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Noto Cathedral: soil and foundation investigation

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Abstract

On 13th March 1996 the dome of the St. Nicolò Cathedral of Noto fell due to a post-seismic structural collapse. In order to study the soil–structure interaction a comprehensive laboratory and in situ investigation has been carried out to obtain a soil profile. In this paper the dynamic characterisation results and normalised laws are proposed to consider shear modulus decay and damping ratio increase with strain level. The existing foundations of the cathedral were investigated by means of excavations and tests on the stones and the mortar. In this way the foundations were subjected to visual inspections to detect their size and their embedment level. Now, the soil–foundation interaction has been analysed by means of the finite element code SOFIA, considering at this stage the superstructure weight through the influence area approach. In particular, the effects of the designed remedial work of the foundation have been studied, comparing the two configurations before and after the foundation improvement.

Keywords: Foundations; Soil analysis; Finite element analysis; Soil-structure interaction

1. Shear modulus and damping ratio from laboratory tests

The site investigation was performed within an area of 3200 mq (40×80 m^2) and reached a maximum depth of 81 m. Undisturbed samples were retrieved by means of an 86 mm Shelby tube sampler. The Pliocene Noto deposits mainly consist of a medium stiff, normal consolidated clayey-sand [1].

The values of the natural moisture content w_n prevalently range from 15 to 37%. Characteristic values for the Atterberg limits are $w_l=37$–58% and $w_p=17$–22%, with a plasticity index of PI$=15$–40%. These general characteristics and index properties of the Noto soil indicate a low degree of homogeneity with depth of the deposits. This dishomogeneity with depth is also confirmed by analysing the number of blows N_{SPT} from mechanical standard penetration test (SPT) performed over the investigated area. The soil deposits can be classified as inorganic soil of low to medium plasticity.

The Menard pressure meter tests, piezometer tests, down-hole and cross-hole tests, seismic tomography tests, ground penetrating radar tests and surface seismic tests have also been performed.

The Resonant Column/Torsional shear apparatus [3] was used for evaluation of Shear modulus G and damping ratio D of the Noto Cathedral soil.

The laboratory test conditions and the obtained small strain shear modulus G_o are listed in Table 1. The undisturbed specimens were isotropically reconsolidated to the best estimate of the in situ mean effective stress.

The same specimens were first subjected to cyclic loading torsional shear test (CLTST), then to resonant column test (RCT) after a rest period of 24 h with opened drainage. The size of solid cylindrical specimens are radius$=25$ mm and height$=100$ mm.

The G_o values, reported in Table 1, indicate a moderate but measurable influence of strain rate and type of loading even at very small strain where the soil behaviour is supposed to be elastic [4–7]. In order to appreciate the rate effect on G_o, it is worthwhile remembering that the equivalent shear strain rate ($\dot{\gamma}=240f\gamma$ [%]/s) experienced by the specimens during RCT can be three orders of magnitude greater than those adopted during CLTST. Moreover, the effects of the rate and loading conditions on the shear modulus are the same over the entire strain interval investigated where G_o(RC)/

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Table 1
Test condition for Noto Cathedral soil specimens

Test number	Boreholes	H (m)	σ'_{vo} (kPa)	e	PI	CLTST RC	G_o (1) MPa	G_o (2) MPa
1	S3C1	9.00	166	0.6410	15	U	92	116
2	S1C1	13.00	196	0.6113	20	U	64	77
3	S7C2	15.50	237	0.7178	27	U	68	84
4	S4C2	22.20	294	0.6298	29	U	100	116
5	S3C3	22.50	297	0.7490	22	U	178	190
6	S1C3	51.00	522	0.5840	36	U	221	237

Where U, Undrained; G_o (1) from CLTST; G_o (2) from RCT.

G_o(CLTST)$\cong$1.24. This experimental finding is different from that observed by Cavallaro [4], Lo Presti et al. [8] Tatsuoka et al. [9], who have shown an increasing rate effect with an increase of the strain level. This different behaviour can be tentatively explained by considering that in this study solid cylindrical specimens with a shear strain variable from zero (at the centre of the section) to a maximum value (at the edge) have been used, while in previous studies mainly hollow cylinder specimens were used. In the case of hollow specimens, the shear strain is quite constant along the radius.

Fig. 1 shows the results of RCTs normalised by dividing the shear modulus $G(\gamma)$ for the initial value G_o at very low strain in which $G(\gamma)$, strain-dependent shear modulus; γ, shear strain and α, β, soil constants.

The experimental results of specimens were used to determine the empirical parameters of the equation proposed by Yokota et al. [10] to describe the shear modulus decay with shear strain level:

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

Eq. (1) allows the complete shear modulus degradation to be considered with strain level. The values of $\alpha = 15$ and $\beta = 1.28$ were obtained for the Noto Cathedral foundation soil.

As suggested by Yokota et al. [10], the inverse variation of damping ratio in respect to the normalised shear modulus has an exponential form, like that reported in Fig. 2:

$$D(\gamma)(\%) = \eta \ \exp\left[-\lambda \frac{G(\gamma)}{G_o}\right] \tag{2}$$

in which $D(\gamma)$, strain-dependent damping ratio; γ, shear strain and η, λ, soil constants. The values of $\eta = 25.6$ and $\lambda = 1.952$ were obtained for the Noto Cathedral foundation soil. Eq. (2) assumes the maximum value $D_{max} = 25.6\%$ for $G(\gamma)/G_o = 0$ and a minimum value $D_{min} = 3.64\%$ for $G(\gamma)/G_o = 1$.

Therefore, Eq. (2) can be re-written in the following normalised form:

$$\frac{D(\gamma)}{D(\gamma)_{max}} = \exp\left[-\lambda \frac{G(\gamma)}{G_o}\right] \tag{3}$$

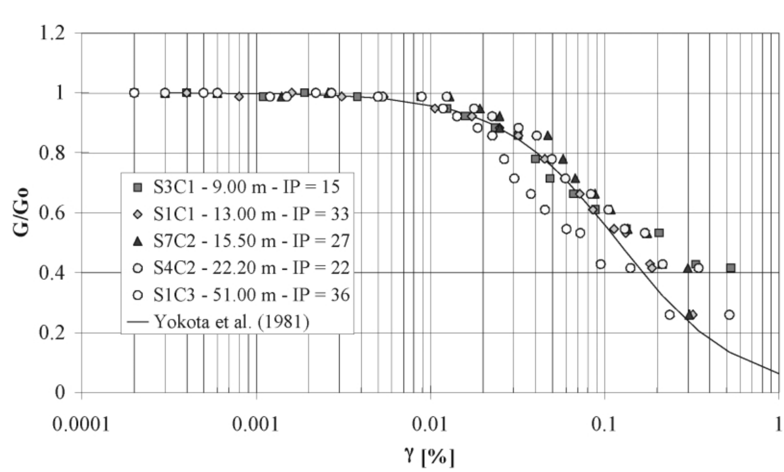

Fig. 1. G/G_o–γ curves from RCT tests.

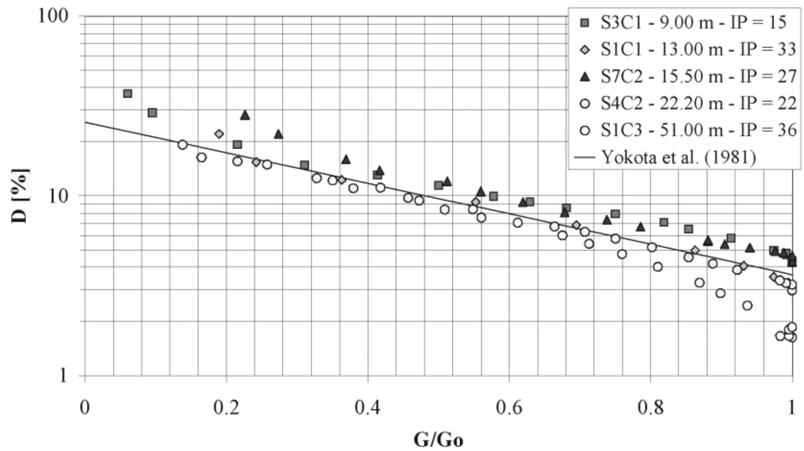

Fig. 2. D–G/G_o curves from RCT tests.

2. Shear modulus from in situ tests and evaluation from SPT tests

Fig. 3 shows the values of G_o vs. depth obtained in laboratory tests and those evaluated by means of the following empirical correlations based on SPT results.

(a) Ohta and Goto [11]:

$$V_S = 54.33(N_{SPT})^{0.173}\alpha\beta\left(\frac{Z}{0.303}\right)^{0.193} \tag{4}$$

where V_s, shear wave velocity (m/s); N_{SPT}, number of blows from SPT; Z, depth (m); α, age factor (Holocene = 1.000, Pleistocene = 1.303); β, geological factor (clays = 1.000, sands = 1.086).

(b) Yoshida and Motonori [12]

$$V_S = \beta(N_{SPT})^{0.25}\sigma'^{0.14}_{v0} \tag{5}$$

where V_s, shear wave velocity (m/s); N_{SPT}, number of blows from SPT; σ'_{vo}, vertical pressure; β, geological factor (any soil = 55, fine sand = 49)

$$G_o = \rho V_s^2 \tag{6}$$

where ρ, mass density.

Moreover, correlations of G_o with routine laboratory test results in static field could be established, like that given by Jamiolkowski et al. [14]

$$G_o = \frac{600\sigma'^{0.5}_m p_a^{0.5}}{e^{1.3}} \tag{7}$$

where $\sigma'_m = (\sigma'_v + 2\sigma'_h)/3$; $p_a = 1$ bar is a reference pressure; G_o, σ'_m are also expressed in bar. The values for parameters that appear in Eq. (6) are equal to the average values that result from laboratory tests performed on quaternary Italian clays and reconstituted

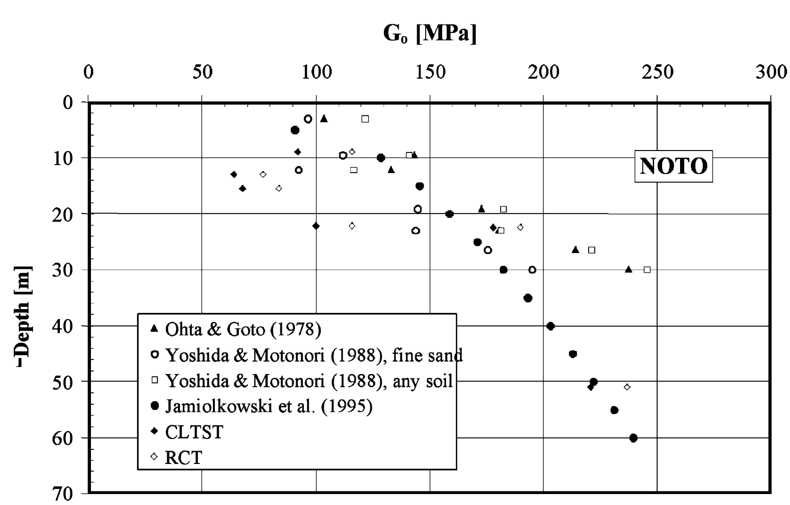

Fig. 3. G_o values from laboratory tests and different empirical correlations.

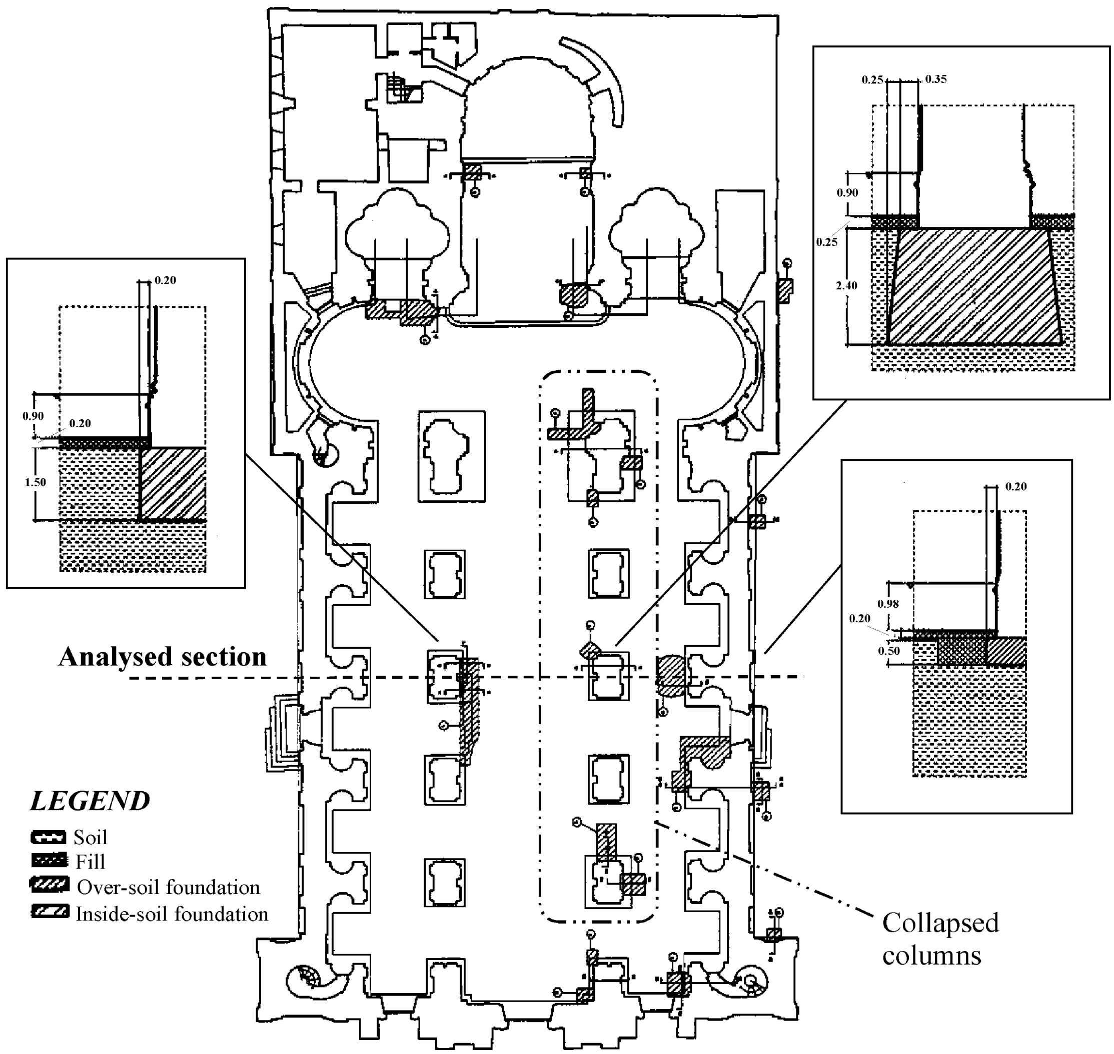

Fig. 4. Foundation plant in the existing configuration.

sands. Shibuya and Tanaka [13] proposed a similar equation for Holocene clay deposits.

The G_o values obtained with the methods indicated above are plotted against depth in Fig. 3. The method by Jamiolkowski et al. [14] was applied considering a given profile of void ratio and K_o.

In Fig. 3 the values of G_o measured in the laboratory from RCT and CLTST performed on undisturbed specimens are also shown. In the case of laboratory tests, the G_o values are determined at shear strain levels of less than 0.001%. A reasonable agreement between the laboratory results and the initial shear modulus values evaluated by means of the proposed empirical correlations is observed. On the whole, Eq. (6) seems to provide the most accurate trend of G_o with depth, as can be seen in Fig. 3.

3. The existing and new foundations

Even if the investigation proved that the reason for the collapse of the St. Nicolò Cathedral was not due to a foundation problem, neither in terms of settlements nor in terms of bearing capacity, the reconstruction project foresees the static and seismic improvement of the foundations. The major reason for the collapse was the poor quality of the masonry and in particular of the masonry of the columns that collapsed (Fig. 4). The choice of re-building the collapsed and surviving columns, such as the collapsed roof and dome, and to improve significantly the foundation, as it will be better explained below, allow us to have a good quality masonry and a symmetric configuration along the transversal direction and to achieve a more uniform response of the foundations, conveniently linked to each other.

The existing foundations were investigated by means of excavations and tests on the stones and the mortar. The investigations show that the quality of the foundation masonry is poor and similar to that of the columns. By means of the excavations it was also possible to detect the dimensions of each isolated part of the foundations. These dimensions vary greatly and some

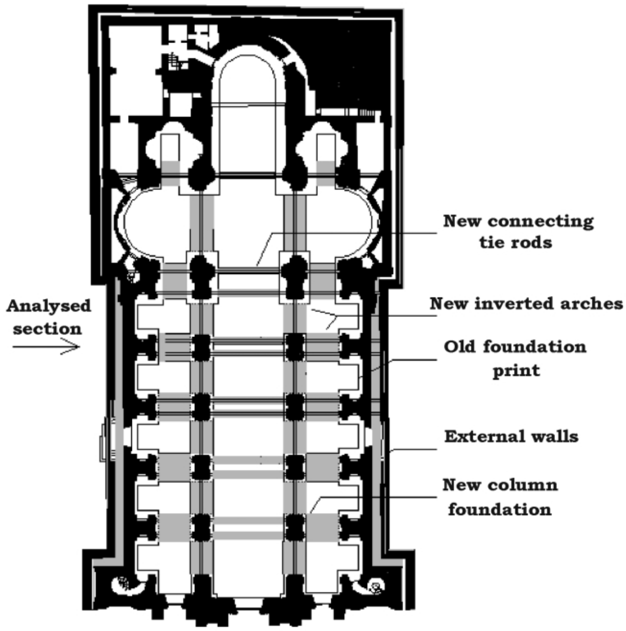

Fig. 5. Foundation plant in the new configuration.

of the foundations rest on the foundations of a previous church. Besides, the depth of the existing foundations is widely variable along the whole plant. Below the columns the foundation depth reaches a minimum value of 0.7 m and a maximum value of 2.40 m, without any symmetry. The external wall foundations are quite regular in plant, the embedment being approximately 0.5 m from the main entrance up to the middle of the cathedral, while from the middle to the altar end it increases, reaching 2.50 m at the end, due to the slope of the soil surface.

The reconstruction of the cathedral includes the collapsed and surviving columns and the collapsed roof and dome. At the same time the foundation will be rebuilt to improve its resistance in static and dynamic conditions. When a superstructure is submitted to remedial works, the improvement of the foundations must be taken into great consideration. The improvement of the superstructure allows it the absorbing of greater seismic actions that consequently are transferred to the foundation and then to the soil, through a complex soil–structure interaction phenomenon.

The previous single isolated foundations will be linked to each other by means of longitudinal and transversal tie rods and inverted arches (Fig. 5), to allow a better behaviour, reducing the horizontal differential displacements during seismic events. The new configuration also decreases the load per square metre transferred by the foundation to the subsoil and reduces the differential settlements.

The differences in the foundation depth will be conveniently reduced in the new configuration (Fig. 6). The foundation transversal configuration shown in Fig. 6 will be repeated along the longitudinal direction, achieving a more regular configuration even along this longitudinal direction.

The external walls will be underpinned by means of masonry where the embedment is only 0.5 m. The underpinning is limited only to the external part of these walls, so it can be ignored at this stage. Below the columns the new foundation will reach 2.5 m and finally in the inverted arches the depth will range between 0.5 and 1.45 m.

Fig. 6 also shows the analysed section mentioned in Figs. 4 and 5, and the schematisation used at this stage for the evaluation of the vertical load transferred from the superstructure to the foundations, considering four areas of influence.

4. The numerical analysis of the soil–foundation interaction for the old and new configurations

With the aim of investigating the effects of the foundation remedial works, the soil–foundation system (Fig. 7) is analysed by means of the SOFIA code [2,15–17], which is a non-linear finite element code, developed at the University of Catania, to study the soil–foundation-superstructure interaction in static and pseudo-static conditions. The code works in plane strain condition, but it is possible to take into account the real tridimensionality of the system by means of an approximated procedure, based on the Boussinesq theory [18].

As regards the St. Nicolò Cathedral of Noto, the results of the analyses for the old and new foundation configurations are compared. The analyses are developed considering only the structural weight and, at this stage, only a linear elastic behaviour for the soil and the foundation. The whole system is subdivided by means of isoparametric quadratic elements characterised by nine Gauss integration points.

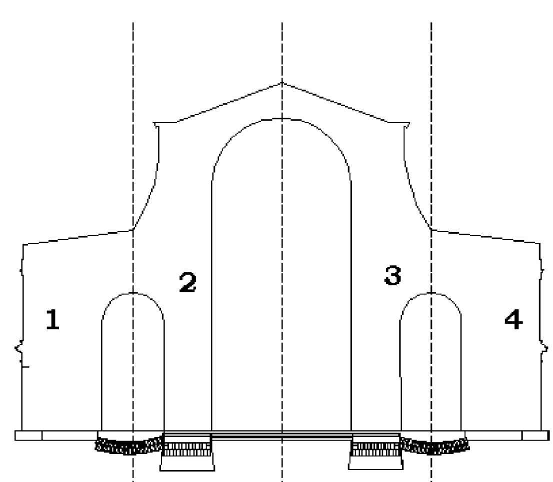

Fig. 6. Schematic view of the analysed cross-section.

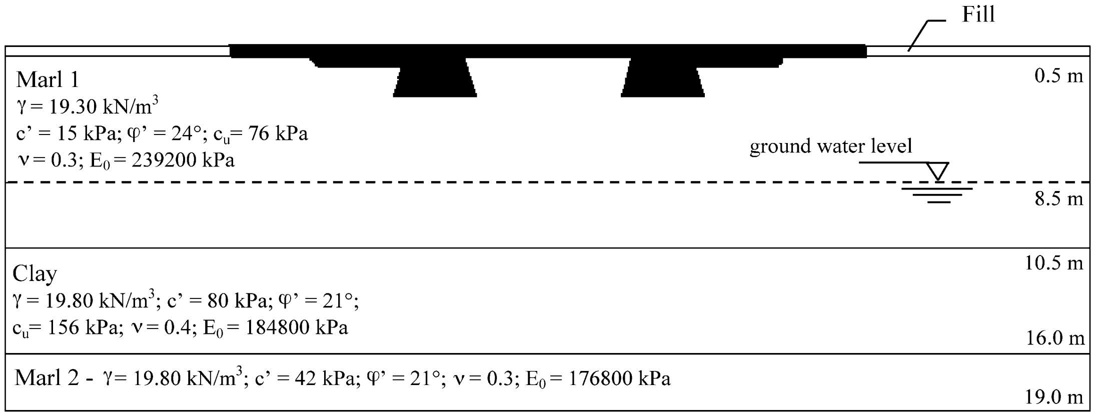

Fig. 7. Soil profile.

Table 2
The dimensions of the new foundations

Foundation	1	2	3	4	5	6
L (m)	5.00	4.20	3.20	3.20	4.20	5.00
D (m)	0.50	0.50–1.45	2.50	2.50	0.50–1.45	0.50

The element sizes decrease conveniently approaching the zone of the applied load, where the maximum variation of the stress and strain level is expected. Moreover, to reduce the boundary effects as much as possible, the two vertical boundaries, fixed in horizontal and tilting movements, are 12.00 m far from the foundation ends, while the lowest horizontal boundary, fixed in horizontal, vertical and tilting movements, is 19.00 m far from the soil surface. Fig. 7 shows the soil profile with the geotechnical properties of each layer, while Table 2 reports the length and depth of each foundation in the new configuration. The dimensions of the existing foundations are shown in Fig. 4. Moreover, for all the

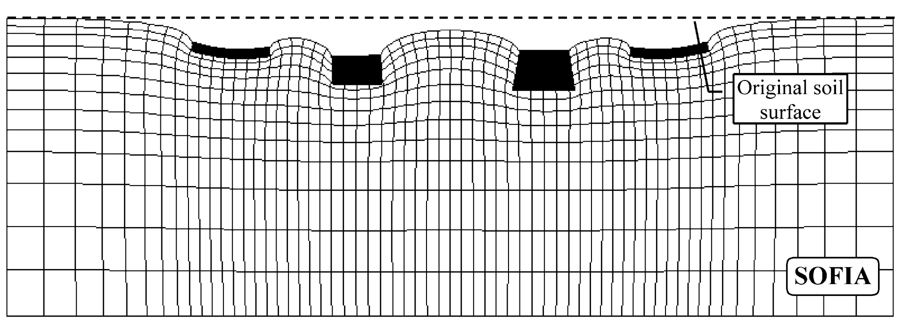

Fig. 8. Deformed configuration—pre-remedial work (movement amplification factor = 200).

Fig. 9. Vertical movements in subsoil (mm)—pre-remedial work.

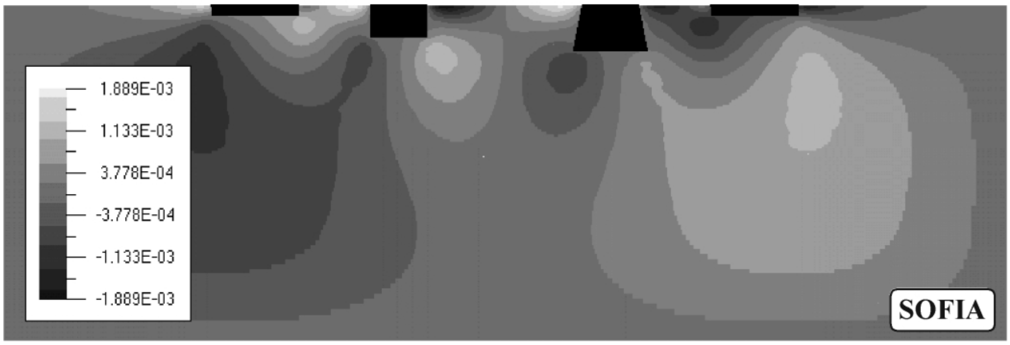

Fig. 10. Horizontal movements in subsoil (mm)—pre-remedial work.

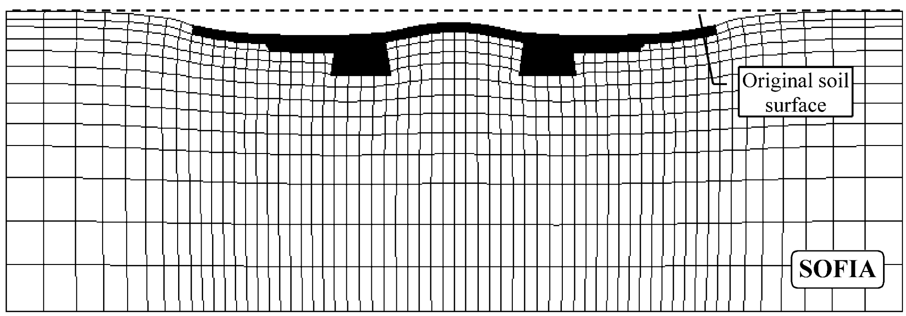

Fig. 11. Deformed configuration—post-remedial work (movement ampliflication factor = 200).

Fig. 12. Vertical movements in subsoil (mm)—post-remedial work.

foundations the following other parameters are fixed: $B = 4.20$ m; $\gamma = 19$ kN/m^3 and $\nu = 0.10$. The foundation Young modulus is fixed equal to 4×10^6 kPa for the old configuration and equal to 6×10^6 kPa for the new configuration.

Figs. 8–10 show, respectively, the soil–foundation deformed configuration and the vertical and horizontal movements in the soil before the remedial work, while Figs. 11–13 show the same information after the remedial work. Before the remedial work a maximum settlement $w_{max} = 12.6$ mm and a maximum horizontal displacement $u_{max} = 0.8$ mm are found for the foundations. These decrease after the remedial work, having, respectively, the following values: $w_{max} = 8.2$ mm and $u_{max} = 0.4$ mm.

The maximum differential settlement Δw_{max} is equal to 4.4 mm and to 3.2 mm, respectively, without and with remedial work for the foundation. The value of the differential settlement must be carefully analysed [19]. First of all, it should be linked to the distance L in which it takes place, so that the numerical and/or experimental ratio $\Delta w_{max}/L$ can be compared with the allowable values reported in literature. In the present case, the differential settlements equal to 4.4 mm (pre-remedial work configuration) regard two foundation points 9.2 m far from each other, one below an internal

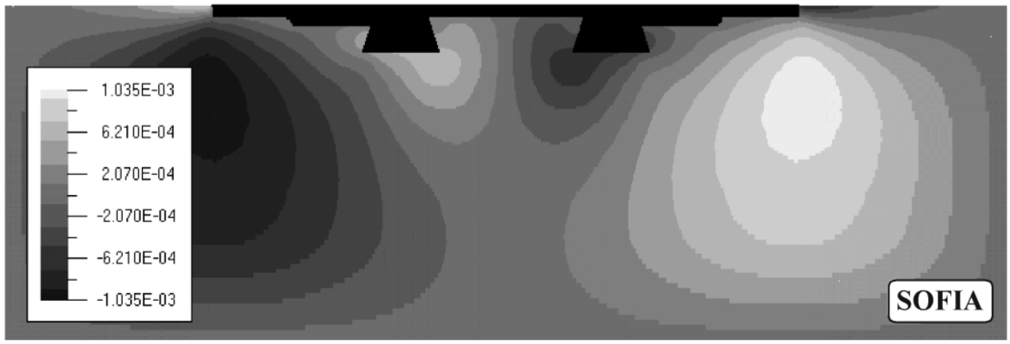

Fig. 13. Horizontal movements in subsoil (mm)—post-remedial work.

column and the other below the nearest external wall. The differential settlement equal to 3.2 mm (post-remedial work configuration) regards two foundation points 8.2 m far from each other, one below a cross tie rod, in proximity to the nearest internal column foundation, and the other below the external wall nearest to the above cross tie rod. The ratio $\Delta w_{max}/L$ is equal to 0.5×10^{-3} without foundation remedial work and to 0.4×10^{-3} with foundation remedial work.

Secondly, the evaluation of the differential settlements must be developed taking into account the construction process, which the cathedral will be subjected to. During the reconstruction the collapsed columns (Fig. 4) will be re-built together with the relative roof and the dome, the surviving columns will be also re-built, while the external walls will not be re-built. Then, the settlements computed underneath the external walls in the schematised post-remedial work configuration will not take place and thus the maximum differential settlement will be practically equal to the absolute settlement underneath the internal columns, that is, equal to 8.2 mm, as previously mentioned.

However, in the post-remedial work situation it is possible to note a more uniform behaviour of the foundation due to the linking system designed.

Besides, during the reconstruction process, the foundation settlements will be continuously measured by means of a sophisticated monitoring instrumentation. In the presented first step of analysis only the structural weight and an elastic-linear behaviour of the system are considered, while the seismic actions are not taken into account here. This first step of analysis is very important, because it allows the comparison between the numerical settlements with the experimental ones and then the calibration of the operational stiffness of the soil to improve the settlement prediction.

Fig. 8 shows different deformed areas for each separated foundation in the pre-remedial work situation, while after the designed foundation improvement it is possible to see a more uniform deformed area (Fig. 11). Finally, comparing Figs. 10 and 13 it is possible to observe smaller horizontal displacements in the last figure. The projected connections in the new configuration are a strategic foundation improvement to absorb seismic actions, very important for this part of Sicily, characterised by a high seismic geotechnical hazard.

5. Conclusions

As regards the soil characterisation, on the basis of the data shown it is possible to draw the following conclusions:

- Empirical correlations between the small strain shear modulus and the SPT results could be used to infer G_o from SPT.
- Normalised laws are proposed to consider shear modulus decay and damping ratio increase with strain level.
- The values of G_o were compared to those measured in laboratory tests. This comparison clearly indicates that a certain relationship exists between G_o and the SPT results, which would encourage one to establish empirical correlations for a specific site. Relationships like those proposed by Jamiolkowski et al. [14] or Yoshida and Motonori [12] seem to be capable of predicting G_o profile with depth in Pliocene deposits.

As regards the foundations, even if the real cause of the fall was the poor quality of the majority of the columns, the reconstruction project also takes into account the foundations. The reconstruction of the superstructure causes a redistribution and an increasing of the loads transferred to the foundations with displacement and/or bearing capacity problems and then possible dangerous consequences for the soil–structure interaction. In particular, the new foundation configuration decreases the load per square metre transferred from the foundation to the soil and reduces the differential displacements; this will be also more evident during seismic events, due to the designed foundation connections.

The analysis of the soil–foundation interaction, performed by means of the SOFIA code to estimate the effects of the foundation improvement, underlines the following factors:

- The internal connection of the whole foundation in the longitudinal and transversal sections reduces the localised tilting and the absolute and differential settlements in static conditions.
- The reduction of the differential settlements will be also more significant for the case of seismic loading, as it will be presented in the next work.
- This first step of analysis allows the comparison of the computed numerical foundation settlements with the ones that will be experimentally observed during the reconstruction process by means of a sophisticated monitoring system.
- The comparison between the numerical and the experimental foundation settlements in static condition could improve the reliability of the settlement prediction, by means of a back-analysis for the evaluation of the soil operational stiffness.

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