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The restoration of San Paolo Church in Pisa: geotechnical aspects

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Summary

San Paolo a Riva d'Arno is an important medieval church located in Pisa, on the south bank of the Arno river. Its existence is documented before 1032, most probably around 925: its actual configuration, however, was reached only at the end of 14th century.

Structural diseases, following damages by bombing during World War II, have affected the church over the last decades; cracks in the perimeter walls and problems to the wooden frames of the roof are progressively appearing. After recent earthquakes the evolution of the statical situation has forced the authorities to close the church to the public.

A comprehensive investigation on both the subsoil and the structure has been carried out with the aim of conceiving, designing and implementing suitable remedial works. Particular attention has been given to the collection of quantitative data about the foundation: layout, depth, thickness and state of conservation of the masonry. Geophysical techniques, such as electric tomography and ground penetrating radar, have been attempted with doubtful results; a special technique, consisting in small diameter holes drilled through the masonry with an instrumented drilling machine, was eventually developed.

To investigate earthquake effects, a thorough analysis of local amplification of seismic action has been performed. Local seismic response of the subsoil has been analysed by different techniques applied to two different subsoil models; the results obtained evidence of significant differences among the different analyses and in comparison with the prescriptions of the Italian Seismic Code.

Introduction

Severe earthquakes, with significant damages to historical buildings and monuments, have recently hit the central part of Italy; among them, we may recall the one occurred in 2009, which heavily damaged the city of L'Aquila. Similar episodes occurred in Assisi during the 1997 earthquake, showing once again that ancient structures can be highly vulnerable, even if they have survived over the centuries.

The attention to the preservation of monuments and historic buildings, one of the most challenging issues in Italy, further increased due to these events; in particular, the safety of places of worship has been carefully considered.

Within this frame, the paper reports the geotechnical investigation and analyses carried out in order to design the conservation measures of the Church of San Paolo a Ripa d'Arno, which was closed to the public in 2012 due to evident structural diseases.

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Brief history of church

The San Paolo a Ripa d'Arno church, located on the south side of the Arno River (Fig. 1), is one of the most important medieval churches of Pisa. Some sources report its existence before 1032, probably in 925; its present configuration, however, dates back to the end of 14th century. During the construction of the Cathedral in Piazza dei Miracoli, all religious functions took place in San Paolo Church by permission of Pope Alessandro II. At present, the Church consists in an Egyptian cross with three naves, a transept and an apse. The main façade, oriented towards west, was deeply remodelled at the beginning of 14th century by Giovanni Pisano. The interior of the Church is very simple, but includes some important frescos by Buffalmacco.

The present Church can be considered as the second one: in fact some documents describe the church built in the 10th century as a single nave and deeply different from the present one. The first church was probably built in correspondence of the chapel of a former monastery, of which some remains were discovered in the late 50s, during restoration works. The church increased its dimension from 1063: two lateral naves were added and after

Fig. 1 – Facade of the Church of San Paolo a Ripa d'Arno in Pisa.
Fig. 1 – Prospetto ovest della Chiesa di San Paolo a Ripa d'Arno.

Fig. 3 – North side of the Church.
Fig. 3 – Prospetto nord della Chiesa.

1118 the Church was further extended with the construction of the transept.

Between 1149 and the end of XIV century the Church did not undergo any structural intervention.

In the XIII century a bell tower was built, which was structurally independent by the Church and was located on north side of the main facade. At present, no traces of the bell tower are visible: it was destroyed by the Florence army in 1509 and never rebuilt.

After the destruction of the bell tower, the Church has been subjected only to maintenance, particularly of the wooden roof, up to Word War II.

On August 31st, 1943 the southern side of Pisa was intensely bombed and the Church was heavily damaged (Fig. 2), with the destruction of the wooden roof and the partial collapse of the southern wall. The reconstruction, which lasted 16 years, took place between 1944 and 1960.

The history of the construction reflects in the structural characteristics of the Church: the northern wall (Fig. 3) is different from the southern one (Fig. 4) thanks to the monastery that bordered the Church. The layout of the foundations was also influenced by the different phases of construction; as a matter of fact, the different portions of the Church have a different geometry of foundation. It is well known that the southern wall is weaker than the northern one and poorly arranged not only in the reconstructed portion, but also in the original one.

After the Emilia earthquake in 2012, the Church has been closed to the people, because of roof instability. The subsequent check of structural safety, however, put into evidence that the whole structure was unsafe and some interventions were needed. In particular, safety against seismic actions was a critical aspect; an accurate definition of seismic input was therefore necessary for a proper design of interventions.

Details about structural problems are out of the scope of the present paper, which focuses on the investigations of soils and foundation system.

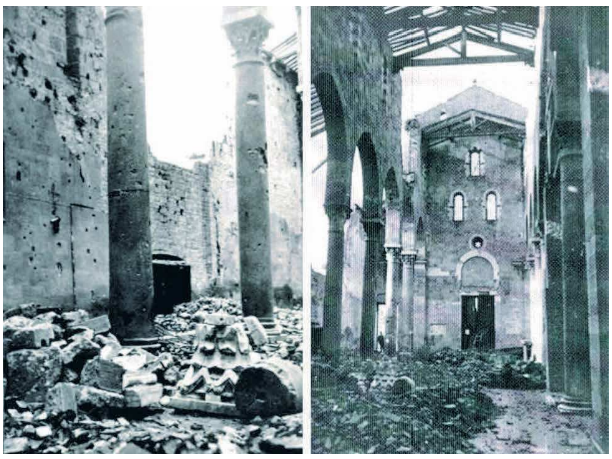

Fig. 2 – Damages due to bombing of 1943.
Fig. 2 – Danni conseguenti al bombardamento del 1943.

Fig. 4 – South side of the Church.
Fig. 4 – Prospetto sud della Chiesa.

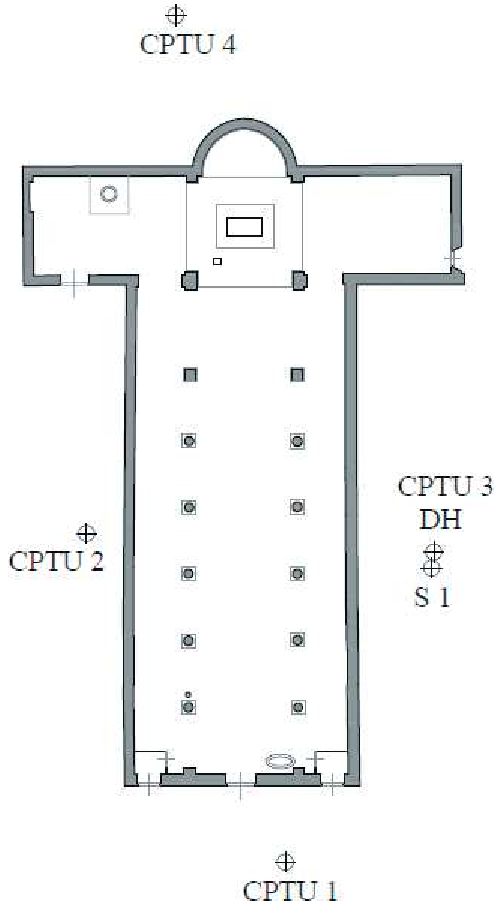

Fig. 5 – Plan of geotechnical investigations.
Fig. 5 – Planimetria delle indagini effettuate.

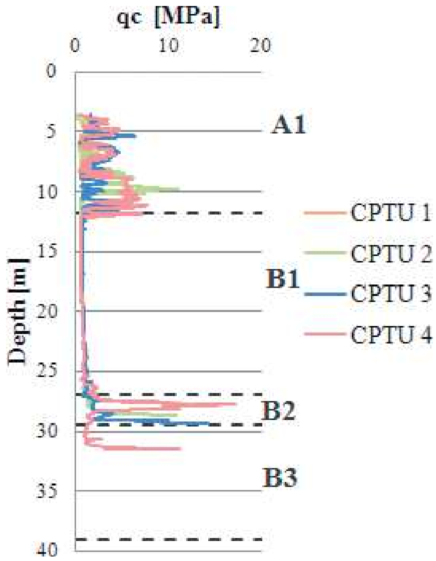

Fig. 6 – CPTU point resistance profiles.
Fig. 6 – Andamento con la profondità della resistenza alla punta.

Soil characterization

A brief research revealed that there was a complete lack of information about the subsoil; accordingly, a 40 m borehole and four CPTU profiles were performed at the beginning of 2014 (Fig. 5). Eleven undisturbed samples have been retrieved in the borehole between 3.00 and 36.00 m depth by means of an Osterberg sampler (Tab. I); furthermore, a Casagrande type piezometer was installed at the depth of 11 m.

The CPTU 3, located at the south side of the Church, was carried out by means of a probe equipped for the measurement of shear velocity Vs; a profile of the velocity down to 32.5 m below ground surface was thus obtained.

Besides the usual classification tests – grain size distribution and Atterberg limits – six oedometer tests, six triaxial consolidated undrained tests and six resonant column tests have been carried out.

Soil profiles deduced by borehole and CPTUs revealed an essentially horizontal layering (Fig. 6). The sequence of strata is very similar to that of Leaning Tower subsoil [VIGGIANI and PEPE, 2005]; the same denomination of strata was therefore adopted (Fig. 7).

Below these two groups of strata, starting from 39 m deep there is a medium sand, whose thickness will be discussed in the following. The similitude with the Tower subsoil will be used in definition of seismic action, in particular for soil characterization between 40 and 70 m depth.

The soils that were found between the ground surface and a depth of 11.8 m are named Complex A. Below a shallow layer of made ground about 2.5 m thick, Complex A consists mainly of silty sand with a thickness of about 9 m. This layer features a very high value of friction angle (about 40 degrees (Fig. 8), as deduced by triaxial tests on undisturbed samples).

Below the complex A, an essentially clayey Complex B 27.2 m thick is found; it may be subdivided in

Tab. I – Specimens retrieved in borehole S1.
Tab. I – Sondaggio S1: elenco dei campioni indisturbati.

Sample	Depth (m)
C1	3.0-3.5
C2	6.0-6.5
C3	9.0-9.5
C4	10.5-11.0
C5	13.5-14.0
C6	16.0-16.5
C7	19.5-20.0
C8	24.0-24.5
C9	27.0-27.5
C10	30.0-30.5
C11	36.0-36.5

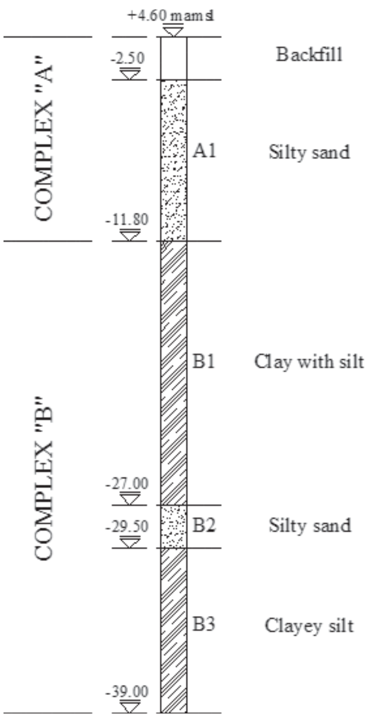

Fig. 7 – Stratigraphic model.
Fig. 7 – Rappresentazione del modello stratigrafico.

turn into three different layers. The shallower layer B1, between 11.8 m and 27 m depth, consists of a very compressible clay with silt, normally consolidated or slightly overconsolidated (OCR lower than 2, with an average value of 1.3). Layer B2 is a silty sand and layer B3 is a clayey silt with a PI lower than layer B1.

Figures 9 to 11 report the main parameters of described layers.

From the point of view of dynamic properties, figure 12 shows the profile of Vs as determined in CPTU 3; it is evident that the value of Vs is lower than 200 m/s down to 25 m depth.

The dynamic characterization of the subsoil includes six resonant column tests, which provide information about the dependence of stiffness and

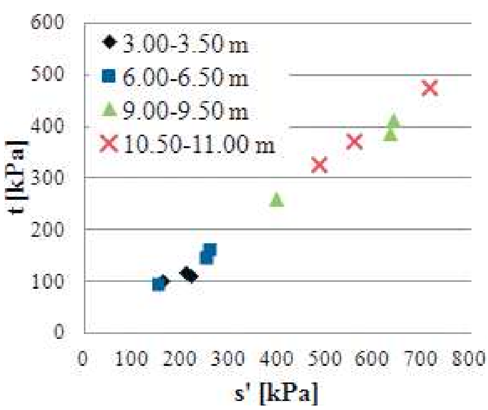

Fig. 8 – Failure stresses of A1 layer.
Fig. 8 – Stato tensionale in condizioni di rottura per lo strato A1.

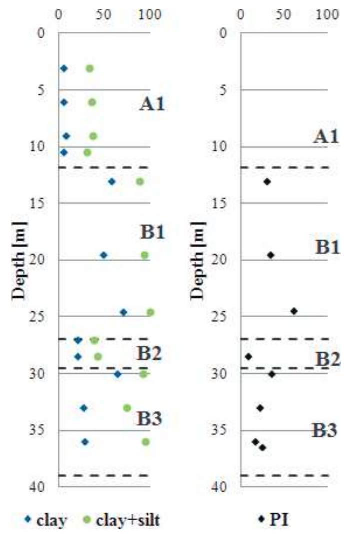

Fig. 9 – Grain size distribution and plasticity index profiles.
Fig. 9 – Andamento con la profondità della composizione granulometrica e dell'indice plastico.

damping from the deformation. More homogeneous layers have been characterized by means of one test, whereas layer A1 needed three tests. The results of RC tests are reported in figure 13 and 14.

Foundation investigation

Electric tomography

Since no information was available about the foundation system, an electrical tomography (ERT)

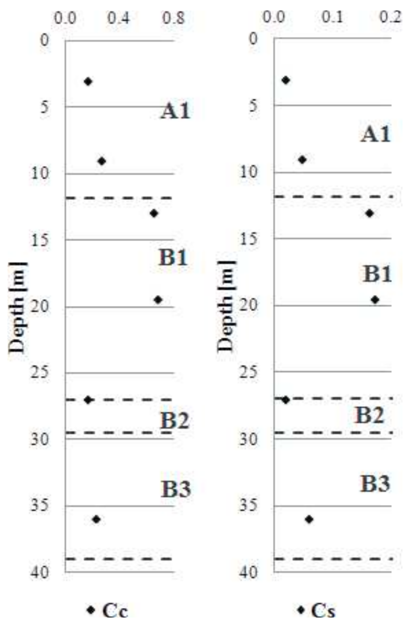

Fig. 10 – Compressibility parameters.
Fig. 10 – Andamento con la profondità dei parametri di compressibilità.

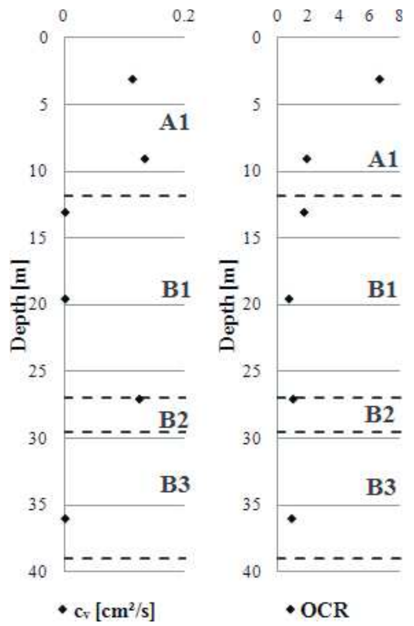

Fig. 11 – Consolidation coefficient and OCR profiles.
Fig. 11 – Andamento con la profondità del coefficiente di consolidazione e del OCR.

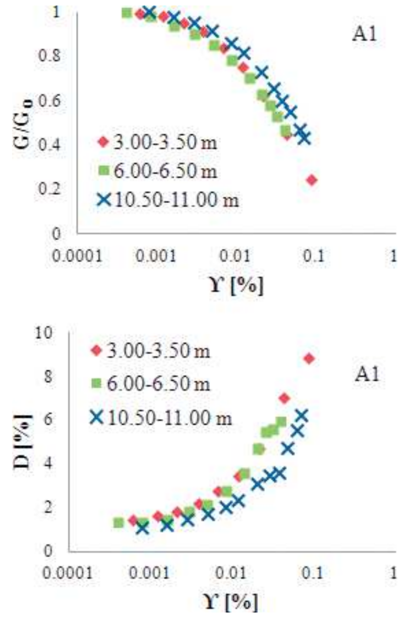

Fig. 13 – Modulus decay and damping for A1 layer.
Fig. 13 – Decadimento del modulo di rigidezza tangenziale e smorzamento per lo strato A1.

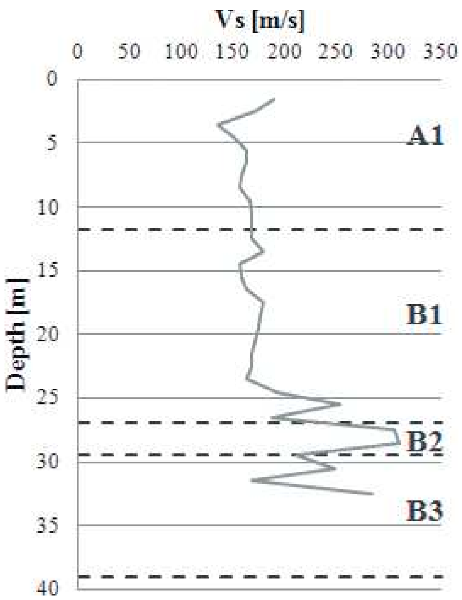

Fig. 12 – Profile of shear wave velocity.
Fig. 12 – Andamento con la profondità della velocità delle onde di taglio.

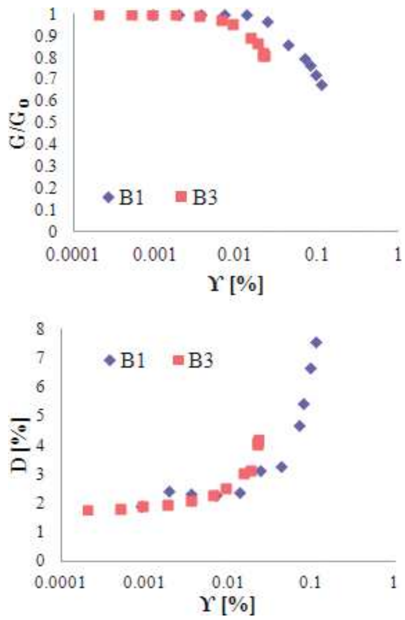

Fig. 14 – Modulus decay and damping for B1 and B3 layers.
Fig. 14 – Decadimento del modulo di rigidezza tangenziale e smorzamento per gli strati B1 e B3.

has been carried out as a first step. The test consists in the installation of a number of electrodes in the soil and the application of an electrical current using some of the electrodes as active ones and the other as passive. The result of the test is a three dimensional map of apparent resistivity of the material passed by the electrical current; the focus is on the variations of apparent resistivity rather than its absolute values. The investigation involved the whole foundation system of the Church with the installation of more than 100 electrodes, in order to investigate the whole plan of the Church up to 14 m depth. A partial view of the results is reported in figure 15, whereas figure 16 shows a vertical section with details concerning the geometry of the foundations.

Usually, the interpretation of such tests is based on the changes in apparent resistivity: in the present case an increment of resistivity has been associated to the occurrence of masonry; in figure 16 the foundation seems 2.5 m deep and some meters wide. As reported in figure 15, information about foundation of columns is not available. Such lack of information is due to presence of a metallic grid below internal pavement, discovered after ERT execution.

An accurate analysis of electric tomography and a comparison with direct inspection (say excavation) revealed that the information deduced by means of ERT is not reliable. The reasons of such unreliability are probably connected to the small difference in resistivity between the foundation walls and the ground, rather than the presence of the metallic grid. In fact, similar results have been obtained at the Baptistery in Piazza dei Miracoli, in which the metallic grid below the internal pavement is not present, while the material of foundation masonry is similar (Fig. 17). Further investigation revealed that the foundation were deeper than 2.5 m and the ERT tests have been discarded since the technique was evidently unable to discriminate the masonry from the ground.

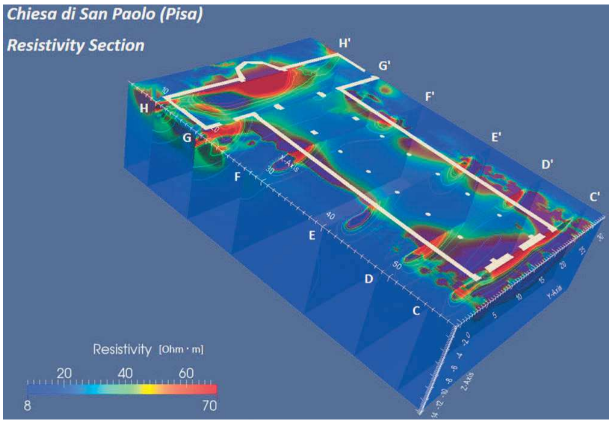

Fig. 15 – Results of electrical tomography.
Fig. 15 – Vista d'insieme dei risultati delle tomografie elettriche.

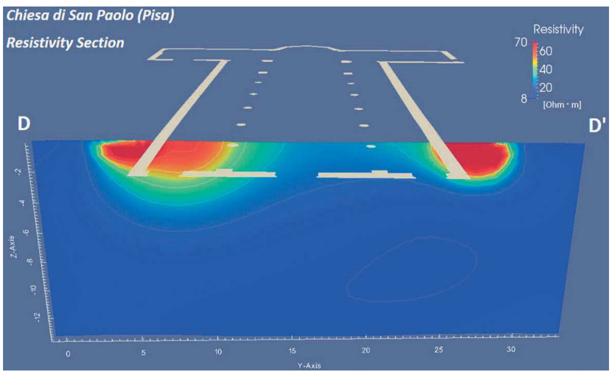

Fig. 16 – Results of electrical tomography.
Fig. 16 – Risultati delle tomografie elettriche per le fondazioni della Chiesa di San Paolo.

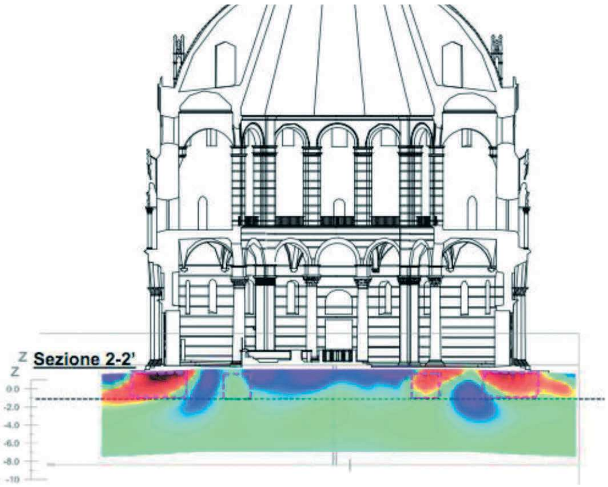

Fig. 17 – Results of ERT for Baptistery.
Fig. 17 – Risultati delle tomografie elettriche sul Battistero di Piazza Duomo. Si noti la "presenza" della fondazione anche sul lato esterno.

Ground penetrating radar

The difficulties connected to the ERT execution and interpretation led to application of other non-invasive techniques to get information about foundation masonry, especially for columns foundations. The Ground Penetrating Radar (GPR) has been tested; it should allow the individuation of objects

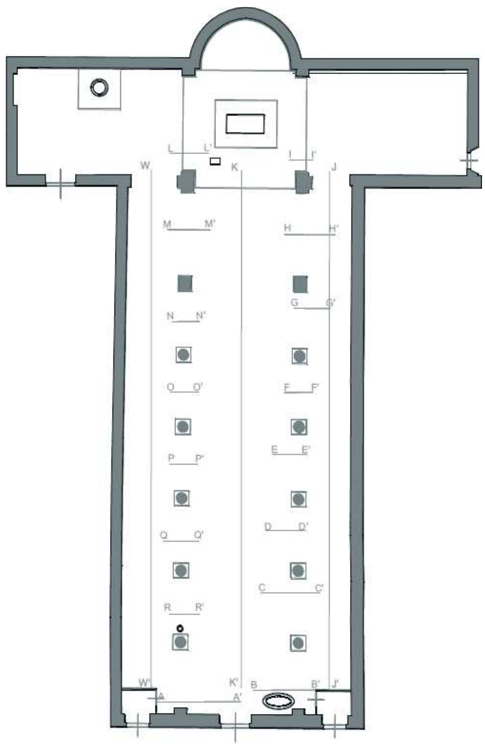

Fig. 18 – Plan of ground penetrating radar investigation.
Fig. 18 – Tracciato delle prospezioni tramite georadar. Il tracciato tiene conto anche della presenza del ponteggio di sostegno della copertura.

with physical properties different from the surrounding materials. In the case of buried structures the GPR technique allows the individuation of upper boundaries only. This limitation is due to the fact that the emitted signal has a single wavelength; in order to change the depth of exploration a change in wavelength would be needed, and usually the apparatuses do not allow the modulation in wavelength. Figure 18 shows the traces of nineteen paths used for the investigation. The paths were chosen taking into account the presence of a very pervasive scaffold sustaining the whole roof.

Figure 19 shows a typical example of GPR results, in which the presence of a buried structure is revealed. As in the case of ERT results, also the GPR results are difficult to read. In general the GPR put in evidence the presence of a connection between the columns in east-west direction and a connection in coincidence of the two columns near to the facade: probably the columns were erected in the 14th century, reusing the walls foundation belonging to the first Church in the 10th century.

In conclusion, the GPR technique allowed the individuation of some buried structures between the columns. Only the depth of the upper part of such structures is defined, and no information about the depth of the foundation plan is available. The GPR technique, therefore, is ineffective in defining the geometry of the masonry walls, at least using the current equipment.

Instrumented perforations

The above investigations provided some information about the overall layout of the foundation, but missed the geometrical information needed to evaluate the geotechnical performances of the foundation.

In order to obtain a reliable evaluation of foundation width and depth, a special instrumented coring machine was developed. The small diameter coring provided the measurement of the rate of penetration, the axial force applied, the pressure of cooling water and the electrical energy consumption. Improved by such sensors, the instrumented coring can be considered a sort of penetration test, rather than a machine to recover samples of masonry. As matter of fact, the possibility to retrieve good quality samples from masonry is connected to the quality of masonry itself and the diameter of borehole. In the case of very small diameter, say 35 ÷ 40 mm, only a very good masonry or large stones allow the retrieval of intact cores. The recorded parameters, therefore, are the only data available to judge the quality of masonry and to deduce its geometry. At present, it is not yet possible to directly relate the recorded parameters to the masonry characteristics; as a consequence, the identification of masonry is deduced by comparison. Figure 20 shows an example of the results in terms of rate of penetration versus depth. In the figure is evident the role of rate of penetration in defining the geometry of the foundation.

Information about the whole foundation system can be summarized as follows:

– columns rest on a masonry wall 4 m deep with a base at least 1 m wide. This wall connects all columns in E-W direction. The columns are not connected in N-S direction with the exception of western columns, where a masonry wall based at 3m depth has been found. This system probably coincides with the foundation of the smaller church built at the beginning of X century;
– all walls foundations are 4 m deep, with a base no more than one meter wide.

This information, summarised in figure 21, is somewhat unexpected. There is about a century between the two part of the construction, but the depth of the foundations of the different portion is quite the same and rather large.

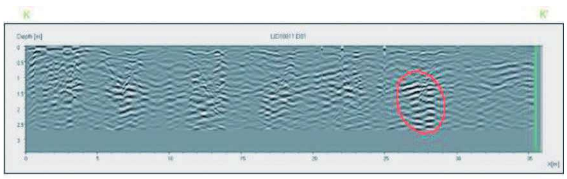

Fig. 19 – Results for section K.
Fig. 19 – Risultati ottenuti in corrispondenza della sezione K.

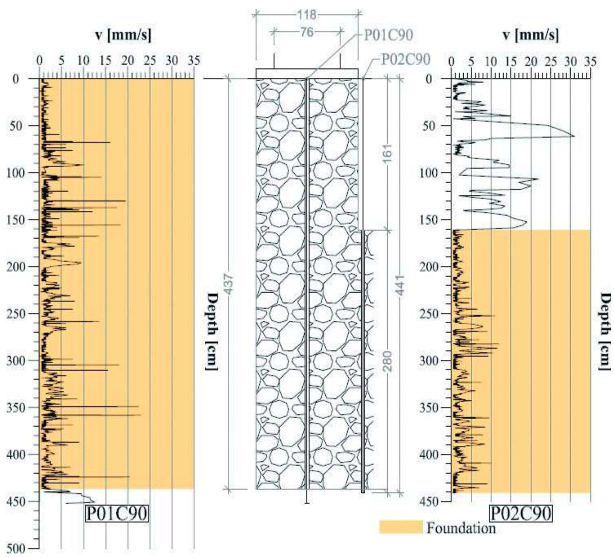

Fig. 20 – Results of instrumented coring.
Fig. 20 – Risultati tipici ottenuti mediante la carotatrice strumentata.

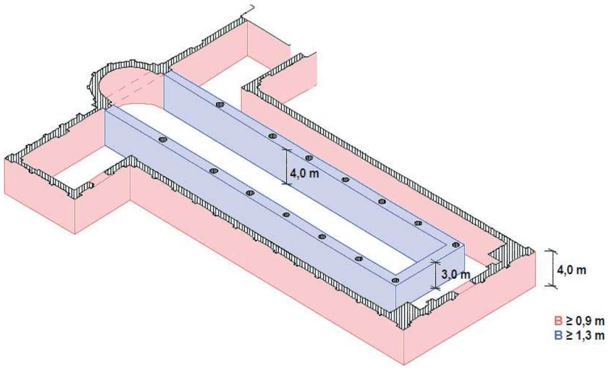

Fig. 21 – Scheme of foundation system deduced by the whole investigations.

Fig. 21 – Schema della fondazione della Chiesa di San Paolo così come dedotto dall'insieme delle prove.

A hand excavation down to 4 m depth and under water does not appear credible. In an alternative hypothesis, the original foundations were shallower, and then the ground level has been risen by natural sedimentation or artificial infill to protect the church by the Arno river floods.

Site effect analysis

For the analysis of seismic performances of the structure, apart the structural problems due to roof decay and WWII bombing damages, a careful evaluation of the expected seismic action is required. A routine site response analysis has been firstly carried out, but it was immediately evident that the extremely deep bedrock (say more than 1000 m) could significantly affect the results.

As reported before, the dynamic properties of the subsoil have actually been investigated down to 40 m depth, by means of the Vs profile and the Resonant Column tests (Figs. 12, 13, 14). Accordingly, in a first model the bedrock has been considered at a depth of 40 m below ground surface (Tab. II). In a second model, the information obtained by the investigation carried out directly at the Church site and the information collected on several adjacent sites have been considered; these allow the definition of the soil profile and properties down to 110 m depth. Table II summarizes the relevant data of the first model; the definition of second model needs however some additional discussion.

Tab. II – Main characteristics of Model 1.

Tab. II – Profilo sismostratigrafico del Modello 1.

Layer	Depth [m]	Thickness [m]	V_S [m/s]	[kN/m^3]
A1	0	11.8	162	19.22
B1	11.8	15.2	175	16.84
B2	27	2.5	252	20.37
B3	29.5	10.5	234	20.24

A one-dimensional model for the analysis of seismic site effects requires the definition of three ingredients: geometry (i.e. thickness of the layers), stiffness (shear wave velocity profile) and dynamic properties of layers (stiffness decay and damping). Until 40 m the two models are obviously coincident. The information deduced by several wells for water extraction provided the information about geometry below 40 m depth; figure 22 shows a hydrogeological section of the Church area. In particular the hole P5, near to the Church site, suggests the occurrence of a "bedrock" at a depth of 110 m. Figure 22 shows two additional layers: a sandy layer C starting from 39 m depth and a clayey layer D starting from 65 m depth.

To define the dynamic properties of layer C, reference has been made to cross-hole measurement performed near the Leaning Tower. Figure 23 shows a comparison between Vs profile of Church site and Leaning Tower site, in which the satisfactory overlap of the two profiles is evident. About G/G_0 decay and dumping variation, the curves proposed by IDRISS [1990] have been used.

Below 70 m depth no measurements of shear wave velocity is available, and the values of small strain stiffness of layer D have been evaluated by means of empirical relationships. The relation (1) proposed by JAMIOLKOWSKI *et al.*[1995] for Pisa clay has been used

$$G_0 = 480 e^{-1.43} \sigma_v'^{0.22} \sigma_h'^{0.22} p_a^{0.66} \quad (1)$$

Although some Authors (*e.g.* HAMILTON, 1976) propose a linear relationship between Vs and depth, the relationship (1) has been used to extra-

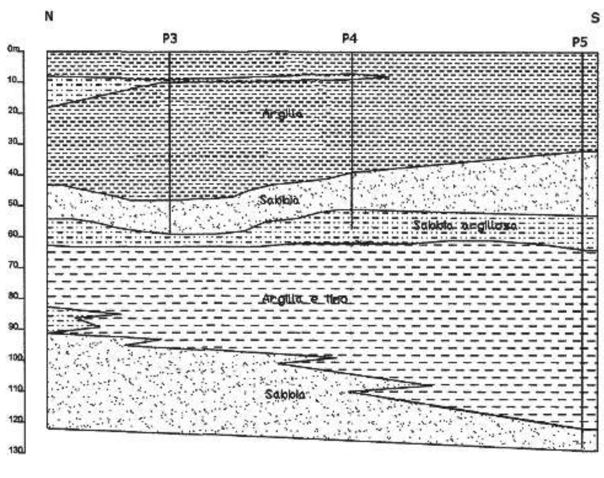

Fig. 22 – Recovered information about subsoil.

Fig. 22 – Sezione stratigrafica ricostruita da informazioni provenienti da siti adiacenti.

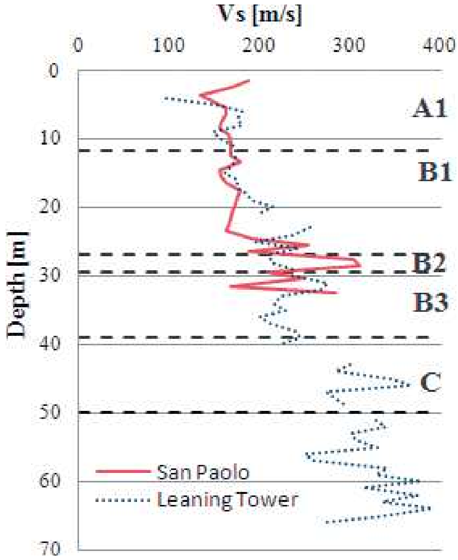

Fig. 23 – Comparison between Vs profile of Church site and Leaning Tower site.
Fig. 23 – Confronto tra il profilo della velocità delle onde di taglio misurato nel sito della Chiesa di San Paolo e quello misurato in prossimità della Torre Pendente.

Tab. III – Main characteristics of Model 2.
Tab. III – Profilo sismostratigrafico del Modello 2.

Layer	Depth [m]	Thickness [m]	V_S [m/s]	[kN/m^3]
A1	0	11.8	162	19.22
B1	11.8	15.2	175	16.84
B2	27	2.5	252	20.37
B3	29.5	9.5	234	20.24
C	39	26	317	20.37
D	65	45	358	19.00

polate the values of small strain stiffness of the clayey layer D between 65 and 110 m depth. For G/G_0 decay and dumping variation, the relations proposed by VUCETIC and DOBRY (1991) have been used. Table III reports the main features of the second model.

The site effects analysis has followed the usual steps, as follow:

– Definition of fundamental seismic parameters using the software Spettri-NTC ver. 1.03 [CSLP, 2008]. Within this step the definition of a reference period is mandatory. Usually the Italian Code defines a reference period in function of the structural characteristics and typology, whereas in this particular case the definition of such period is not “automatic”. In fact, assuming the reference period as the actual lifespan of the structure, which is measured in centuries, the definition of seismic action leads to unreasonable results. According to recent tendencies and regulations [MIBAC, 2011] the reference period has been defined as the time interval between two subsequent structural interventions. This period, in accordance with Superintendence Officers, has been fixed in 50 years with an excedence probability of 10% (SLV). Figure 24 shows the comparison between the response spectra of a type C soil and a type A soil as prescribed by the Italian Code; the differences between two graphs are intended to represent site amplification.
– The type A soil response spectrum is the input information for the site effect evaluation. This has been obtained by means of selection of seven time histories of acceleration so as their mean response spectrum was similar to that of type A soil. This selection have been done by means of the software SCALCONA 2.0 [LAI

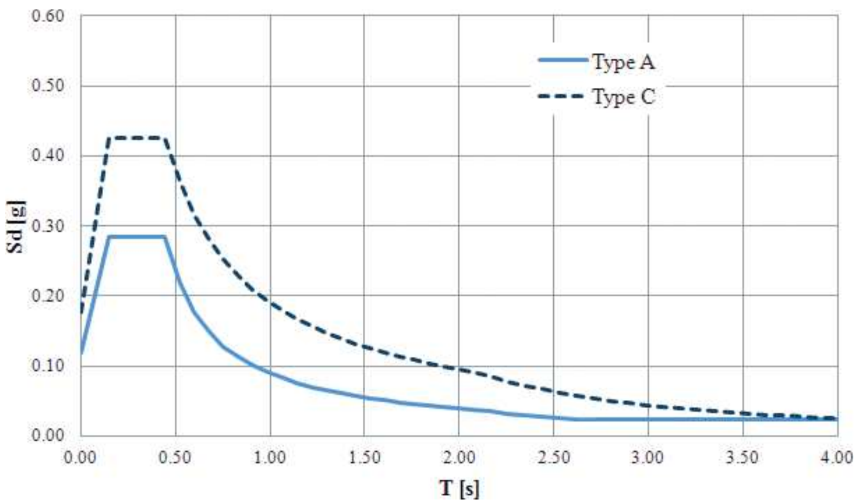

Fig. 24 – Response spectra as suggested by Italian Code for type C and type A soils.
Fig. 24 – Spettri di risposta suggeriti dalle Norme Tecniche per le Costruzioni del 2008.

and ZUCCOLO, 2012], specially created to perform this selection in Tuscan territory. This software allowed smaller scale factors – maximum was 1.47 – among that obtained by means of other software (e.g. REXEL [IERVOLINO *et al.,* 2010]).

– Selected time histories of acceleration have been processed by means of two different software devoted to one-dimensional site effect analysis: STRATA [KOTTKE and RATHJE, 2008] and DEEPSOIL [HASHASH, 2009]. The first software allows only a linear equivalent analysis, whereas the second one allows both linear equivalent and nonlinear analyses. In linear equivalent analysis the time history of acceleration has been processed in the frequencies domain and in non-linear analysis it has been processed in the time domain.

Figures 25 and 26 shows the results of a linear equivalent site effect analysis for both the subsoil models considered. For sake of comparison, the response spectrum provided by the Italian Code has been also reported in both figures. The similarity of results is quite evident, showing the substantial equivalence of the two software. The effect of the different soil model seems to consist in a more regular diagram below a period equal to 0.75 second and in a significant relative increase of amplitudes above the same limit.

Figure 27 shows the results of the non linear analysis performed by means of DEEPSOIL. In this case the differences between the two soil models are again evident above the period of 0.75 seconds, but the amplitudes in the first part of the spectrum are smaller than those of the previous case.

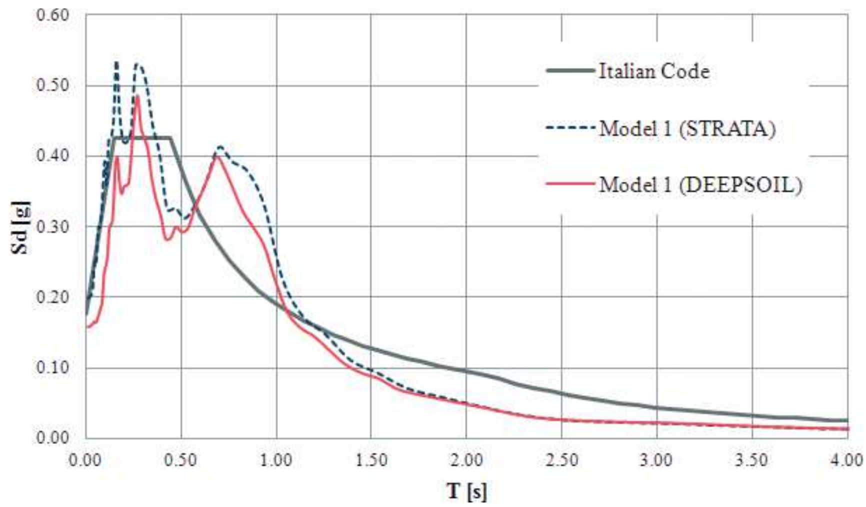

Fig. 25 – Comparison between results of two software for Model 1.
Fig. 25 – Confronto tra I risultati dei software STRATA e DEEPSOIL per il Modello 1 ed analisi lineare equivalente.

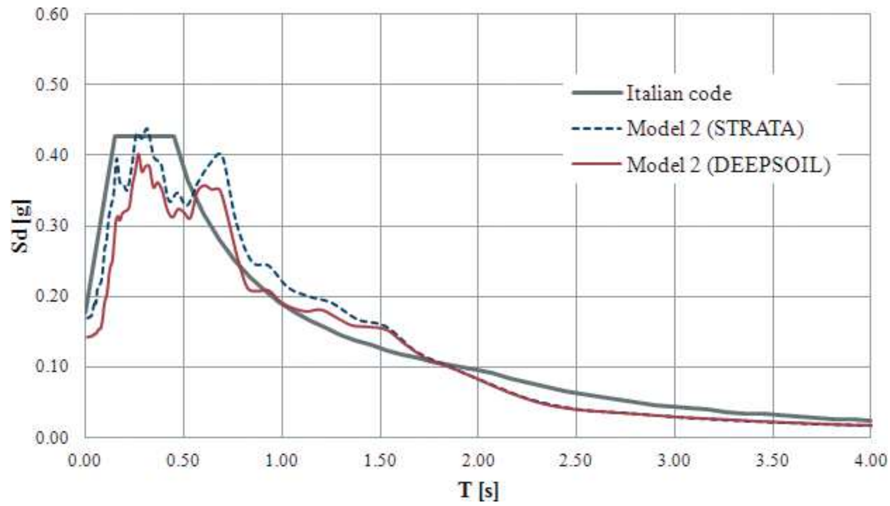

Fig. 26 – Comparison between results of two software for Model 2.
Fig. 26 – Confronto tra I risultati dei software STRATA e DEEPSOIL per il Modello 2 ed analisi lineare equivalente.

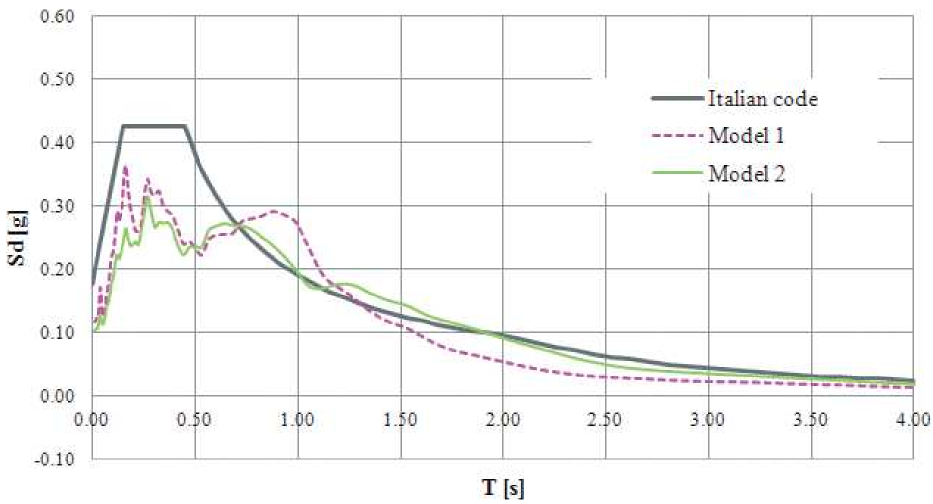

Fig. 27 – Comparison between results of two model in non linear analyses.
Fig. 27 – Confronto tra i risultati ottenuti mediante i due modelli con il software DEEPSOIL e analisi non lineare.

The influence of spectral differences on the structural response depends obviously on the dynamic characteristics of structure.

Closing remarks

Some structural diseases affect the Church of San Paolo a Ripa d'Arno in Pisa (Tuscany). These problems concern mainly the wooden structure of the roof and the presence of some cracks in the perimeter walls. After recent earthquakes in central Italy, these diseases increased to such an extent to force the authorities to close the Church.

An innovative technique to investigate foundation masonry has been specially developed. The technique consists in an instrumented coring which is able to provide quantitative information about geometry and qualitative information about masonry. Geophysical investigations, in particular ERT, seem not to work correctly, probably because the relevant characteristics of the stone masonry and the soil are rather similar, especially below groundwater level.

Since the main concern regards the seismic behaviour of the structure, a site effects analysis has been carried out. Two different models of subsoil have been used and two different types of analyses have been carried out. The analyses put into evidence the differences, often not negligible, between seismic action defined in Italian Code and that obtained by site effect analysis, and the differences due to an appropriate subsoil model. The importance of a more extended subsoil model seems relevant.

In the light of the previous considerations, beyond the suggestions of Guidelines for cultural heritage preservation, the site effects analysis seems always appropriate since the design of proper conservation measures are also a consequence of a proper definition of actions.

References

CSLP (2008) – *Azioni sismiche - Spettri-NTC ver 1.03.* Consiglio Superiore dei Lavori Pubblici.

HAMILTON E.L. (1976). *Shear-Wave Velocity Versus Depth in Marine Sediments: a Review.* Geophysics, 41, n. 5, pp. 985-996.

HASHASH Y.M.A. (2009) – *DeepSoil USER MANUAL and TUTORIAL.* English Journal. Urbana, Illinois, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign.

IDRISS M. (1990) – *Response of Soft Soil Sites during Earthquakes.* Proc. H. Bolton Seed Memorial Symposium, pp. 273-289.

IERVOLINO I., GALASSO C., COSENZA E. (2010) – *REXEL: Computer aided record selection for code-based seismic structural analysis.* Bulletin of Earthquake Engineering.

JAMIOLKOWSKI M., LANCELLOTTA R., LO PRESTI D. (1995) – *Remarks on the stiffness at small strains of six Italian clays.* In: "The 1st International Symposium, Pre-failure Deformation Characteristics of Geomaterials", Sapporo, pp. 817-836.

KOTTKE A., RATHJE E. M. (2008) – *Technical Manual for Strata.* PEER Report 2008/2010. Berkeley, California.

LAI C., ZUCCOLO E. (2012) – *SCALCONA 2.0 – Definizione di input sismici sismo-compatibili e spettro-compatibili per i comuni della Regione Toscana.* Retrieved from http://www.rete.toscana.it/sett/pta/sismica/01informazione/banchedati/input_sismici/img_input/rapporto_regione_toscana_2012_finale.pdf

MIBAC (2011) – *Linee Guida per la valutazione e riduzione del rischio sismico del patrimonio culturale allineate alle nuove Norme tecniche per le costruzioni* (Circolare n. 26 2010 - Segretariato Generale), Ministero dei beni e delle attività culturali e del turismo.

VIGGIANI C., PEPE M. (2005) – *Il sottosuolo della Torre*. In: "La Torre Restituita", Bollettino d'Arte, pp. 1-40.

VUCETIC M., DOBRY R. (1991) – *Effect of Soil Plasticity on Cyclic Response*. Journal of Geotechnical Engineering, 117, n. 1, pp. 89-107.

Aspetti geotecnici del restauro della chiesa di San Paolo in Pisa

Sommario

La Chiesa di San Paolo a Ripa d'Arno è una delle più importanti chiese medievali della città di Pisa. L'impianto dell'edificio nella sua configurazione attuale risale alla fine del XIV secolo, ma la prima pietra fu probabilmente posata molti secoli addietro. Vi sono indicazioni certe della sua esistenza già nell'anno 1032 d.C. con alcune fonti documentali che fanno risalire l'inizio della costruzione al 925 d.C. Negli ultimi decenni l'edificio ha subito diversi dissesti, probabilmente collegati ai pesanti danneggiamenti conseguenti al bombardamento del 31 agosto 1943. In particolare si ha notizia di un progressivo incremento del quadro fessurativo ed un peggioramento della condizione della struttura lignea del tetto di copertura. La situazione è peggiorata ulteriormente dopo gli eventi sismici del 2012 che hanno indotto le autorità a chiudere la chiesa al culto e ai visitatori.

Per la definizione degli interventi più appropriati da eseguire per incrementare la sicurezza dell'edificio, è stata realizzata un'approfondita indagine sia sul sottosuolo sia sull'edificio con particolare attenzione al sistema di fondazione. In una prima fase sono state utilizzate tecniche di indagine non invasive come la tomografia geoelettrica e il georadar. Entrambe le tecniche si sono mostrate poco efficaci nel fornire indicazioni di dettaglio sulla fondazione, spesso limitandosi a dare indicazione della semplice presenza di un manufatto sepolto. In particolar modo la tomografia geoelettrica si è dimostrata difficile da interpretare e poco chiara nella indicazione della geometria delle strutture interrate. Per incrementare il grado di conoscenza della struttura di fondazione è stata utilizzata una carotatrice strumentata specificamente realizzata presso il laboratorio di Ingegneria Strutturale dell'Università di Pisa. La carotatrice, che così equipaggiata sarebbe più appropriato definire sonda penetrometrica, consente di registrare una serie di grandezze, quali ad esempio la velocità di avanzamento, che sono utili alla comprensione della qualità e della geometria della struttura sepolta. La tecnica risulta poco invasiva grazie al piccolo diametro del foro praticato (35 mm).

Poiché uno degli aspetti da approfondire era la sicurezza sismica dell'edificio, è stato effettuato un approfondito studio dell'amplificazione locale del segnale sismico, avendo ritenuto inappropriato l'uso della procedura semplificata riportata nella normativa vigente. Nell'analisi, oltre ad una accurata caratterizzazione dinamica del sottosuolo fino alla profondità di circa 40 m dal piano di campagna, è stata considerata anche l'influenza della profondità del bedrock e l'influenza sui risultati di un modello costitutivo non lineare del terreno. Gli effetti sulla definizione dell'azione sismica dei due aspetti studiati sono stati significativi. Ciò sottolinea la necessità di una appropriata modellazione geometrica e meccanica del sottosuolo, in particolar modo nel caso di edifici di elevato valore storico e culturale per i quali la stima errata o la sovrastima delle azioni può portare alla progettazione di interventi non rispettosi della integrità culturale dell'edificio.