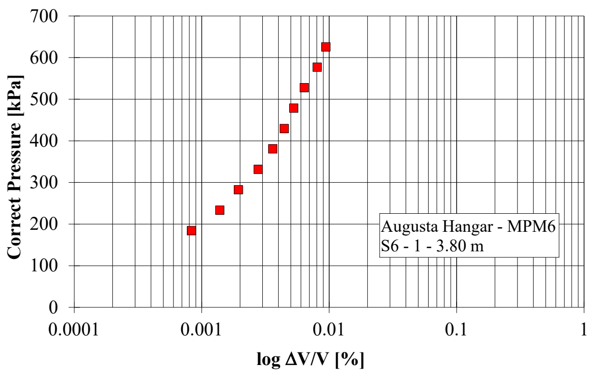

Figure 4. P-log(ΔV/V) curve from Menard pressuremeter tests.

This modulus can be directly determined from the expansion curve because it is possible to neglect soil non-linearity as a first approximation. However, the initial shape of the expansion curve can be sensitive to soil disturbance and to the compliance of the measuring system. Consequently, the direct assessment of G_o from the expansion curve can be affected by some mistakes and the obtained shear modulus seems quite suitable and reliable if appropriate assumptions are made (Byre et al., 1990; Fahey and Carter, 1993; Ghionna et al., 1994).

For these reasons, it is appropriate to evaluate G_o along the section of the expansion curve characterised by pseudo-elastic behaviour, thus excluding the initial section influenced by disturbance phenomena, by the relationship:

$$G_o = v_o \cdot (\Delta p / \Delta V) \qquad (1)$$

where: v_o is the initial volume. It corresponds to the volume necessary to bring the cylindrical cavity into contact with the wall of the hole, also recovering any volume due to swelling of the ground (Mair and Wood, 1987).

The values of G_o evaluated by the MPMT are reported in Figure 6.

Using the Menard Pressuremeter results, the c_u values were also obtained considering the slope of correct pressure-log (ΔV/V) curve (Figure 4). Also for MPMT, the c_u values obtained are smaller than those obtained by laboratory tests. Seismic tests enable one to determine the velocity of body waves [compressional (P) and/or shear (S)] which induce very small strain levels into soil, i.e., $\varepsilon ij < 0.001\%$ (Woods 1978). It is possible, on the basis of the measured wave velocities, to obtain the small strain deformation characteristics according to the well-known relationships of the linear elasticity theory.

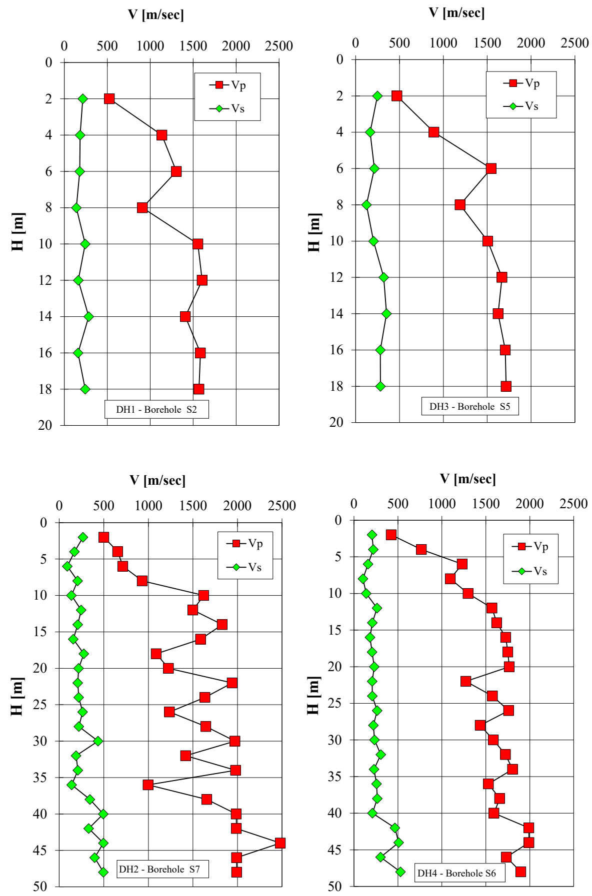

Figure 5. V_s and V_p from Down Hole tests (DH1, DH2, DH3, DH4).

Among the various borehole seismic methods, the down-hole (DH) test is performed in this study. Current practice and recent innovations of borehole methods for seismic exploration are covered by many comprehensive works (Jamiolkowski et al. 1995). In Figure 5, the shear (V_s) and compression (V_p) wave velocities are shown against depth.

V_s may be converted into the initial shear modulus G_o by the theory of elasticity by the well known relationships:

$$G_o = \rho \cdot V_s^2 \qquad (2)$$

where: ρ = mass density.

A comparison between G_o values obtained from in situ test performed on the area under consideration is shown in Figure 6. The down-hole tests performed in Augusta Hangar show G_o values increasing with depth. Very high values of G_o are obtained for depths greater than 40 m.

According to these data, it is possible to assume G_o values oscillate around 100 MPa for depths smaller than 40 m.

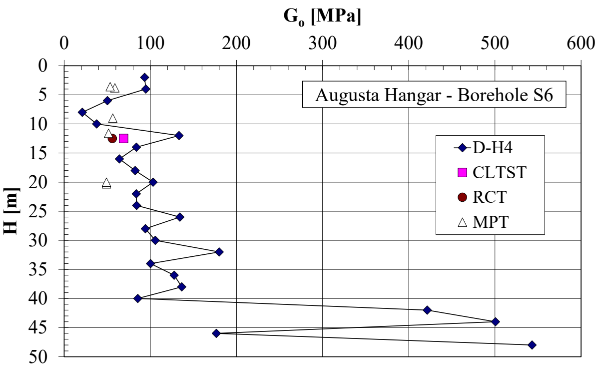

Figure 6. G_o from laboratory and in situ tests.

5. Dynamic laboratory tests

Shear modulus G and damping ratio D of Augusta Hangar deposits were obtained in the laboratory from resonant column tests (RCT) and cyclic loading torsional shear tests (CLTST).

G_o is the maximum value or also "plateau" value as observed in the G-log(γ) plot; G is the secant modulus. Generally G is constant until a certain strain limit is exceeded. This limit is called elastic threshold shear strain (γ^e_t) and it is believed that soils behave elastically at strains smaller than γ^e_t.

The elastic stiffness at $\gamma<\gamma^e_t$ is thus the already defined G_o.

For CLTSTs, the damping ratio (D) was calculated as the ratio between the area enclosed by the unloading-reloading loop and represents the total energy loss during the cycle and W is the elastic stored energy. For RCTs, the damping ratio was determined using the steady-state method during the resonance condition of the sample.

The undisturbed specimens were isotropically reconsolidated to the best estimate of the in situ mean effective stress. The same specimen was first subject to RCT, then to CLTST after a rest period of 24 h with opened drainage. CLTST were performed under stress control condition by applying a torque, with triangular time history, at a frequency of 0.1 Hz. The size of solid cylindrical specimens is a radius of 25 mm and a height of 100 mm.

The G_o values obtained from RCT and CLTST, indicate moderate influence of strain rate even at very small strain where the soil behaviour is supposed to be elastic. Values of shear modulus G [MPa], damping ratio D [%], and pore pressure build-up ΔU [KPa] versus γ [%] from RCT and CLTST tests are reported in Figure 7. The shear modulus obtained during CLTST shows the effect of the soil hardening because of RCT and the dissipation of pore pressure build-up.

Higher values of pore pressure were obtained during RCT rather than during CLTST. For a strain level of about 0.005%, it is possible to observe a rapid pore pressure buildup during RCT. Finally, higher values of the initial shear modulus (Figures 6 and 7), were obtained from down-hole tests.

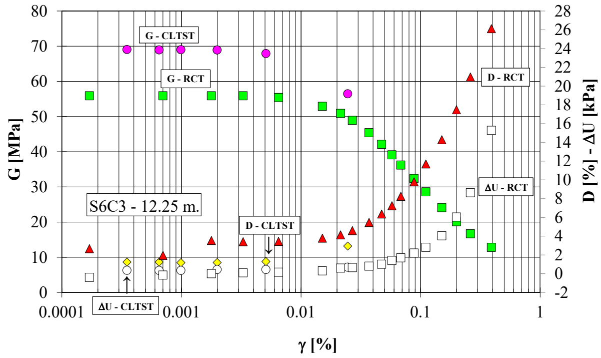

Figure 7. G, D and pore pressure build-up vs. γ from RCT and CLTST.

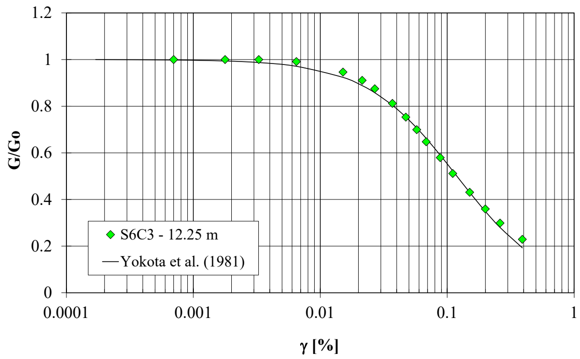

Figure 8. G/G_o-γ curves from RCT.

Generally, the small strain stiffness, determined in the laboratory on high-quality undisturbed samples using appropriate apparatuses and procedures, is very close to that obtained in situ from seismic tests (Jamiolkowski et al. 1995). Probably, in the case of Augusta's Hangar, disturbance phenomena occurred during sampling operations and differences in stress conditions determined lower values of the initial shear modulo in the laboratory (Ng and Wang 2001). The damping ratio values obtained from RCT by steadystate method are quasi-constant until strain level of about 0.02%; higher values of D have been obtained from a strain level higher than 0.02%. It is possible to see that the damping ratio from CLTST, at very small strains, is equal to about 1%.

The experimental results were used to determine the empirical parameters of the equation proposed by Yokota et al. (1981) to describe the shear modulus decay with shear strain level (Figure 8):

$$\frac{G(\gamma)}{G_o} = \frac{1}{1+\alpha\gamma(\%)^{\beta}} \tag{3}$$

in which $G(\gamma)$ = strain dependent shear modulus; γ = shear strain; α, β = soil constants.

Expression (3) allows the complete shear modulus degradation to be considered with strain level. The values of α = 13 and β = 1.206 were obtained for Augusta Hangar area.

Greater values of D are obtained from RCT for the completely investigated strain interval.

As suggested by Yokota et al. (1981), the inverse variation of damping ratio with respect to the normalized shear modulus has an exponential form as that reported in Figure 9 for the Augusta Hangar area:

$$D(\gamma)(\%) = \eta \cdot \exp\left[-\lambda \cdot \frac{G(\gamma)}{G_o}\right] \tag{4}$$

in which $D(\gamma)$ = strain dependent damping ratio; γ = shear strain; η, λ = soil constants. The values of η = 45 and λ = 2.65 were obtained for Augusta Hangar area.

The Eq. (4) assume maximum value D_{max} = 45% for $G(\gamma)/G_o$ = 0 and minimum value D_{min} = 3.18% for $G(\gamma)/G_o$ = 1.

Therefore, Eq. (4) can be re-written in the following normalized form:

$$\frac{D(\gamma)}{D(\gamma)_{max}} = \exp\left[-\lambda \cdot \frac{G(\gamma)}{G_o}\right] \tag{5}$$

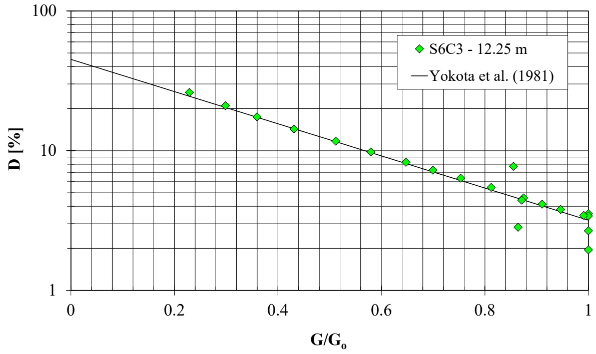

Figure 9. D/G/G_o curves from RCT.

6. Ground response analysis

When an area appears rather flat and characterized by lithological units trending sub-horizontal, it is possible to perform a one-dimensional local seismic ground response analysis to assess the ground amplification due to local stratigraphic conditions. So, in the case of Augusta Hangar, the ground response analyses were performed using 1-D linear equivalent codes (Cavallaro et al. 2018, 2022, 2024) assuming a geometric and geological model of substrate as 1-D physical model.

These analyses were carried out here with EERA (Bardet et al. 2000), STRATA (Kottke and Rathje 2008), and DEEPSOIL (Groholski et al. 2016) linear equivalent codes operating in the frequency domain.

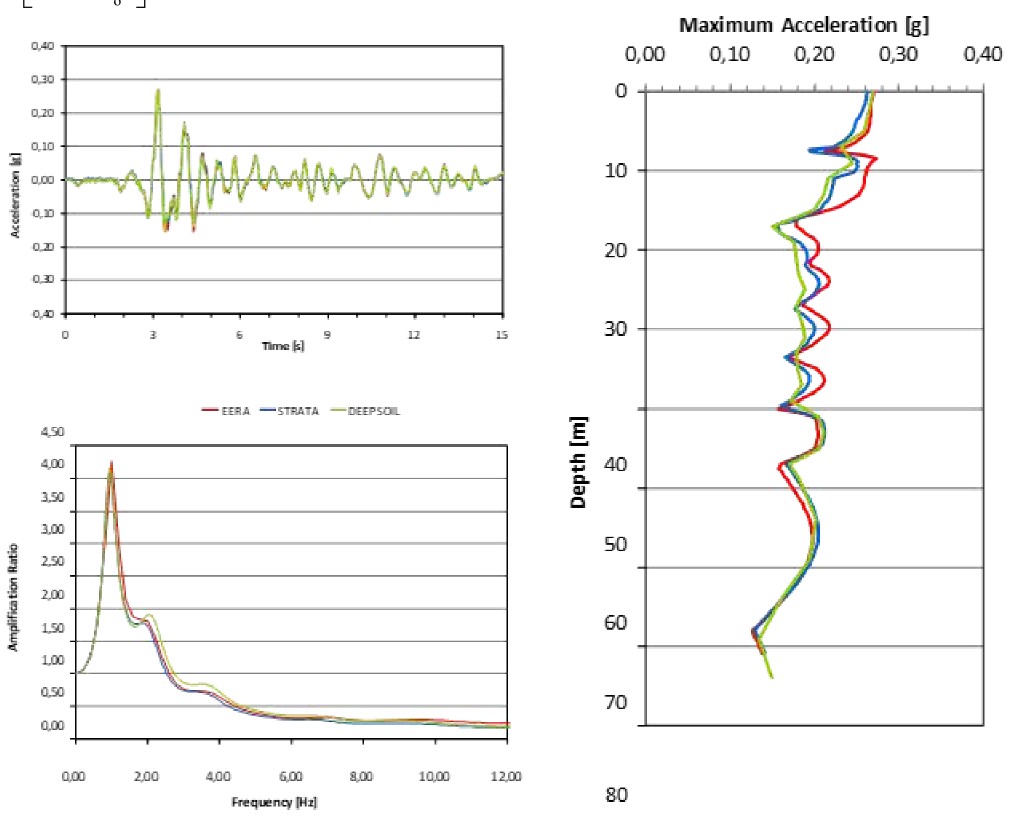

Figure 10. Comparison of 1-D soil response analysis obtained by the three codes EERA (red line), STRATA (blue line), and DEEPSOIL (green line) using the V_s profile of D-H4 (borehole 6) and using as input the 1990 scaled recorded accelerogram.

As regards the input motion at the ultimate limit state (ULS), the site response analyses at the Augusta Hangar site were made using as excitation at the base of the models the recorded accelerograms by the recordings of the December 13, 1990 earthquake, at the Sortino (a_{max} = 1.00 m/s^2) recording station.

Recorded accelerograms of December 13, 1990 earthquake have been scaled to the PGA at the bedrock of 0.231 g corresponding to a return period of 475 years in the current Italian seismic code "*seismic hazard and seismic classification criteria for the national territory*" obtained by a probabilistic approach at ULS in the interactive seismic hazard maps.

The dynamic response model requires the knowledge of the depth of bedrock, which in the case of the Augusta site was not clearly known. Therefore, the criteria of choice adopted to evaluate the depth of bedrock consists in the extrapolation of the shear waves profile obtained from down-hole tests. The conventionally adopted depth of the seismic bedrock is 70 m to which corresponds a V_s value of about 800 m/s (CEN EC8 1993).

The ground response analysis at ULS, performed with EERA, STRATA, and DEEPSOIL linear equivalent codes, provides the time-history response of displacements, velocity and acceleration at the surface. Figure 10 shows the results of soil response analysis obtained by the three 1-D codes in terms of maximum acceleration at the surface, maximum acceleration with depth and amplification function using the V_s profiles obtained respectively by D-H4 (borehole 6) tests and using as input the scaled 1990 recorded accelerograms. According to the analysis of the soil response, response spectra concerning the investigated sites have been also deduced.

Figure 11 shows the response spectra (5% damping) obtained by the three 1-D codes using the V_s profiles obtained by D-H4 (borehole 6) tests and using as input the scaled 1990 recorded accelerograms.

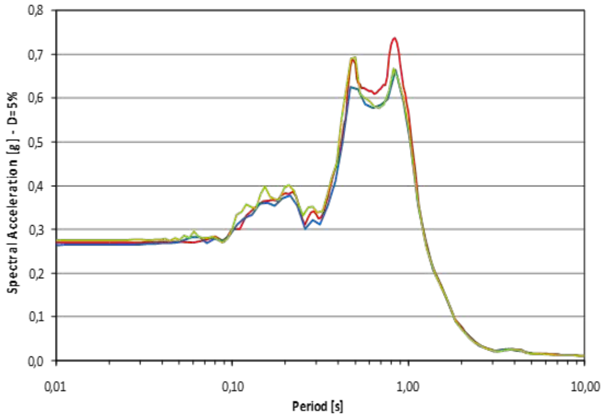

Figure 11: Response spectra (5% damping) obtained by the three codes EERA (red line), STRATA (blue line), and DEEPSOIL (green line) using the V_s profile of D-H4 (borehole 6) and using as input the 1990 scaled recorded accelerogram.

Amplification ratios obtained in seismic response analyses associated to different depths are reported in Figure 12.

According to the current Italian seismic code, the equivalent shear wave velocity, $V_{s,eq}$, is equal to 186 m/s for DH4 (soil type C) which correspond a soil amplification factor S_s = 1.385. Using this site amplification value, the horizontal acceleration at the surface must be 0.231 × 1.385 = 0.320 g, which is not in very good agreement with the acceleration obtained by the site response analysis. The horizontal acceleration at the surface obtained by the site response analysis is of about 0.270 g, with a soil amplification factor S_s of about 1.17.

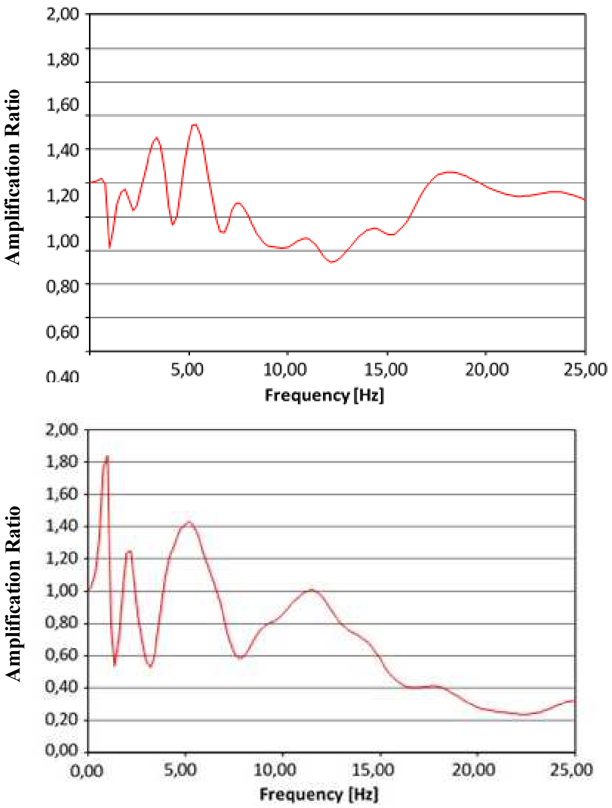

Figure 12. Amplification ratios in seismic response analyses associated to different depths, using as input the 1990 recorded accelerogram: Up, at the depth of 25 m using the V_s profile of D-H4 (borehole 6); Down, at the depth of 50musing the V_s profile of D-H4 (borehole 6).

The difference between the two soil amplification factors is due to the fact that $V_{s,eq}$ is not a key parameter in the evaluation of soil amplification. Effectively, the V_s value at the depth of 30 m provided by DH4 (borehole 6) is of about 230 m/s, not in correspondence with the conventionally adopted values for seismic bedrock.

The preliminary results seem to show some slight soil amplification effects due to the soil foundation and seem to indicate that, when using proper laboratory and field measurements of soil properties, one-dimensional soil amplification analyses can explain not only the possible trends of the intensity and distribution of damage to buildings but can explain the soil response, causing the possible severe damage of structures.

A careful dynamic soil properties determination for the Augusta Hangar site was due for the evaluation of the design spectrum to give it to the structural engineers for the design of the retrofitting of the hangar to resist the given earthquake.

Furthermore, elastic pseudo-acceleration response spectra at the ground surface have been compared in Figures 13 and 14 (graphically) with the reference ones, provided by the NTC 2018 Italian seismic code at ULS (both equivalent linear and nonlinear analyses) and DLS.

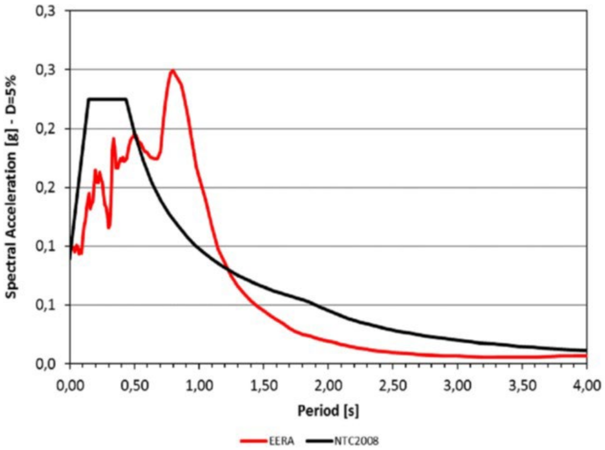

Figure 13. Comparison at DLS between elastic pseudoacceleration response spectra obtained by EERA code (using the V_s profile of D-H4 and using as input the 1990 scaled accelerogram) and NTC 2018 Italian seismic code.

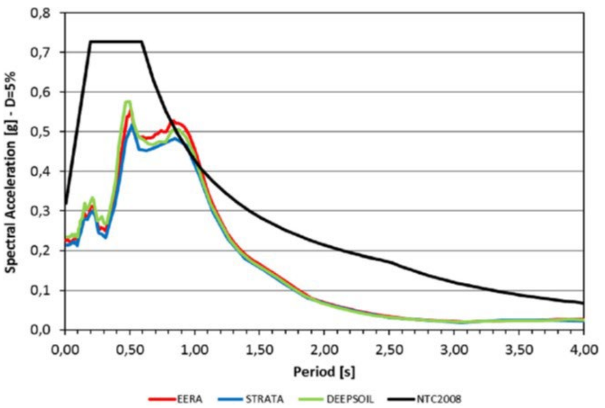

Figure 14. Comparison at ULS between elastic pseudoacceleration response spectra obtained by EERA, STRATA, and DEEPSOIL codes (using the V_s profile of D-H4 and using as input the 1990 scaled accelerogram) and NTC 2018 Italian seismic code.

7. Conclusions

A site characterization of Augusta Hangar area for seismic response analysis was presented. The present soil consists of silt and clay of medium-to-high plasticity characterized by medium-to-high values of the overconsolidation ratio. Available data enabled one to define for this site the small strain shear modulus G_o profile variation with depth.

On the basis of in situ test results, it is possible to stress that the small strain shear modulus measured in the laboratory is smaller than that obtained in situ by means of MPM and DH tests; this is probably due to a disturbance phenomenon occurred during sampling.

Local site response analyses have been brought for the site area at Ultimate Limitation State ULS by 1-D linear equivalent computer codes for the evaluation of the amplification factors of the maximum acceleration. Results of the site response analysis show some slight soil amplification effects of the soil.

Through 1-D performed numerical analyses, it has been possible to evaluate the influence of stratigraphic effects in seismic response of the site. The PGA values obtained from DEEPSOIL are sometimes higher than those obtained by EERA and STRATA codes.

In addition, seismic response analyses at Damage Limitation State DLS using as input selected accelerograms scaled with reference to the PGA at the bedrock corresponding to a probability of exceedance of 63% in 50 years have been added to the paper.

References

Baguelin F., Jezequel J.F., Le Mee E. and Le Mehaute A. 1972. "Expansion of Cylindrical Probes in Cohesive Soils". Journ. Of the Soil Mech. And Found. Div., ASCE, Vol. 98, SM11.

Baguelin F. and Jezequel J.F. 1978. "Le Pressiometre Autoforer". Annales de l'ISTBTP, 97, 135-159.

Baguelin F., Bustamante M. and Frank R. 1986. "The Pressuremeter for Foundations : French Experience". IN-SITU'86, Blacksburg.

Bardet, J.P., Ichii, K., Lin, C.H. (2000): "EERA - A Computer Program for Equivalent-Linear Earthquake Site Response Analyses of Layered Soil Deposits"; University of Southern California, Dept of Civil Eng.

Byre P. M., Salgado F. M. and Howie J. A. 1990. "Relationship between the Unload Shear Modulus from Pressumeter Tests and the Maximum Shear Moduli of Sand". Proceeding of the 3rd International Symposium Pressumeter, Oxford, 231-242.

Cavallaro A., Lo Presti D.C.F., Maugeri M. and Pallara O. 1999a. "A Case Study (The Saint Nicolò Cathedral) for Dynamic Characterization of Soil from in Situ and Laboratory Tests". Proceedings of the 2nd International Symposium on Earthquake Resistant Engineering Structures, Catania, 15-17 June 1999, 769-778.

Cavallaro A., Massimino M.R. and Maugeri M. 2003. "Noto Cathedral: Soil and Foundation Investigation". Construction and Building Materials, N°. 17, 2003, 533-541.

Cavallaro A., Cessari L. and Gigliarelli E. 2013. "Site Characterization by in Situ and Laboratory Tests for the Structural and Architectural Restoration of Saint Nicholas Church, Nicosia, Cyprus". Proceedings of the 2nd International Symposium on Geotechnical Engineering for the Preservetion of Monuments and Historic Sites, Napoli, 30-31 May 2013, 241-247.

Cavallaro A., Castelli F., Ferraro A., Grasso S. and Lentini V. 2018. "Site Response Analysis for the Seismic Improvement of a Historical and Monumental Building: The Case Study of Augusta Hangar". Bulletin of Engineering Geology and the Environment, Vol. 77, Issue 3, August 2018, Springer Germany, pp. 1217-1248. (ISSN: 1435-9529) DOI: 10.1007/s10064-017-1170-9.

Cavallaro A., Grasso S. and Sammito M.S.V. 2022. "A Seismic Microzonation Study for Some Areas Around the Mt. Etna Volcano on the East Coast of Sicily, Italy". Proceedings of the 4th International Conference on Performance-Based Design in Earthquake Geotechnical Engineering PBD-IV 2022, Beijing, 15-17 July 2022, Geotechnical, Geological and Earthquake Engineering, Vol 52. Springer, Cham, pp. 863-870. (Print ISBN: 978-3-031-11897-5, Online ISBN: 978-3-031-11898-2, ISSN: 15736059, EISSN: 18724671) DOI: https://doi.org/10.1007/978-3-031-11898-2_61.

Cavallaro A., Fiamingo A., Grasso S., Massimino M.R. and Sammito M.S.V. 2024. "Local Site Amplification Maps for the

Volcanic Area of Trecastagni, South-Eastern Sicily (Italy)". Bulletin of Earthquake Engineering, 2024. (ISSN: 1573-1456, ISSN-L: 1570-761X) DOI: 10.1007/s10518-023-01834-4.

CEN EC8. 1993. "Design of Structures for Earthquake Resistance". European Prestandard, ENV 1993. European Committee for Standardisation.

Fahey M. and Carter J. P. 1993. "A Finite Element Study of the Pressumeter Test in Sand Using a Non-Linear Elasto Plastic Model". Canadian Geotechnical Journal, 30, 348-362.

Fahey M. and Carter J.P. 1993. "A Finite Element Study of the Pressumeter Test in Sand Using a Non-Linear Elasto Plastic Model". Canadian Geotechnical Journal, 30, 348 - 362.

Ghionna V.N., Jamiolkowski M., Pedroni S. and Piccoli S. 1994. "Interpretation of Unload-Reload Modulus from Pressumeter Test in Sand". Proceeding of the Conference on the Pressumeter and its New Avenue, Sherbrooke, Canada, 471-480.

Groholski D., Hashash Y., Kim B., Musgrove M., Harmon J. and Stewart J. 2016. "Simplified model for small-strain nonlinearity and strength in 1D seismic site response analysis". Journal Geotech Geoenviron Eng. 04016042.

Jamiolkowski M., Lo Presti D.C.F. and Pallara O. 1995. "Role of In-Situ Testing in Geotechnical Earthquake Engineering. Proceeding of the 3rd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, April 2-7, 1995, vol. II, 1523-1546.

Kottke A.R. and Rathje E.M. 2008. "Technical Manual for Strata". PEER report. In: Pacific Earthquake. Engineering Research Center College of Engineering. University of California, Berkeley.

Laboratoire Central des Ponts et Chaussées (LPC). 1971. "Essai Pressiométrique Normal". Mode Opératoire MS.IS-2, Paris.

Laboratoire Central des Ponts et Chaussées (LPC) - Divis. Geotechnique. 1985. "Essai Pressiométrique. Méchanique des Sols N° 1". Paris.

Mair R.J. and Wood D.M., 1987. "Pressuremeter Testing Methods and Interpretation". CIRIA Ground. Eng. Report: In-Situ Testing.

NTC 2018. Italian Technical Standards for Construction - D.M. January 17, 2018.

Ng C.W.W. and Wang Y. 2001. "Field and Laboratory Measurements of Small Strain Stiffness of Decomposed Granites". Soils Foundation 41(3):57-71.

Tornaghi R. 1983. "Metodo Pressiometro Menard". 11° Ciclo di Conf. Di Geotecnica. Politecnico di Torino, N° 610.

Woods R.D. 1978. "Measurement of Dynamic Soil Properties". Earthq Eng Soil Dyn Pasadena 1:91-179.

Yokota K., Imai T. and Konno M. 1981. "Dynamic Deformation Characteristics of Soils Determined by Laboratory Tests". OYO Tec Rep 3:13-37.

INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

The paper was published in the proceedings of the 8th International Symposium on Pressuremeters (ISP2025) and was edited by Wissem Frikha and Alexandre Lopes dos Santos. The conference was held from September 2nd to September 5th 2025 in Esch-sur-Alzette, Luxembourg.