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# Bearing capacity diagnosis of Santiago church (Jerez de la Frontera, Spain)

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ABSTRACT

The aim of this research has been to determine the causes of the damages located in Santiago church (Jerez de la Frontera, Spain), such as the collapse of the pillars, the wall and vault fissures and others, which have occurred over its five hundred year existence. The most significant aspects of the investigation works performed are related to the use of combined non-destructive techniques, such as the use of georadar with ultrasounds to detect voids inside pillars and main walls, videoscope, thermograph systems, accelerator and other seismic techniques, reducing up to 20% the excavations and unnecessary bore holes in order to know the composition and heterogeneities of the elements. In addition, by using these techniques, the deformation module E – dynamic and static – at different heights in pillars, as well as the bearing capacity could be obtained, not only among the pillars, but also at the bottom, central and upper parts of each pillar. An important goal of the investigation was to determine the extent of the reinforcement works developed in 1961, checking geometric features and conservation status of concrete elements. From all the data collected, the origin of the existing damages and pathologies can be set out, and hence, a building restoration can be adequately carried out.



## 1. Description and historical background of the building

The church of Santiago in Jerez de la Frontera (Fig. 1), in late Gothic style, with Baroque and Renaissance elements, is rectangular in shape, with three aisles (the central one the highest) (Fig. 2) and covered with ribbed vaults over six decorated pillars on polygonal bases.

The chancel is located in the north-eastern zone, at the opposite side of the main entrance along the central aisle (Fig. 3), and is covered by ribbed vaults. The church has three Gothic doors, the main one located in the beginning of the central aisle, and the other two facing lateral façades.

The construction of the present church started at the end of the 15th century, following the Gothic model of the Western Andalucía area. The church was built leaning to the old chapel of la Paz, constructed in 1260, when the Muslim domination in Jerez had finished. According to the chronicles [1], the building presented many structural problems already from the end of the construction, being the most important:

- In 1663 the reinforcing works of the walls from the lateral façades were completed. The works consisted on an important section increase by bonding stone masonry, as cracks, detachments and spallings were located in walls.
- On February 24th 1695 pillars 2 and 3 of the left aisle collapsed, dragging down the six vault stretches over them, as well as the supporting wall and the flying buttresses, and important fissures appeared in pillar 4. The reconstruction began immediately after, substituting the original by thicker ones, finishing in 1699.

During 1879 to 1894 some works were done to bring the church back to the initial monumental style, that had disappeared with the reinforcing operations and reconstructions works that had taken place until that moment. The construction works were performed regarding stylistic criteria, which, in turn, produced important structural alterations. As a result of these works, two arches were opened at the chancel lateral walls; the two chancel lateral pillars were cut to leave room for the choir; the tiercerons of the vault ribs were eliminated and other minor changes were carried out. These interventions affected the structural performance of the building, as can be drawn from the following damages:

- On January 10th 1902, Rafael Esteve, the municipal architect, reported damages in the vaults and pillar 1. The restoration started in 1905 after the shoring of the area had been done. The pillar was rebuilt using stone masonry, of greater strength than the existing one in both the skin and core of the building.
- In 1928, important cracks were noticed in pillar 7, similar to those of pillar 1, so in restoration works stone ashlars were substituted by others of higher strength.

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**Fig. 1.** Main entrance door.

**Fig. 2.** View of the central aisle.

– On March 22nd 1956, during the restoration works, pillar 6, situated at the left aisle (the gospel aisle is located in the left side of an altar or church to a person looking from the nave toward the chancel) broke down and collapsed together with the vaults. An important part of the vaults detached and the section was noticeably reduced. However, four days before this had happened, the church had been examined and it did not show any sign of ruin, which shows that a compression fracture was the reason for this damage.

In 1961 restoration works began and the foundations were reinforced by placing a concrete tie beam (Fig. 4) (galleries were packed with demolition and waste materials); reconstruction of pillar 6 with the same stone ashlars and a light reinforced concrete core (Fig. 5), including arches and vaults, were built; similarly, adjacent pillars were strengthened substituting the existing core by light reinforced concrete after a vertical cut and hollowing out up to 10.50 m had been carried out. These works continued for five years.

During the 1990s, new damages affecting walls, vaults and pillars appeared. The intensity of the damages increased through time, and therefore in 2003 this research started in order to find the causes for these damages and to establish constructive solutions needed to give back the church its structural stability. The preliminary survey started in 2004, and it showed that the stability was noticeably affected, and with a high risk of collapse. As a result, in May 2005 the decision to close the church for mass was taken. The research finished in 2007 and immediately after, restoration works started under the coordination of the Culture Department of the Regional Government, under the direction of the architect Emilio Yanes Bustamante.

## 2. Location of damages

The initial works were to determine the structural stability of the church, and in order to do so, the damages of the building were identified [5,12,18]. Most relevant damages were:

– Vertical cracks in almost all pillars, except for the one reconstructed by Rafael Esteve in 1905/06 (pillar 1). Pillars 4 and 5 had severe cracks with material detachment, typical of compression fracture. These cracks appeared mainly at the three lower meters (Fig. 6), that is a clear sign of flattening by excessive load or by the smaller bearing capacity of the element.
– Cracks and material detachments in different areas of the church wall, specifically in the area of the chancel chapel, where the wall showed fractures typical of the flattening by compression. In some points, the flattening of an area had also horizontal cracks, and in other cases, bulging of the outside skin of the wall could be seen.
– A burst out of the masonry on the solid wall had appeared at the left of the entrance to the Tabernacle chapel (Fig. 7), which is a clear sign of an excessive concentration of stresses.
– The lateral wall of the right aisle (Epistle side, located in the right side of an altar or church to a person looking from the nave toward the chancel) was much deteriorated, in spite of the reconstruction work of the masonry performed on the outside.
– There was a risk of stone fragments detaching from the high areas of the façades, the flying buttresses and the pinnacles.
– Façades showed humidity patches by capillarity, reaching 1.2 m on the main façade; and more than 2 m at the back façade; at the lateral ones, the patches reached even 4 m, with green mould and vegetation.
– Lateral walls had vertical fissures, 1 cm thick. Stones had lost mass, and the mortar of the joints had detached.

## 3. Methodology used

In order to determine the foundations of the pillars, the technique of opening trial bore holes has been carried out (Fig. 8) [10,17]. Although it is a destructive test; it only affects the floor paving, as the excavation is done parallel to the pillar bases, without touching the existing masonry work.

The voids, crypts and constructions performed underground have been analyzed using the georadar technique, with a 250 MHz antenna recommended for greater depths.

To locate the damages, and to characterize the pillars and walls, the following techniques [4,5] have been used:

- Kraus–Kramer ultrasounds, with a frequency range between 100 and 1000 Hz, able to find irregularities up to 6 cm in diameter [7].
- Malá georadar, with a 800 MHz antenna, able to perform vertical profiles, with a separation in between readings of 1 mm.
- Extraction of sample pieces using a Hilti drill, to carry out physical–mechanical and chemical tests of the materials.
- An XLPRO measurement system videoscope, with a probe of 6 mm Ø and 2.5 m in length, which has shown the evidence of the composition and the existence of irregularities inside the pillars and foundation, as well as being capable of detecting crypts.

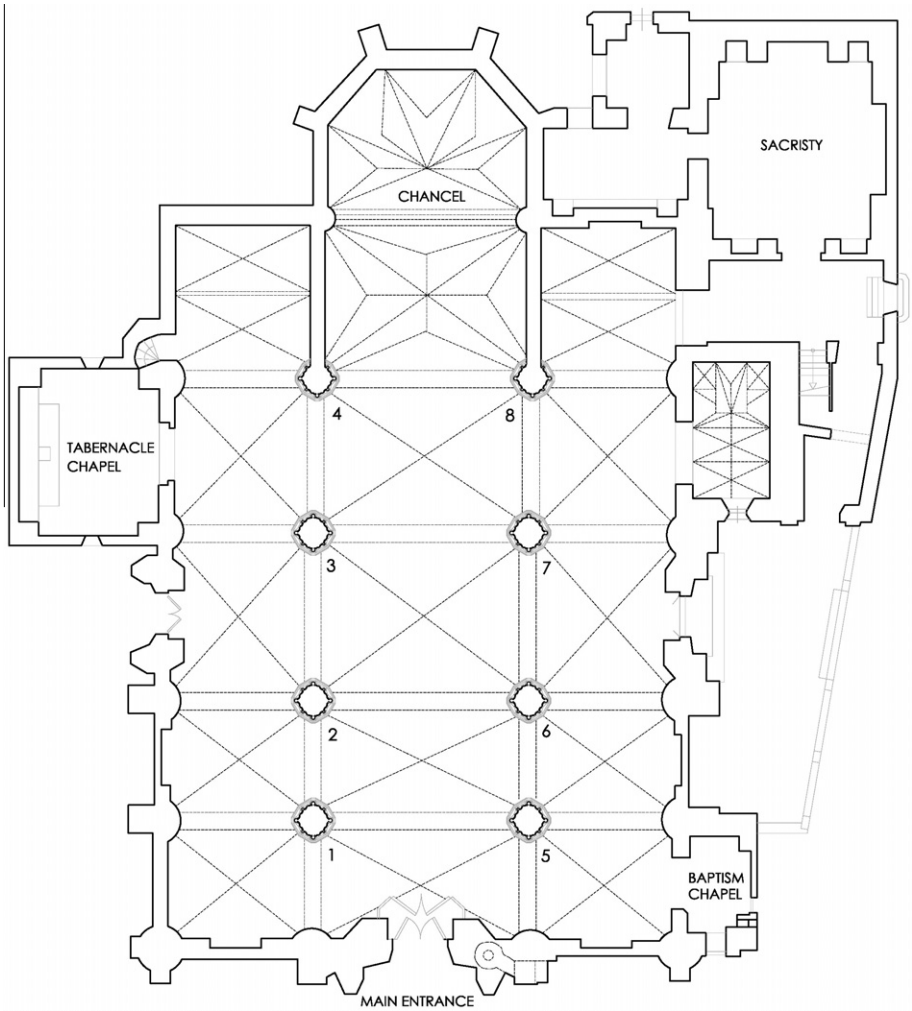


**Fig. 3.** Church plan.

To study geotechnical characteristics of the subsoil, rotary boring techniques (Tecoinsa bore machine) with continuous sample extraction have been used. In addition, on site tests have been carried out to determine the bearing capacity of the supporting soil.

A survey was carried out using accelerometers in Walls samples. The equipment used has been a triaxial accelerometer with a frequency range of 100 Hz–6.5 Hz, which was fitted to the surface being study by a metal glued base, and which registered the vibration of the structural element. The method entails the generation of seismic waves [11] by hammer blows in one of its sides, placing the accelerometer on the opposite one. Once the transmission of the ultrasound waves have been obtained, different elastic parameters can be acquired: Poisson coefficient, rigidity module, bulk modulus, and Young modulus. These elastic parameters show a dynamic character [15], so in order to obtain the module of static deformation; the relationship between Eissa and Kazi was used.

## 4. Results of the tests performed

### 4.1. Subsoil

Using georadar technique, the following irregularities have been found:

- 0.2 m Deep section: irregularities could clearly be seen in a elongated shape, in between the pillars of the left, corresponding to the bracing performed in the 1961 intervention; one of them, the closest to the apse, is broader and reinforced.
- In two of the chapels studied, the presence of crypts was found.
- Several irregularities were found in the central aisle; next to pillars 7 and 8 these irregularities could be related with the foundation carried out then.
- 0.5 m Deep section: most of the irregularities are related to the ones previously described. The different composition of the bracings next to the apses at the right and left aisles can be seen. The staircase leading to the crypt was revealed at that height level.
- 0.8 m Deep section: some of the previously described irregularities were still present, and also the reinforced bracings were identified.
- 1.2 m Deep section: very few irregularities were observed. The only remaining irregularities which could be identified were the burials in both lateral chapels, the descent to the crypt in the main altar and two bracings at the font of the lateral aisles.

### 4.2. Bearing soil stratigraphy

The geology of the location [14,16] was established from the two bore tests performed as it corresponds to:

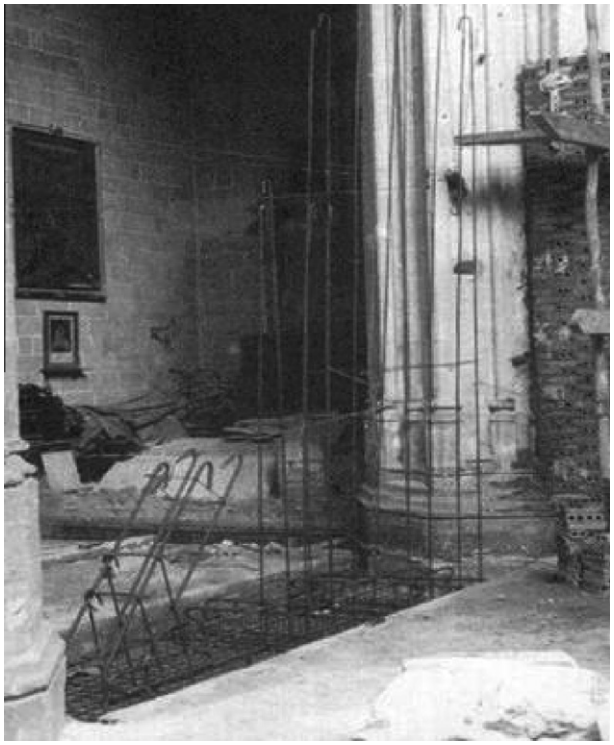

**Fig. 4.** Foundation reinforcement with a tie beam (1961).

**Fig. 5.** Pillar 6 situation after the collapse of one of the sides, with the remains of the stretches fallen (1961 intervention).

- Level I: packing formed by stone embankment blocks and construction waste rubble. The width of this coat varies from 1.2 to 2.0 m in depth.
- Level II: formed by silty sand of a soft brownish colour with green streak and presence of carbonated nodules. It is a non plastic soil with $N$ (STP) = 28 values, and a 1.7–2.3 N/mm$^2$ admissible load. The thickness of this layer is 9 m, reaching a variable depth of 10–11.5 m
- Level III: corresponds to a clay sand with bioclastic and limestone debris and a medium–high compactness ($N$ values in between 30 and 35), and a bearing capacity of 2.5–3 N/mm$^2$.

Foundations are set over level II, a soil with a medium compactness and strength, and unstrainable by swell or retraction phenomena. Therefore, the main settlement of the building would have been produced during the construction works, since there is no evi-

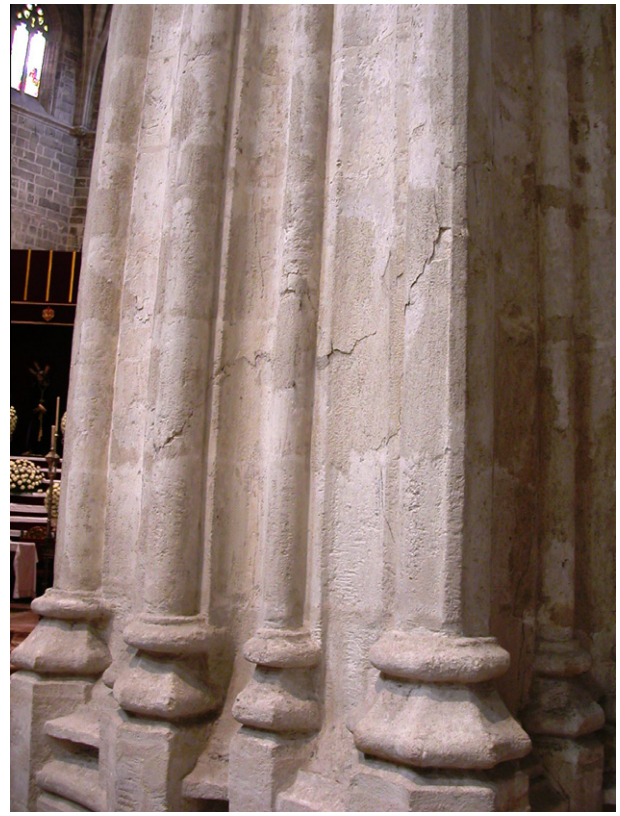

**Fig. 6.** Pillar 8 cracked by compression stresses.

**Fig. 7.** Wall cracked by compression stresses.

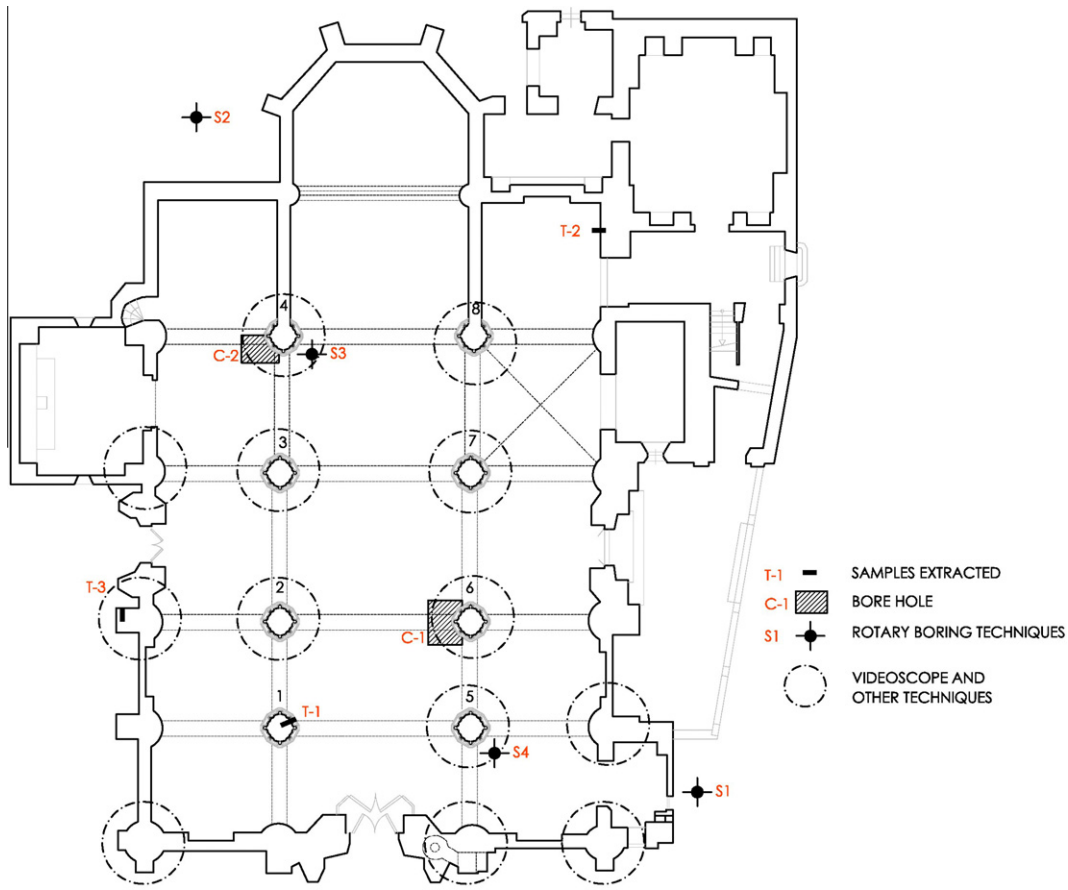


**Fig. 8.** Church plan showing the different tests performed.

dence of the bearing soil to have caused the damages produced throughout the history of this building.

### 4.3. Foundation characterization

The data obtained by the bore hole tests made at the perimeter of pillars 4 and 6 indicate that the foundation of the first one is composed by masonry of bioclastic limestone and ceramic fragments together with lime mortar. The overweight of the pillar regarding its perimeter is of 20–30 cm and the depth is 2 m. It stands on the level II silty sand stratum. The discovery of bracing beams made of reinforced concrete, with a brick formwork corresponding to the 1961 intervention, is to be highlighted.

Foundation of pillar 6, built that same year to substitute the pillar collapsed in 1956, is set up by a solid concrete footing 123 cm thick over which a reinforced concrete footing of 55 cm thickness is placed, tied with a reinforced concrete tie beam, 70 cm wide and of side 140 cm. Bearing soil is also level II silty sand, as in pillar 4.

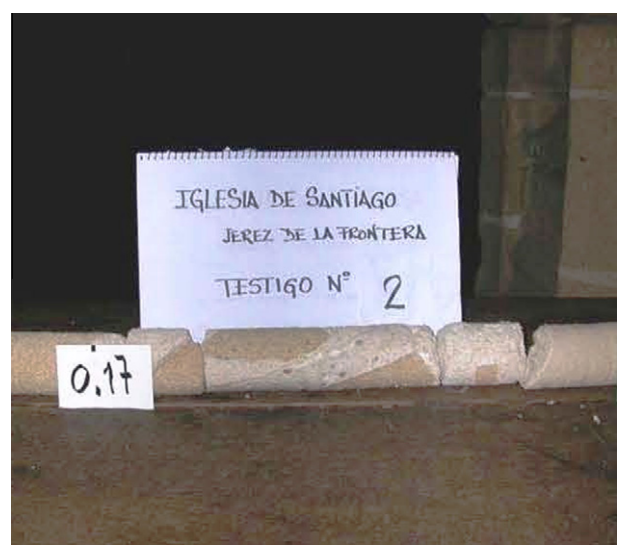


**Fig. 9.** Test showing composition of right lateral wall.

### 4.4. Wall composition

Georadar and videoscope techniques have also been used. From the study carried out, the following data can be highlighted: heterogeneity in the composition, presence of voids, and section change between upper and lower side due to stresses (1663 intervention). For example, the right lateral wall, at a height of 155 cm, measured from the interior floor paving, has a double wall structure, preseting the following composition looking from the inside to the front façade: bioclastic limestone ashlar joined with lime mortar (45 cm thick), solid brick masonry joined with lime mortar (70 cm), stone and ceramic piece agglomerate with low compactness lime mortar (Fig. 9) (65 cm) and stone ashlar masonry of bioclastic limestone joined with lime mortar (street façade). As a consequence of the regrowth performed in 1663 the whole section is of 237 cm.

However, the composition of this wall at height 620 cm is different. The wall is of double leaf: calcarenite stone ashlars joined with lime mortar (30 cm on each side) and an agglomerate core

of stone and ceramic debris joined with lime, a disperse unit of low compactness (65 cm). Total thickness is 125 cm; hence, the thickness increase of the lower part was of 112–115 cm, corresponding to stone ashlars and solid brick masonry.

The interior walls have the same composition in both the lower and the upper parts. It is composed by double leaf: stone ashlars of thickness 35 cm and a disperse aggregate of calcarenite stone and lime (60 cm). Total thickness is 130 cm core.

The façade wall of the left side has the same composition with thicknesses of 26 cm for the first leaf (stone ashlars), 64 cm for the disperse aggregate and 25 cm the calcarenite ashlars masonry. Total thickness is 115 cm.

The wall located between the left aisle (the gospel aisle) and the baptismal chapel has exactly the same composition: calcarenite stone ashlars masonry with lime mortar (25 cm); agglomerate lime core, stone and ceramic pieces, disperse unit with voids (60 cm); the other leaf is made of stone ashlars (33 cm) which adds up to a total of 118 cm, at 470 cm in height.

The inspection performed at the same wall at a height of 155 cm, shows the following composition: calcarenite stone ashlars joined with lime mortar (33 cm), ashlars masonry (equal to the one before, 22 cm), disperse conglomerate (15 cm), brick packing with lime mortar (9 cm) and stone ashlars masonry (30 cm). The thickness is 209 cm.

### 4.5. Composition of pillars

The following data has been obtained using videoscope, georadar and drill techniques:

- Pillar 1. Rebuilt in 1906 by the architect Esteve. It is formed by calcarenite stone ashlars of greater compacity and greater resistance than that of the other pillars. The core is also made by stone ashlars, and as a consequence, the pillar is totally formed by stone.
- Pillar 2. Calcarenite stone ashlars masonry joined by lime mortar of thickness 20 cm covering the core made of stone pieces and lime. The result product is of low compactness. This pillar fell down and was replaced (1695–1699).
- Pillar 4. Skin of calcarenite stone ashlars (33 cm), and core of lime masonry aggregate with a low compactness and dispersed. It is the original one.
- Pillar 5. Constituted by calcarenite stone ashlars in the outer leaf and an agglomerate of stone pieces and lime in the core, where voids of almost 37 cm were found. It is the original one (Fig. 10).


**Fig. 10.** Void found with the videoscope inside 5.

- Pillar 6. Formed by calcarenite stone ashlars 18 cm in the outer leaf, and an aggregate core with concrete additions. Voids can be seen. It fell down and was replaced in 1956.
- Pillar 7. Formed by calcarenite stone with a lime mortar covering (33 cm) and a aggregate and concrete core with voids (repaired in 1928).
- Pillar 8. Formed by a calcarenite stone skin and a reinforced concrete core (core packing in 1956).

To identify the irregularities in the pillars, georadar techniques have been used in vertical profiles. The results obtained have been divided in three categories according to their nature:

*Type 1*: Small irregularities associated to small material contrasts or bulk discontinuities, as the ones that can be found in the ashlars joints.
*Type 2*: Larger irregularities associated to voids or detachment of mortar in the joints and fissures.
*Type 3*: Irregularities associated to fractures or noticeable discontinuities inside the pillars.

Type 1 irregularities appear in almost all pillars analyzed, whereas they do not appear in columns 1 to 5 from height 3.0 approximately. In general, these irregularities are present at a depth which does not exceed 50 cm from the surface. As an exception, pillar 7 can be highlighted, where a void has been located in the core at a depth of almost 1 meter.

Type 2 irregularities, associated to defects in the pointing (open limits) among the ashlars, appear in the vast majority of surfaces studied. As an exception, pillar 7 can be seen, where a void has been located in the core at a depth of one meter.

Type 3 irregularities are characterized by continuous reflectors, generally placed vertically 'o' slightly inclined, associated to fractures or discontinuities in the materials. When they appear in disorder, they are considered fractures, sometimes large in size, forming high reflectivity areas. They can also appear in an orderly way, generally in a vertical layout. In this case, they are considered as a contact or discontinuity between the different material types,

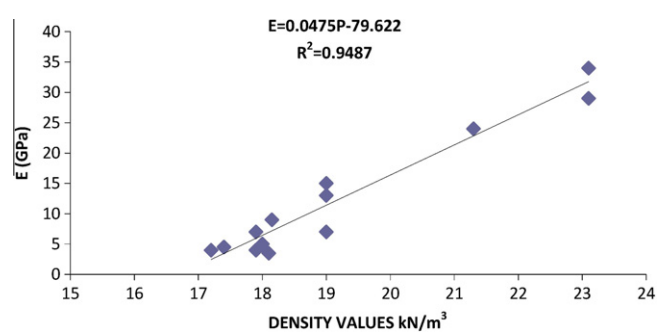


**Fig. 11.** Correlation between density values obtained in laboratory and static deformation calculated.

**Table 1**
Values obtained from testing calcarenite samples.

| Calcarenite stone properties tested | Original stone | Replaced stone |
|---|---|---|
| Bulk density | 17.1–18.2 kN/m$^3$ | 19.0–22.9 kN/m$^3$ |
| Porosity | 23.5–25.4% | 16.6–20.4% |
| Compressive strength in dry state | 1.7–2.4 N/mm$^2$ | 2.8–3.7 N/mm$^2$ |
| Compressive strength in water saturated state | 1.2–1.81 N/mm$^2$ | 2.5–3.4 N/mm$^2$ |

as in the case of pillars 6, 7 and 8 where concrete was introduced in the core.

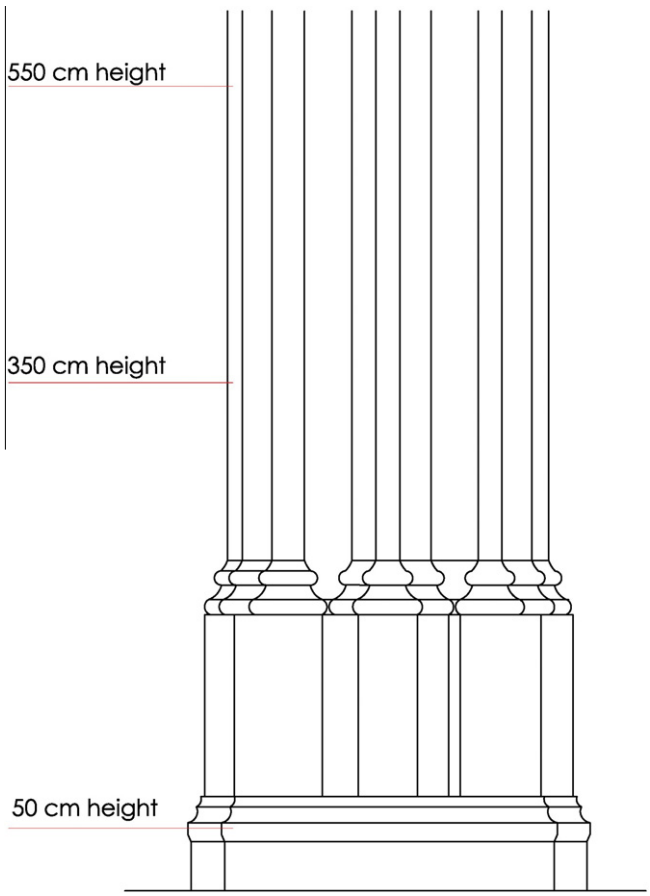


**Fig. 12.** Measuring points.

In order to determine the material static formation forming the pillars, an ultrasound technique has been used. The collecting data campaign was originally set up for interpretation of this data. For this reason, it is necessary to perform an inspection hatch or manhole through the complete section of the pillars with a maximum diameter of 2.17 m. However, when collecting the data, it came out that the transmission of ultrasound pulses generated was not effective enough to be detected by the transducer placed at the other edge of the pillar. This situation (total damping of the ultrasound wave front) cannot be attributed to high fading of the material studied due to its porosity. The most possible cause is the presence of discontinuities (cracking, fractures, and voids as the ones found with the endoscope) in the path covered by the ultrasound wave.

With these precedents, ultrasound measurements were performed on the sample test pieces taken from the calcarenite ashlars stone forming the skin, from the core mortar of pillars 2, 3, 4 and 5, and from concrete of pillars 6, 7 and 8. The values obtained for the elastic deformation modulus are more precise than the "on site" values obtained, since the exact length of the path and material density is known.

Parameters obtained by this method are of dynamic character. This is due to the stresses suffered by the material when the ultrasound wave passes in a infinitesimal time period. Therefore, the deformations produced by these stresses do not coincide with the ones obtained by the static compression tests in labs. Several studies can be found on the existing relations between elastic and dynamic parameters of materials: Cheng and Johnston, 1981 [2]; Eissa and Kazi (1988) [3]; Ciccotti and Mulargia (2004) [13]. In the study performed here, to obtain the static deformation module, the relation proposed by Eissa and Kazi has been used. This one is obtained from the compilation and comparison of numerous data measured in different types of stone. This relation can be expressed as:

$$\log(E_{\text{static}}) = 0.02 + 0.77\log(\rho E_{\text{dynamic}})$$

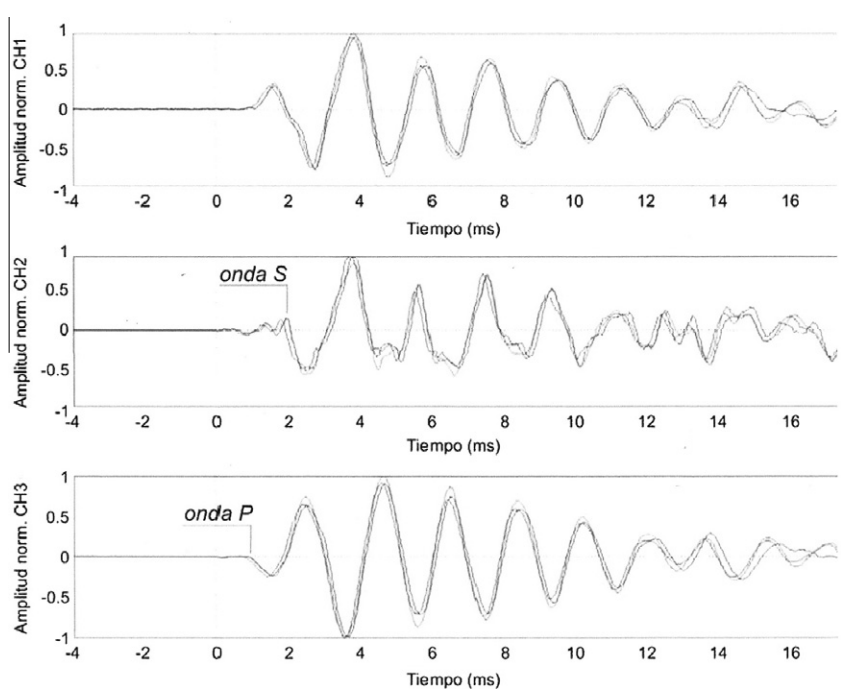


**Fig. 13.** Example of accelerograms processed from the recordings. Top: horizontal component (*X*) of the acceleration registered in the sensor. Centre: vertical component (*Y*) of the acceleration registered in the sensor. Bottom: compressive component (along the energy propagation direction) of the acceleration registered in the sensor. Ranges are standardized.

where $E_{static}$ is the static Young module (GPa) and $E_{dynamic}$ is the dynamic Young module (GPa).The following graph (Fig. 11) shows the existing correlation between the density values obtained in the laboratory and the static deformation module. As can be seen, both parameters have a 0.95 correlation degree.

The static deformation modules obtained are distributed into three ranges:

(a) $E < 5$ GPa. Corresponds to greatly weathered coarse grain calcarenite stone samples and to mortar or original material of the core.
(b) 5 GPa $< E <$ 20 GPa. Corresponds to compact calcarenite stone samples used in the replacements.
(c) $E > 20$GPa. Corresponds to concrete samples of the core parking from the pillars.

### 4.6. Materials characterization

The bore tests performed allowed us to have samples of the different materials forming the masonry. These samples have been subject to different analysis and test in the lab [4,8]. Most relevant data obtained are:

– Ashlars blocks are made of a light yellow bioclastic calcarenite stone, with a reduced primary porosity and an alteration type II degree (Larger irregularities associated to voids or detachment of mortar in the joints and fissures). Within this classification, there are two varieties: the first one corresponds to the original construction, and the second one is from the same calcarenite stone but with greater strength and compactness (Table 1).
– Tests of tie beam concrete:
• Bulk density: between 21.9 and 22.4 kN/m$^3$.
• Porosity: between 10.4 and 11.6%.
• Compressive strength: between 12.8 and 19.5 N/mm$^2$.
– Core packing concrete from the pillars:
• Bulk density: between 21.1 and 21.6 kN/m$^3$.
• Porosity: between 13.8 and 14.2%.
• Compressive strength: between 7.8 and 9.6 N/mm$^2$.

**Table 2**
Values of the dynamic and static modules for the two measurements performed. Last column shows the corresponding percentages in relation to the maximum value obtained.

| Pillar and height (m) | Dynamic $E$ (GPa) | Static $E$ (GPa) | Value obtained percentage (%) |
|---|---|---|---|
| 1–0.5 | 13.1–14.0 | 12.4–13.1 | 99.2 |
| 1–3.5 | 13.3–13.5 | 12.6–12.7 | 99.2 |
| 1–5.5 | 13.6–13.7 | 12.8–12.9 | 100 |
| 2–0.5 | 6.2–6.6 | 6.7–7.1 | 65 |
| 2–3.5 | 9.2–9.4 | 9.1–9.2 | 71.2 |
| 2–5.5 | 10.9–12.4 | 12.4–11.4 | 100 |
| 3–0.5 | 8.5–7.2 | 8.6–7.5 | 78.5 |
| 3–3.5 | 9.8–11.7 | 9.5–11.0 | 100 |
| 3–5.5 | 10.1–10.1 | 9.7–9.8 | 95.6 |
| 4–0.5 | 5.2–6.2 | 5.9–6.7 | 71.6 |
| 4–3.5 | 7.4–6.8 | 7.7–7.2 | 85.2 |
| 4–5.5 | 9.3–9.6 | 9.2–8.5 | 100 |
| 5–0.5 | 2.7–3.1 | 3.5–3.9 | 74 |
| 5–3.5 | 3.3–3.7 | 4.2–4.5 | 86 |
| 5–5.5 | 4.2–4.1 | 5.0–4.9 | 100 |
| 6–0.5 | 6.7–5.9 | 7.1–6.4 | 79.8 |
| 6–3.5 | 7.9–8.6 | 8.1–8.7 | 100 |
| 6–5.5 | 7.9–8.2 | 8.1–8.3 | 97.6 |
| 7–0.5 | 8.1–8.0 | 8.9–8.8 | 98.9 |
| 7–3.5 | 8.4–8.1 | 9.1–8.9 | 100 |
| 7–5.5 | 7.4–7.7 | 8.2–8.5 | 93.3 |
| 8–0.5 | 12.4–12.3 | 12.3–12.2 | 100 |
| 8–3.5 | 11.6–10.7 | 11.7–11.0 | 93.4 |
| 8–5.5 | 10.3–11.1 | 10.7–11.3 | 90.2 |

## 5. Obtaining the Young module by accelerometer

This seismic method consists on determining the time P and S waves (Fig. 13) take to cover the pillar section by using accelerometric recordings with a triaxial accelerometer, with a frequency range of 100 Hz to 6.5 kHz. Time measurements of the wave passing have been made at 50, 350 and 550 cm height (Fig. 12), and two measurement for each one of them (direction EW and NS). The distance between the translators has been obtained by digital cartography. From the speed values obtained, Poisson coefficient, rigidity module, volumetric compressibility module and Young module were then calculated (Table 2).

For the density of the pillars, a mean value of 18.0 kN/m$^3$ has been considered (calcarenite stone skin and mortar core) and for pillar 1, constructed with calcarenite stone ashlars both the skin and the core, 19 kN/m$^3$. In order to obtain the static deformation module, Eissa and Kazi (1988) formula has been applied.

## 6. Conclusions

This research has been developed in order to determine the causes of damages and structural injuries that Santiago church has had since its construction, in the beginning of the 17th century, and which caused the collapse of many structural elements, until complete ruin and closure of the church in 2005; during this time it has undergone research and rehabilitation works.

The main goal of the research was to characterize the structural elements and soil, by using traditional and innovative techniques, as well as destructive and non-destructive tests.

The study of the soil with rotary boring techniques and georadar showed that the foundations of walls and pillars of the church are placed over a compact layer of unstrainable silty sands, confirming that the damages that had appeared in these three centuries were not caused by failures in foundations or soil. Inspections by georadar allowed us to further demonstrate the existence of tie beams between pillars, and some crypts and burials.

To study the foundations of the walls and pillars, bore hole tests and inclined rotary techniques bores were performed. An innovation in this study is that the slanted drilling allowed knowing the thickness of foundations, extracting samples of materials, and detecting homogeneity. This technique has also unveiled concrete foundation from 1956, when pillars 4 and 6 were reconstructed.

In order to study walls and pillars videoscope, georadar, ultrasonic techniques, and samples extractions were performed, detecting a very poor quality of materials used, and very heterogeneous compositions, as a result of the multiple reconstruction works done as well as the injuries, fractures and hollows suffered.

Original pillars 4 and 5, are constructed with an exterior envelope ring of 20 cm calcarenite stone ashlars. Inside it, a heterogeneous mixture of lime and stone pieces were inserted. Pillars 2 and 3 were also rebuilt with a similar composition, and practically identical to the original one. Reconstructions of the different pillars throughout time have been very diverse, and various configurations have been found: for example, pillar 1 was reconstructed completely with stone ashlars and pillars 6, 7 and 8 were filled with concrete.

Nevertheless, a high heterogeneity in the composition of every pillar has been identified. This causes important changes in the bearing capacity, which is aggravated at the basis due to the intense ascending by capillarity humidity.

Calcarenite stone samples were obtained from the pillar envelope to find the dynamic deformation module, and having this data, we calculated static deformation module and established the correlation with density values obtained in the laboratory. Due to the difficulty of obtaining the load bearing capacity of

pillars and walls, as they are constructed with a thin calcarenite stone envelope of a low bearing capacity (1.2–1.8 N/mm$^2$) filled with a poor and not compacted material of lime and stone pieces, accelerometer techniques had to be used. These techniques, measured the time spent in the wave transmission at different heights and positions in each pillar, allowing us to determine rigidity module, volumetric compression module and Young module.

From the data obtained and the analysis performed, the following conclusions can be drawn:

– Measurements obtained at height +0.5 m show lower values than those at heights +3.5 and +5.5 m in pillars 2, 3, 4, 5, and 6. The values of pillar 1 are very similar at the three levels. In pillars 7 and 8, they are greater than +0.5 and +3.5 m high and smaller than at height +5.5 m.
– Taking pillar 1 as reference – the one with the highest modules – , the mean values obtained in the three heights of the other pillars would be from lower to higher:
• Pillar 5, 34.4% of the value obtained in pillar 1; this pillar presents important voids in the core and a low quality of the materials used.
• Pillar 4 (57.8%); pillar 6 (60.9%); pillar 7 (68%); pillar 2 (72.7%); pillar 3 (73.4 %); pillar 8 (89.8 %).

Pillars 5 and 4, the ones with a lower bearing capacity, are the original ones, with no intervention performed. Pillar 6, which fell down and was replaced in 1956, has lower strength because of the voids present in concrete, – similarly to what happens with pillar 7, repaired in 1928. Pillars 2 and 3, replaced in 1699 when the original ones collapsed, and using calcarenite stone of higher strength, show better values. Finally, pillar 8 has the greatest percentage of all, because the core was replaced with reinforced concrete.

– Within these pillars, important differences can be seen at the three levels. The most uniform one is pillar 1, and to a lesser extent, pillar 8, since its maximum value appear in height 0.5 m and the minimum values (90.2%) at 5.5 m height. The reason for this can be found in the placing of Portland cement concrete used to substitute the mortar core, as it is more compact due to the facility to be placed on site at the lower part and more difficult to place at the upper one. The greater differences appear at 3.5 m high (71.2%), followed by the 0.5 m values of pillars 3 (78.5%); 4 (71.6%); 5 (74%) and 6 (79.8%).
– These differences in the bearing capacity make some pillars to be more overloaded than others. Thus, together with the low bearing capacity explains the consecutive falls and the important fissures pillars 4 and 5 presented when this study was started.
– The question of whether the same bearing capacity is maintained in the lower part of the pillars (heights +7 m and +8.5 m) still remained. To solve it, a scaffold was installed to perform an accelerometric test in pillar 4 – as it showed the second to last smaller values – and in pillar 8 (filled with reinforced concrete), as it had showed 90% of the value obtained at a height of 0.5 m.

Results were:

| Pillar and height (m) | Dynamic *E* (GPa) | Static *E* (GPa) | % Static *E* maximum value |
|---|---|---|---|
| 4–7 m | 10.3–10.5 | 9.9–10.1 | 112.4 |
| 4–8.5 m | 11.2–10.5 | 10.6–10.1 | 116.3 |
| 8–7 m | 8.7–8.9 | 8.7–8.9 | 80 |
| 8–8.5 m | 10.0–8.8 | 9.7–8.8 | 84 |

As can be seen in pillar 4, the greater the bearing capacity, the greater the height, with 112% at 7 m height and 116% at 8.5 m height from values obtained under the ground level. The opposite occurs in pillar 8, where at 7 and 8.5 m heights, that is, values obtained are 80% and 84% respectively from the value at 5.5 m. This shows that at that height, no reinforced concrete packing was done, although it was definitely performed at the lower height.

The differences obtained in the bearing capacity in one pillar, together with its low load bearing capacity, are the causes of the damages and noticeable cracks found in this church in Jerez de la Frontera (Cadiz, Spain), especially in pillar numbers 4 and 5, when this study was started.

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