Seismic response of a small-scale masonry groin vault: experimental investigation by performing quasi-static and shake table tests

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Abstract
The seismic safety assessment of 3D complex structural elements of historic buildings, such as groin and rib vaults, is a challenge and experimental tests can provide relevant data for this purpose. This paper presents the results of two experimental campaigns about a reduced scale 3D printed vault characterized by asymmetric boundary conditions. The specimen adopted the square plan geometry of groin vault and was made of polymeric bricks with radial joints, orthogonally woven at the barrel intersection. The seismic behaviour of the vault was studied through physical modelling: quasi-static using actuators and dynamic using a shake-table. In particular, an incremental dynamic analysis up to the collapse of the vault was carried out. The results obtained experimentally were analysed in terms of damage, collapse mechanisms, ultimate displacements, and response spectrum-based analysis.

Keywords Groin vault · Displacement-based assessment · Experimental tests · Quasi-static tests · Seismic response · Shake table tests

1 Introduction
Masonry cross vaults represent one of the most important structural typologies within European cultural heritage buildings. Mostly developed during the Roman Empire and in the period between the High Middle Ages and Renaissance until nowadays, groin and rib vaults reached a level of beauty and construction technique that still amazes the modern observer. For centuries, experiences and instructions had been secretly transmitted between ancient masons as rules of thumb, which have guaranteed their long-lasting history (Gaetani et al. 2016). However, being conceived to withstand only gravitational loads, masonry vaults are threatened by seismic events (Doglioni, Moretti, and Petrini 1994; Regione Toscana 2003; National Civil Protection Service 2013; Podestà et al. 2010).
In this regard, considering the limited research in the field, the present paper aims at investigating the seismic behaviour of masonry groin vaults subjected to shear displacement of the abutments (Fig. 1). The need of this research is due to the fact that, in most cases, the groin vaults are capable of sustaining significantly larger vertical displacements than horizontal displacements before collapse occurs (Mcinerney and Dejong 2015). This particular type of cross vault (groin vault) is theoretically the geometrical result of the intersection at a right angle of two semi-circular barrel vaults, characterised by the lack of diagonal ribs. Groin vaults are very often used in the covering of churches’ naves, palaces’ rooms, and porticos, especially in the Italian territory. Thus, the present research work has three main aims:

- Evaluation of the damage and collapse mechanism of the tested specimen under different loading conditions;
- Depiction of the resistance to shear displacement of the abutments, under quasi-static and dynamic actions, analysing the same test specimen;
- Studying the influence of the frequency content of the ground-motion on the same test specimen, considering a registered past Italian earthquake and one expected by the Italian code;

The first aim is to evaluate the three-dimensional damage and collapse mechanism of masonry groin vaults subjected to horizontal shear forces caused by the seismic action. In-plane shear behaviour is a common type of failure that occurs during an earthquake as it was widely observed during post-earthquake surveys (Arcidiacono et al. 2016; Carfagnini et al. 2018; Borri et al. 2002).

The second aim of this research work is to investigate the shear capacity of groin vaults in terms of forces and displacement, using two different experimental approaches. The first approach is static and corresponds to the quasi-static tests performed by Rossi et al. (2015) at the University of Genoa, where monotonic horizontal displacements were applied at the abutments. The second approach is dynamic and is based on an extensive shake table tests campaign performed in the National Laboratory for Civil Engineering, Lisbon, Portugal (LNEC) within the context of the project “Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe” (SERA). The crucial point of this research
is that the same test specimen, a reduced scale (1:5) groin vault made of 3D printed polymeric blocks, has been used in the two approaches. Experiments where a same test specimen is used for both quasi-static and dynamic testing are rare. For example, Calderini et al. (2015) studied both statically and dynamically a scaled arch-pier system reinforced with tie-rods, but no curved structures such as vaults were studied through both static and dynamic campaigns. The main results of Calderini et al. (2015) showed that, in general, the force and displacement capacity of the tested model is lower than the ones predicted by theoretical analysis; this may be attributed to the deformability of the system in both the elastic and inelastic range and the presence of imperfections in the contacts between blocks. On the other hand, shake table tests proved the importance of considering displacement capacity for the seismic assessment of ancient masonry structures and, even more, in the design of strengthening techniques. From these considerations, it was possible to compare in terms of displacements the two approaches, using the response spectra for a Single-Degree-Of-Freedom (SDOF) to assess the performance of the structure based on linear kinematic analysis (Cattari et al. 2008; Lagomarsino and Resemini 2009; D’Ayala 2005; D’Ayala and Tomasoni 2011; Mousavian and Casapulla 2020). Even if such study led to important conclusions, the results are only valid for a 2D structure. The research presented in this paper aims at extending the study to a three-dimensional behaviour. The comparison between static and shake table tests is fundamental because, in the most recent codes and guidelines, nonlinear static analysis is still the most used type of analysis to assess the behaviour of the masonry structures (static approach) (Modena 2020; Bianchini et al. 2019). Still, nonlinear dynamic analysis, despite the heavier computational effort, may lead to more realistic results.

Finally, the third aim of this research work is related to the influence of two different types of ground motion on the performance of the test specimen. Namely, the recorded ground motion which hit Emilia region (Italy) in 2012, and an artificial accelerogram based on the code spectrum were selected. This is a region where many monumental buildings present groin vaults as proved by post-earthquake damage and surveys.

1.1 Shear failure investigation

Historically, groin and rib vaults showed advantages which made them more desirable than other vaults. Advantages include the structural stability, the simplicity, and the flexibility to provide natural light to the interior places, and the aesthetics. At the same time, a groin or a rib vault requires less quantity of material, compared to other kinds of vaults (e.g. about half of the material of a barrel vault or two-thirds less material than a pavilion vault) and it is likely one of the most visually appealing horizontal elements that allow transferring gravitational loading to piers in monumental buildings (Huerta 2004; Cangi 2012).

Despite the favourable behaviour of cross vaults under vertical static loads, the in-plane horizontal shear distortion is a damage mechanism recurrently observed during post-earthquake surveys. This kind of mechanism is rather unexplored, even if it is particularly common in single-nave churches (Fig. 1b) characterised by a large difference in stiffness between the nave and the façade and/or transept, and in three-nave churches characterised by a large difference in stiffness between the lateral wall and the colonnade (Fig. 1c) (Rossi et al. 2015; Milani et al. 2016; De Matteis et al. 2019). For this research, Fig. 1c represents the chosen condition.

In rather common cases, the lower stiffness of the central nave’s colonnade, compared to the external walls, can lead to differential displacements along the longitudinal direction.
and, consequently, to the development of shear damage in the horizontal structural elements. This failure is mainly identified by diagonal cracks and the test specimen used in the experimental campaign was designed to reproduce this failure using appropriate geometrical details and boundary conditions.

1.2 Experimental campaigns on masonry cross vaults

Despite the fact that the present topic interests a significant amount of real cases across the world, only a few researchers worked on masonry cross vaults (groin and ribbed ones). Bertolesi et al. (2019) provided an overview about the academic interest on this topic, highlighting that less than 40 scientific papers deal with masonry cross vaults in the period from 1960 to 2018. Moreover, in the last four years, they account for less than 1% of all the documents dealing with historic masonry structures, as found by the authors of this paper. In addition, very few researchers dealt with full experimental campaigns on shear behaviour. Table 1 summarises experimental studies developed using different materials and types of tests. Tests with specimens built at reduced scale represent an interesting approach to study the structural behaviour of masonry cross vaults, mainly the collapse mechanism, and, consequently, have been recurrently used.

Mark and O’Neill (1973) presented the first experimental study on the subject, pointing out that the major in-plane vault forces are directed to its supports, validating ancient treatises and semi-empirical rules like the ones from Abraham (1934). Among other examples, Bati et al. (2002) carried out a complete study in which different load conditions, symmetric and asymmetric boundary conditions, and strengthening techniques (mainly ties and Glass Fibre Reinforced Polymer externally bonded reinforcement) were evaluated. Theodossopoulos et al. (2002, 2003) applied an increasing horizontal displacement at the abutments up to failure, and highlighting the importance of the stability of the supporting piers and the abutments. This study has been crucial for the definition of the setup of this research, since it stressed the fact that, under service loads, the groin vault showed a high reserve in strength, developing an asymmetric deformation pattern. Such behaviour is due to the difference between the stiffness of the wall and the nave arch. Williams et al. (2012) applied a unidirectional horizontal base shaking to a reduced scale model, representing the first shake table tests performed on a cross vault. Foti et al. (2015, 2018) introduced the important aspect of the masonry bond (or unit arrangement) of the vault. They studied parallel and radial masonry bond from the experimental point of view, keeping the same geometry and material of the test specimen. Moreover, they applied displacement control tests, namely in the vertical, horizontal, and diagonal directions. In the case of diagonal failure, the experimental laboratory model showed a premature collapse due to the presence of local hinges at some edge blocks’ location.

The specimen tested by Rossi et al. (2020) consists of the first shake table test performed on a 1:1 scale model. Even though the target failure mechanism was the one related to the shear failure, the vault collapsed at 0.4 g, due to the activation of an out-of-plane mechanism that led to the development of the typical four-hinge mechanism. Carfagnini et al. (2018) and Baraccani et al. (2020) performed also important works on this topic. They tested a 1:4 scaled specimen of a Gothic cross vault, subjected to shear deformation at two of its springings. It was shown that the initiation of cracks appeared perpendicularly to the diagonal ribs. It was also identified that the failure occurs at about 2.7% of the longitudinal span.
Table 1  Experimental works carried out in masonry vaults, from the 1970s until the most recent works

| References                        | Scale   | Dimensions       | Type of action   | Type of tests                           | Material                       |
|-----------------------------------|---------|------------------|------------------|-----------------------------------------|--------------------------------|
| Mark et al. (1973)                | 1:50    | 0.1 × 0.2 m²     | Static           | Dead load (Photoelastic technique)       | Epoxy resin                    |
| Ceradini (1996)                   | 1:1     | 9.6 × 10.6 m²    | Quasi-static     | Vertical settlement of the supports     | Bricks                         |
| Faccio et al. (1999)              | 1:1     | 7.36 × 7.36 m²   | Quasi-static     | Monotonic point load at the key of the vault | Bricks                         |
| Bati et al. (2002)                | 1:3     | 2.3 × 2.3 m²     | Quasi-static     | Monotonic point load at the key of the vault | Bricks 0.5 cm lime joints       |
| Theodossopoulos et al. (2002, 2003) | 1:4   | 1.5 × 1.84 m²    | Quasi-static     | Horizontal displacement                  | Wood                           |
| Miltiadou-Fezans (2008)           |         | 1.275 × 0.945 m² | Dynamic          | Shaking table tests (Athens earthquake) | Bricks                         |
| Williams et al. (2012)            | 1:25    | 0.89 × 0.896 m²  | Dynamic          | Shaking table tests (sinusoidal pulse)  | Plaster and sand               |
| De Matteis and Mazzolani (2012)   | 1:5.5   | ~2.0 × 1.1 m²    | Dynamic          | Shaking table tests (Calitri earthquake) | Bricks                         |
| Shapiro (2012)                    | ~1:10   | 278 × 278 mm²    | Quasi-static     | Monotonic point load Spreading supports Tilting tests | Plaster and powder material   |
| Rossi et al. (2016), Milani et al. (2016) | 1:5 | 0.65 × 0.65 m² | Quasi-static     | Horizontal shear displacement Tilting plane tests | Plastic powder sintering Steel |
| Rossi et al. (2020)               | 1:1     | 3.50 × 3.60 m²   | Dynamic          | Shaking table tests (Keddara earthquake) | Bricks                         |
| Fagone et al. (2016)              | 1:5     | 1.63 × 1.58 m²   | Quasi cyclic tests | Constant vertical load and cyclic horizontal load | Sandstone Cement lime mortar |
| Rossi et al. (2017b)              | 1:10    | 343 × 343 mm²    | Quasi-static     | Monotonic pointed loads Vertical displacements of the abutments | Plastic and powder material |
| Cardagnini et al. (2018), 1:4     |         | 0.88 × 1.1 m²    | Quasi-static     | Horizontal shear displacements           | Timber bricks lime mortar      |
| Baraccani et al. (2020)           |         |                  |                  |                                         |                                |
| Foti et al. (2015, 2018)          | 1:5     | 1 × 1 m²         | Quasi-static     | Vertical and horizontal displacements at one support | Polystyrene                    |
| Torres et al. (2019a, b)          | 1:1     | 4 × 4 m²         | Quasi-static     | Vertical displacements at one support    | (timbrel) Bricks               |
| Mark et al. (1973)                |         |                  |                  |                                         |                                |
| Faccio et al. (1999)              |         |                  |                  |                                         |                                |
| Bati et al. (2002)                |         |                  |                  |                                         |                                |
| Theodossopoulos et al. (2002, 2003) |         |                  |                  |                                         |                                |
| Miltiadou-Fezans (2008)           |         |                  |                  |                                         |                                |
Still, the previous studies have different assumptions and limitations. Vaults are complex structural elements, characterised by three-dimensional behaviour and connected to different types of boundary conditions. Therefore, the originality of this research is that it aims at contributing to overcome some limitations of the previous studies, reproducing a well-defined boundary condition setup to study the shear collapse, and comparing the outputs obtained by a quasi-static investigation and by a shake table test, using the same reduced-scale test specimen.

2 Description of the test specimen and test setups

2.1 Geometry and materials

The test specimen is composed of 3D printed plastic blocks with dry joints made at 1:5 scale. The shape and the geometry were defined by considering standard rules of thumb (Heyman 1995; Cangi 2009; Gaetani et al. 2015; Croci 2000; Huerta 2004) and a generic cross-section of a central bay located in a lateral nave of a three-nave church, derived from the intersection of two semi-circular barrel vaults with low rise. It is representative of a generic monumental church (Fig. 1) in Central Italy, generated by a squared base groin vault with a net span of 3.125 m and 1.125 m rise at full scale. Figure 2 presents the geometry of the test specimen.

In historical applications, it is common to find filling material placed above the corners of the vaults, which helps to reduce the thrust forces applied in the lateral piers, stabilising the thrust line (Huerta 2004). In fact, according to Heyman (1969), the safety geometric coefficient of the arch (or of the vault) is obtained by dividing the weight of the element by its thickness, which increases close to the abutments where the section of the vault intersects the filling. In this case, due to the peculiar setup, characterized by tiny dimensions and particular boundary conditions, the filling material above the corners of the vault was not considered. For the application of the filling material, more complex boundary conditions would be needed.

Thus, the test specimen is composed of 1132 blocks (Fig. 3), being 232 made fully of plastic and 880 made of plastic with a steel core (Fig. 4). They have been 3D printed with the SLS (Selective Laser Sintering) technology, which allows to generate small-scale models by a numerically controlled machine starting from a 3D digital model, with high

Table 1 (continued)

| References                          | Scale          | Dimensions | Type of action | Type of tests | Material                       |
|-------------------------------------|----------------|------------|----------------|---------------|--------------------------------|
| Williams et al. (2012)              |                |            |                |               |                                |
| De Matteis and Mazzolani (2012)     |                |            |                |               |                                |
| Shapiro (2012)                      |                |            |                |               |                                |
| Foti et al. (2015, 2018)            |                |            |                |               |                                |
| Rossi et al. (2016); Milani et al.  |                |            |                |               |                                |
| (2016)                              |                |            |                |               |                                |
| Fagone et al. (2016)                |                |            |                |               |                                |
| Rossi et al. (2017b)                |                |            |                |               |                                |
| Rossi et al. (2020; 2020)           |                |            |                |               |                                |
| Carfagnini et al. (2018)            |                |            |                |               |                                |
| Baraccani et al. (2020)             |                |            |                |               |                                |
| Torres et al. (2019a, b)            |                |            |                |               |                                |
geometrical accuracy (± 0.1 mm) and a reasonably short time of production. Indeed, the time of production was estimated between 4–5 h for the blocks of the shell, and 2 h for the supports, counting a total duration of about 6–7 h.

The adopted plastic material is a composite of zp150 powder and zb61 clear binder, printed with a ZPrinter 650. After production, the blocks have been impregnated with Z-bond 101 for improving the strength and durability. This technique was previously adopted by Rossi et al. (2017a) and ensures good stiffness and friction to take into account rigid block assumption, to allow the repeatability of the tests by minimizing the damage. Indeed, the choice of adopting this particular material and technology was led by the need of performing several tests with different configurations without causing damage to the units.

The standard blocks (red blocks in Fig. 3a) were designed, by scaling per 5 the typical dimensions of clay bricks, namely 0.06 × 0.12 × 0.24 m³ (respectively width, thickness, and length), aiming at simulating the most common bond stereotomy of groin vaults in Italy. Width and thickness dimensions are fixed at the values of 12 and 24 mm, while blocks with different lengths are placed along the outer edges of the webs to guarantee the offset of the joints, which was set ¼ of the length of the longest unit (Fig. 3a). Each block was identified by a numeric code, in order to easily rebuild the test specimen after each test. The blocks are arranged radially. Their shape is slightly trapezoidal to compensate for the absence of mortar between them (Fig. 4a) and special care was taken to design the stereotomy of the elements located along the diagonals (grey blocks in Fig. 4b) to guarantee the correct interlocking between adjacent webs. The geometry of the vault, the choice of the radial orientation, and the definition of the cuts along the diagonals were based on the rules of thumb reported by Rodrigo Gil de Hontañón (cited in Sanabria 1982), Formenti (1893), Heyman (1982), Heyman (1995), Huerta (Huerta 2004), Cangi (2012), Como (2013), Giovanetti (2000).

**Fig. 2** Reduced scale vault: a details of the construction on the plywood scaffolding and b at the end of the construction; c main dimensions of the test specimen in mm
This masonry bond needs a temporary structure for the construction of the shells. Therefore, a scaffolding made of plywood has been designed, which is composed of four pieces corresponding to the vault webs. Once the test specimen is built, the scaffolding is removed by letting the pieces slide on proper inclined rails. Care and attention are required to remove the scaffolding, in order not to incur into undesirable configurations and defects. This aspect is important to ensure the repeatability of the tests, since defects in the construction process could compromise the analysis of the results and their comparisons.

The material density ($\rho$) of the composite of zp150 powder and zb61 clear binder was determined to be equal to $0.55 \pm 0.02$ g/cm$^3$. Since it corresponds to a low-density value, the weight of the standard blocks and the blocks along the outer edges was increased by inserting a steel core inside each block (Fig. 4a). This was done to increase the axial compressive stress of the shell of the vault, improving its stability.

In this way, the density of the standard blocks (yellow blocks in Fig. 2b) was measured at the value of $2.70 \pm 0.05$ g/cm$^3$. This strategy was not possible to be performed for the
blocks of the diagonals (orange blocks in Fig. 2b), because of their irregular shape and tiny volume which could not host a steel plate inside, as shown in Fig. 4b. Thus, the disposition of the densities along the test specimen follows the scheme already introduced in Fig. 2b, where standard blocks (in yellow) and diagonals (in orange) are associated to $\rho = 2.70 \text{ g/cm}^3$ and $\rho = 0.55 \text{ g/cm}^3$, respectively, for all the four symmetric webs. Both densities were measured with an electronic scale loaded with several blocks.

The final weight of the test specimen is about 35.6 kg, while the whole structure is about 43 kg, considering the steel base at the bottom of the test specimen. The friction angle ($\mu$) of the blocks is estimated equal to $29.6^\circ \pm 2.46^\circ$, determined by testing 12 samples of standard blocks on the tilting table. Due to the accumulation of error in the 3D printing of the blocks, very thin steel plates ($\rho = 7.8 \text{ g/cm}^3$, $E = 210,000 \text{ MPa}$) have been properly designed and placed at the key of the vault to perfectly close its shell (Fig. 2b). Their thickness varies from 3 mm (at the outer edges) to < 1 mm (at the centre of the vault), while their length is fixed (48 mm).

Although the test specimen presents different types of materials and the scale reduction factor equal to 5 is considerable, the test specimen is still able to represent the main features of masonry groin vaults. As referred by ancient treatises (Heyman 1995), masonry cross vault’s behaviour is mainly linked to the geometry of the system, namely size and shape, less depending on mechanical properties. The scale is less important for static tests (unless of important nonlinear geometric effects, such as in shallow arches or vaults), but it is much relevant for dynamic tests, as highlighted by the work of De Lorenzis et al. (2007).

The choice of using a dry joint test specimen is also an important aspect. Dry joints models may represent not only real dry joint masonry structures, but also ancient mortar joint structures whose mortared joints suffered decay during the time, decreasing their already low tensile strength (Lourenço and Ramos 2004; Pulatsu et al. 2019). A dry joint test specimen also allows to easily perform several tests, without changing the initial conditions. Several authors refer that the stiffness of the joints does not have a significant contribution for the response at the collapse, which is also valid for static loading (Sarhosis et al. 2015; Giamundo et al. 2014). Moreover, neglecting the contribution provided by the strength of the joints can lead to a conservative response.

### 2.2 Test setup

The dynamic tests were performed in the National Laboratory of Civil Engineering, Lisbon, Portugal (LNEC), namely in the three axial shake table (Fig. 5d). The setup was carefully designed, aiming at obtaining the relevant outputs associated to the shear failure of the test specimen, with similar conditions to real prototypes and necessary for the performance assessment.

In order to simulate the particular boundary conditions that cause an in-plane shear response of the vault, the special testing device adopted by Rossi et al. (2016) was used. It consists of a frame composed of four steel squared plates, linked to each other by the use of aluminium bar couples hinged at both ends with uniball joints. In this way, the distance between the abutments does not vary and their rotation along the vertical axis was prevented. Moreover, the abutments of the vault were rigidly fixed on the top of four steel squared plates through a hexagonal bar by interlocking (Fig. 5b).

As shown in Fig. 5b, c, the abutments $p1$ and $p2$ have been fixed using four bolts each, to connect the corresponding steel plates to the flat aluminium surfaces through the threaded holes (Fig. 5b), while $p3$ and $p4$ were left free to move on the flat aluminium surface above.
four spheres each (Fig. 5c). As discussed in Rossi et al. (2016), boundary conditions are a relevant drawback of the test specimen, representing a specific configuration (fully fixed $p1$–$p2$ and completely free $p3$–$p4$), without a lateral wall along the fixed edge, as it is common to find in the real structures. However, the authors consider this configuration to be a representation of the safe side of the reality, with the test specimen that tends to be more deformable.

In Rossi et al. (2016), the displacements were assigned along the Y (longitudinal) direction, through an external actuator, for the quasi-static tests. The instrumentation and boundary conditions used during the shake table tests were similar to those used for the quasi-static tests. At the same time, due to the small geometrical dimensions of the test specimen, specific instrumentation was also implemented. One linear variable displacement transducer ($LDVT_1$) was located at the N–W corner, measuring the relative longitudinal displacement of the movable piers. Six piezoelectric accelerometers ($Acc_{1x}$, $Acc_{1y}$, $Acc_{1z}$, $Acc_{2x}$, $Acc_{2y}$, $Acc_{2z}$) were placed at the bottom of the vault to measure the response of the fixed plated, while five variable capacitance unidirectional accelerometers ($Acc_{3y}$, $Acc_{4x}$, $Acc_{5z}$, $Acc_{6x}$, $Acc_{7y}$) were placed on the vault, since they are compatible with the dimension of the blocks in terms of dimensions and weight.

Moreover, two optical cameras were used to record the response of the key of the western arch and the movable piers using automatic tracking, respectively, along the plane $xy$ ($OP_{1xy}$) and $yz$ ($OP_{2yz}$).

Two video cameras were used to record the tests: one exactly at the top of the test specimen, using scaffolding, and another located in front of the East elevation on a tripod outside of the shake table. From those cameras, the collapse mechanism was evaluated (Sect. 4).

### 3 Experimental campaign results

In this Section, the main results obtained from both experimental campaigns with the reduced scale vault are presented. First, the monotonic quasi-static tests performed by Rossi et al. (2016) are briefly described. Next, the new dynamic identification tests and shake table tests performed at LNEC are presented.

#### 3.1 Monotonic quasi-static tests

Rossi et al. (2016), simulated the simple shear mechanism of the masonry vault, as shown in Fig. 6, in order to apply in-plane shear distortion. As previously described, on one side, two plates ($p1$ and $p2$) were fixed to the aluminium surface, while plates $p3$ and $p4$ were left free to move. The displacement was applied by an external actuator at the $p4$ plate.

The displacement $ds_1$ was monitored by means of a LVDT, while the related force $Fs_1$ was measured by a load cell close to the displacement actuator. The development of the mechanisms up to the collapse was recorded by two high resolution/high frame rate cameras. It was observed that in general all webs show a typical four-hinge asymmetric arch mechanism, opposite in sign. The diagonal shear crack in the extrados can be noted...
and the first collapsing web is the one connecting $p_3$ and $p_4$ where the displacement is applied. The maximum force varies approximately from 13 to 17% of the total weight of the specimen (349.2 N) while the shear distortion (measured by the ratio of the displacement to the free span of the vault) is in the range 3.8–4.8% (Fig. 6).
3.2 Shake table tests

3.2.1 Dynamic identification tests

Dynamic identification tests allow to estimate the modal properties in the elastic regime of the test specimen. In this study, forced vibration tests were performed, in which the input and the response are measured. This kind of test is, characterized by a known input (Ramos 2007; Mendes et al. 2014), namely a “white noise” random signal, with wide frequency range (0.1–40 HZ) and low amplitude (3–8 mm). The test aim at determining the modal parameters of the undamaged and damaged configurations. The amplitudes of the white noise input (here defined in terms of nominal displacement) were increased for evaluating the influence of the amplitude of the signal on the dynamic properties. This is particularly relevant given the use of dry joints, as their stiffness is highly dependent on the normal stress and the structural vault stiffness is also dependent on the crack opening of the joints. The frequencies range from 3.22 Hz (for 8 mm of amplitude) to 4.50 Hz (for 3 mm of amplitude), with a variation of about 0.25 Hz per 1 mm of amplitude. As expected, the increase of the signal amplitude (nominal displacement) causes a decrease in the frequency of the test specimen. The reduction of the frequencies is associated with the opening of the joints, i.e. to the damage on the test specimen, which is higher when the amplitude of the input is increased, corresponding to nonlinear behaviour.

In order to verify the sensitivity of the model to the construction process, the test specimen was rebuilt four times, using the same builders, and trying to follow the same procedure. The dynamic identification tests carried out on different days and after different rebuilds of the test specimen present similar frequencies for the same signal amplitude (5 mm), leading to the conclusion that the test specimen exhibits a small scatter of the modal frequencies (between 3.9 and 4.2 Hz), despite the different constructions or environmental conditions. This aspect further validates the methodology adopted for these tests. In addition, the value of 4.0 Hz (for 5 mm of amplitude) was considered as reference frequency of the test specimen because it was a good compromise between the quality of the signal, in terms of peaks to identify the frequency and the very low nonlinear behaviour (opening of the joints).

Fig. 6 Test setup and results of the quasi-static tests (Rossi et al. 2016)
The dynamic identification tests were also used to monitor the dynamic properties of the test specimen, in particular the first natural mode of vibration, during the seismic tests. They have been repeated after each seismic test, aiming at identifying small variations in the frequency of the test specimen as an indicator of permanent damage due to movement of the blocks, sometimes not easily identified by the human eye (Sharma et al. 2020).

### 3.2.2 Seismic tests

In the seismic tests performed in the shake table, two types of ground motions were used, namely the Emilia earthquake and an artificial accelerogram.

The first seismic input motion was recorded by the seismic station located in Mirandola (Station code: MNR) (Italy) (Moretti et al. 2013), which registered the Emilia Romagna’s earthquake on the 29th of May 2012 at 06:59:53 (UTC), with a Richter magnitude of 5.8. MNR is the closest station to the epicentre that belongs to the RAN network (managed by the Italian Department of Civil Protection). The second seismic input is an artificial accelerogram compatible with the elastic response spectrum defined by the Italian Code (Ministero delle Infrastrutture e Trasporti 2018), considering the same municipality (Mirandola) and rock type of soil.

Figure 7 presents the elastic response spectrum of the Italian Code NTC 2018 (for the horizontal component) and the spectrum of the artificial accelerogram. The spectrum of the Emilia target input is also plotted in the same figure. The vertical dashed line represents the first natural frequency of the test specimen equal to 4.0 Hz (average between all the dynamic identification tests performed with 5 mm of amplitude), corresponding to 0.25 s, in which the spectral acceleration of the Emilia target input (0.6 g) is about 0.17 g above the spectral acceleration of the Italian Code (0.43 g). This is the main aspect that influences the behaviour of the test-specimen under these two different seismic inputs.

In the same way, Fig. 8 shows the time histories of the selected inputs, which correspond to 100% of the respective seismic action. The Emilia target input is characterised by a duration of 8.93 s. Its peak ground acceleration (PGA) is equal to 284.6 mg and the maximum displacement (PGD) is equal to 31.00 mm, while its root mean square acceleration (RMSA) and root mean square displacement (RMSD) are respectively equal to 72.50 mg and 7.09 mm. The artificial accelerogram presents the same duration, a PGA equal to 173.3 mg, a maximum displacement equal to 16.00 mm, its RMSA has a value of 46.75 mg and the RMSD is equal to 7.87 mm. The two inputs were also subjected to a signal processing using a high pass Butterworth filter with 8 poles and a cut-off frequency equal to 1 Hz.

The duration of the inputs, applied in the seismic tests, was scaled in order to follow the Cauchy-Froude’s similitude. To select the corresponding scaling factor, the material values of an ideal prototype, identified during the previous work of this research (see Rossi et al. (2016) for all the data) were taken into consideration. A basic comparison was made through a finite element model (full scale) to evaluate the ratio between the frequencies of the specimen and the prototype. The first frequency corresponds to the measured frequency of the tested specimen (4.0 Hz), while the second was obtained by performing eigenvalue analysis on a finite element model that reproduces the full-scale prototype. Due to the three different scales, which derive from the complexity of the model (namely geometry, density, and deformability), none of the laws was completely respected. However, Cauchy-Froude presents a frequency ratio closer to the expected
nominal relation. Thus, Cauchy-Froude’s law was adopted for scaling the duration of the two inputs according to the relation $t^{0.5} = 5^{0.5}$, in which $t$ is the geometric scale factor equal to 5. Following again Cauchy-Froude’s law, the acceleration scale factor is unity (Carvalho 1998).

In this study, only the $y$-longitudinal components (north–south direction), which correspond to the HNN channel of the MNR station for Emilia earthquake, were considered in order to excite the movable piers and induce the shear failure in the vault (Fig. 8).

The use of two different type of inputs (Emilia, impulsive, and Artificial, more regular) was determined by the significative importance of investigating the response with one dominant pulse action and one more regular signal with an almost constant intense phase, expected by the Italian Code (Ministero delle Infrastrutture e Trasporti 2018). Indeed, as stated in literature (e.g. Dejong and Ochsendorf (2010)), it is expected that the primary impulse of an expected ground motion is fundamentally important in predicting the collapse. However, artificial earthquakes have specific characteristics, such as the multiple consecutive impulses and the higher duration of the action, that might have an amplifying effect on the rocking motion, also causing the collapse.

The testing sequence of the vault is presented in Table 2, in which the seismic action was applied with increasing amplitude until collapse and the model was rebuilt 3 times (indicated here as Construction 1 to 3). Before the first and after each seismic test, dynamic identification tests (DIT) were performed, with the amplitude of 5 mm, in order to evaluate the decrease of frequencies as a function of the cumulative damage, which was almost equal to zero until 50% of the Emilia ground motion and until 100% of the artificial ground motion (Table 2). When only a few blocks fell during the test, the test specimen was repaired, in order to keep the same starting point to the test specimen (undamaged test specimen). In the case of Construction 3, the test specimen was not repaired, aiming at simulating a sequence of shocks and the consequent accumulation of damage.

![Elastic response spectra of the selected input signals (Emilia target input and artificial accelerogram) and comparison with the 475-years return period design spectrum for Mirandola according to NTC2018. The periods are scaled according to the similitude relationships](image_url)

**Fig. 7** Elastic response spectra of the selected input signals (Emilia target input and artificial accelerogram) and comparison with the 475-years return period design spectrum for Mirandola according to NTC2018. The periods are scaled according to the similitude relationships.
4 Analysis of the results

To compare the results of the static tests carried out by Rossi et al. (2016) with the shake table tests (ST), the analysis of collapse mechanism was adopted, in which a sequence of photos, with the number and locations of the hinges, was used. Looking at Fig. 9, the centre of the vault is the first to collapse in both tests, due to its geometrical configuration and the verticality of the joints. The shear failure is the first to occur and it is represented by crack opening along the diagonals. It is noted that, in the quasi-static tests, the diagonal cracks appear only along the S–E direction, between the movable piers, due to the monotonic sign of the action. On the other hand, for the shaking table tests, those cracks appear both at N–W and S–W diagonals, between the fixed piers, due to the cyclic movement caused by the action (accelerogram). Similarly, during the shake table tests, the hinges started to appear from the East side of the test specimen between \( p_1 \) and \( p_2 \), the fixed piers, while during the quasi-static tests the first hinge occurred between the movable piers (west side) \( p_3 \) and \( p_4 \), resulting in a mirrored mechanism. This aspect occurs because the movable edge can recover part of the damage, namely closing the joints during the shaking table tests (cyclic movement). In contrast, in the fixed edge, which is more rigid, the opening of the joints is larger. For the quasi-static tests, the joints of the movable edge open without the possibility of recovering the damage (monotonic movement), resulting as the first edge to collapse. Moreover, it is worth noting that the actuator used in the quasi-static tests constituted a constrain, making the boundary conditions slightly different from the dynamic tests.

Analysing the dynamic test results, the external edges (North and West side) continue to stand, even when the other sides already fell down. The hinges are located at the East elevation, both for the Emilia and the artificial inputs (Fig. 9).

The steel plates at the corners make the boundaries of the shell of the vault stiffer and, therefore, two hinges occur where the steel plates do not act anymore. This is in agreement with the prediction of Oppenheim (1992) and De Lorenzis et al. (2007), based on the minimum energy formulations. Considering the response obtained from

| Table 2 | List of shaking table tests carried out on the reduced scale vault (chronological order) |
|---------|--------------------------------------------------------------------------------------|
|         | Tests with Emilia input                                                           | Tests with Artificial input                                    |
|         | DIT | Seismic action | DIT | Input |
| Construction 1 | 4.10 | 10% | Frequency [Hz] | % of action and notes |
|            | 4.10 | 25% | 4.00 | 50% |
|            | 4.00 | 50%—recovered | 4.00 | 75% |
|            | 3.91 | 75%— collapse | 3.90 | 100% |
| Construction 2 | 3.61 | 60%— collapse | 3.71 | 125% |
|            |     | (*initial minor damage) | 3.35 | 200%— collapse |
| Construction 3 | 4.00 | 55%— unrecovered | 3.42 | 150% |
|            | 3.22 | 25%— aftershock1 | 3.22 | 35%— aftershock2 |

*initial minor damage*
the shake table tests, the arch mechanism is clear (four hinges), and it firstly appears between the fixed supports (Fig. 9).

As widely observed for the behaviour of similar structures like arches, different loading distribution leads to a different response. In the case of the vault, the three-dimensional complexity of the structure amplifies this concept. Indeed, the core aspect of this research is that the quasi-static tests are mainly associated with the gravitational set of loads, while the shake table tests consider the inertial force acting on the structure.

The main objective is to assess the groin-vault behaviour independently from the load type and then to stress the fact that for this kind of structures, the nonlinear static approach represents an approximation of the reality that does not replicate all loading scenarios. At the same time, even two different inputs in the shaking table lead to different quantitative conclusions due to the spectra contents of the seismic action itself.
Regarding the comparison between the two ground motions used during the shake table tests, and as shown in Table 3, the specimen remained undamaged until the scaling factor of 75% and 200% for the Emilia and artificial inputs, respectively.

As previously referred, it is possible to notice that the impulsive nature of the Emilia earthquake, caused a rapid collapse of the vault, in comparison with the artificial acceleration (Table 3). However, the type of collapse mechanism remains the same, in terms of the evolution of damage and formation of the hinges (Fig. 10). Both types of seismic action show a clear shear behaviour, which occurs along the diagonals that join $p_2-p_4$ and $p_1-p_3$, due to the positive and negative direction of the accelerations at the base. It is notable also that the hinges interfere only with the parallel edges (East before and West later), while the transversal webs are not involved in the collapse, and they keep a simple-arch behaviour until the end of the tests. These observations are valid for every amplitude of the shake table tests, starting from the value of amplitudes that could cause a more relevant state of damage. For the sake of clarity and conciseness, only the collapse mechanism and the time series of the displacements before reaching the collapse itself are compared, namely for the 75% of the Emilia input and the 200% of the artificial input (Figs. 10 and 11).

From the time series of the displacements of the movable piers (Fig. 11), referred at the amplitude of the collapse, the major peaks occurred between 2.5 and 3.5 s of the duration.

The performance of the test specimen in terms of displacements and accelerations require a more detailed analysis. It is confirmed that the impulsive nature of the Emilia earthquake significantly influences the response of the structure and, even with lower values of spectral acceleration, induces earlier damage in the structure. Acceleration and displacement response spectra of the different time histories, all built for a damping of 5%, are very different in content and peaks (Fig. 12).

Table 4 shows some relevant outputs associated with the recorded response of the structure. Besides the fact that the content of the two inputs is different, the
displacements recorded by the movable piers showed good consistency for all shake table tests. The values of drifts (ratio between the relative displacement and the span), recorded by the optical camera and the LVDT, placed respectively above the movable piers $p3$ and $p4$, ranges from 5.12 to 7.17%, for both inputs. The average value is about 6%, either for the Emilia input or the artificial input at the collapse, independently of the value of PGA. It is noted that the difference between the two values obtained for the different types of inputs is less than 1% for the optical camera and about 0.1% for the LDVT. From the drifts, it is possible to conclude that under seismic motion the limit drift value that can cause the collapse is equal to 6%. The ultimate drift value measured in the quasi-static tests was about 4.3% of its lateral edge and it corresponds to a conservative result, in comparison with the results obtained from the shaking table tests.

It has been shown in the past, e.g. for masonry building, and here confirmed, that dynamic drifts are larger than quasi-static (Abrams et al. 2017; Lagomarsino 2015; Vasconcelos 2005; Gaetani et al. 2017).

For an additional comparison between the types of input, the hysteretic behaviour of the reduced-scale vault is presented in Fig. 13, in terms of a relation between the shear forces/horizontal inertial forces and the displacements. The shear forces developed in the structures, as well as the displacement, have been analysed based on the recorded acceleration and displacement time-histories. The base shear coefficient was calculated by dividing the inertial force associated with each web of the vault by its self-weight. Inertial forces associated with the webs of the vault were calculated by assuming the mass of the reference web to be lumped at the recording accelerometers. In fact, due
to the geometrical symmetry of the test specimen, the mass associated with each web corresponds to \( \frac{1}{4} \) of the total mass of the test specimen. The inertia forces were computed for each shake table test but, for sake of simplicity, only the most representative scenarios are presented in Fig. 13. The shear coefficient was plotted versus the longitudinal displacement placed at the top of the west arch, measured by the optical camera OP2. Figure 13a, which refers to stages with the lower amplitudes, represents a linear elastic phase with a very low value of excitation. This behaviour is associated with the

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**Fig. 10** Damage progression of the test specimen during the intense phase of the Emilia seismic input 75% vs. the artificial input (200%) (5 frames/s). (Left side: fixed supports p1–p2; right side: movable supports p3–p4)

**Fig. 11** Time series of the relative displacements of the movable piers (p3 and p4) at the collapse for Emilia 75% and Artificial 200%
undamaged configuration of the vault, namely when the openings of the joint do not lead to the fall of blocks. The linear range is evident even for the hysteretic curves plotted at the amplitude of the collapse (Fig. 13b), in which two vertical lines are associated with the initial phase. Then, very high nonlinear behaviour, for both inputs, is observed. As noticed before in the evaluation of the drifts, the hysteretic curves also show similarities when considering the Emilia input and the artificial input. The scattered behaviour of the curve is marked by the different slopes of the curves, which represents a change in terms of stiffness and permanent deformation. Emilia input and artificial input show a quite similar nonlinear behaviour but mirrored in sign. In general, when the artificial input is applied, the structure is characterized by a higher capacity in terms of shear coefficient, while with the Emilia input the structure shows a higher deformation. Despite the different response in terms of displacements, when compared with the quasi-static tests, it was observed that the maximum strength capacity ranged between 13 and 17% of the force/weight ratio, which is compatible with the value obtained by the shake table tests (Fig. 13b).

Figure 14 shows the plots for the incremental dynamic tests performed in the shake table tests, for the Emilia input and the artificial input. The maximum base shear at each stage of testing and the corresponding value of displacement recorded by the sensor located at the key of the west arch (OP2y), occurring at the same instant of time, is reported. The first points show the linear range of response, despite the difficulty to observe the differences in terms of stiffness due to the low level of amplitude. During this kind of tests, it is normal to get a better response for a medium value of amplitude and intensity. The tests with the Emilia input and the artificial input show a similar capacity in terms of base shear and deformations. This outcome is in line with what was obtained in previous research on reduced-scale test specimens (Tomaževič 1999).

The differences in terms of inputs and response can be also identified in Fig. 15. As Candeias et al. (2016) stressed, a straight line with a 1:1 slope means that the absolute accelerations are equal in the base and the other selected points of the test specimen. This means a rigid body motion, whereas if the slope is higher means that there is dynamic amplification in the test specimen. Finally, if there is an exponential increase of the plots with the increase of the input motion intensity, it means that the test specimen has reached the nonlinear behaviour. Looking at Fig. 15a, c, it is possible to observe the dynamic behaviour of the vault for the Emilia input. It can be defined as linear from the tests with 10% to 60%, except for the point for the 55%, which corresponds to the third construction (Table 2). This could be associated with some relative movements during the recording state of the sensor or due to the construction process. Before the collapse, firstly achieved with 75% of the seismic motion, it is possible to notice an exponential increase significantly evident from the accelerations recorded on the top of the movable piers (Acc3y, Acc7y), less remarkable for the ones placed on the fixed piers (Acc1y, Acc2y).

| Type of input (at the level of the collapse) | Test input % | PGA [m/s²] | PGV [mm/s] | PGD [mm] | Maximum ABS displacement p3 (Optical camera) [mm] | Drift [%] | Maximum REL displacement p4 (LVDT) [mm] | Drift [%] | Drift average [%] |
|--------------------------------------------|--------------|------------|-----------|---------|-----------------------------------------------|----------|---------------------------------------|---------|-------------------|
| Emilia input                               | 75           | 1.6        | 139.5     | 17.8    | 44.8                                          | 7.2      | 32.0                                  | 5.1     | 6.2               |
| Artificial input                           | 200          | 7.5        | 241.2     | 64.7    | 39.1                                          | 6.3      | 32.6                                  | 5.2     | 5.7               |
| Quasi-static tests                         | Collapse     |            |           |         |                                               |          | 24.7–31.2                             | 3.8–4.8 |                   |

Table 4 Tests results for each dynamic input and the quasi-static tests
On the other hand, for what concerns the artificial input, Fig. 15b, d), the linear range was identified between the 10% and the 120% of the ground motion, while the exponential rate is mainly localised on the last two sequences of tests (150% and 200%), when effectively the severe damage occurred.

Fig. 12 Displacement and acceleration response spectra for the signals measured at the shake table for the two inputs: Emilia input (left) and artificial input (right)

Fig. 13 Hysteretic behaviour (base shear coefficient vs top arch displacement) for different stages of testing: Emilia EQ 35% vs artificial EQ 50% (a) and Emilia EQ 75% vs artificial EQ 200% (at the collapse) (b)
5 Conclusions

This work describes a comprehensive experimental investigation composed by shake table tests developed for the characterisation of the shear failure of a reduced scale test specimen (1:5) of a 3D printed groin vault. Two experimental campaigns using different approaches of loading were discussed, namely: (a) monotonic quasi-static tests, previously performed by Rossi et al. (2016); (b) shake table tests applying the Emilia earthquake (2012) and an artificial accelerogram for the same zone. The shake table tests include also dynamic identification tests.

The objectives of this work were to evaluate the variation of the response considering the two different types of loading (quasi-static and dynamic) and the two different types of seismic action applied in the shake table tests (recorded vs artificial accelerograms). The results were mainly compared in terms of maximum load capacity, drift/displacement capacity and collapse mechanisms. Damage and collapse mechanism. One potential objective was represented by the characteristics of the test specimen, rebuildable which can be used for several experimental applications.

The results obtained from the dynamic identification tests, performed before and after each seismic shake table test, allowed to evaluate the reduction of frequencies of the test specimen, which can be used as an indicator of the damage. The development of a four-hinge symmetric mechanism was observed in two webs during the quasi-static tests. On the other hand, a similar mirrored mechanism was obtained for the shake table tests, but the first location of the hinges appears between the fixed piers of the vault, associated with the stiffer behaviour of the fixed edge.

The displacement/drift levels achieved in the shake table tests were higher (about 6% of the span) than the ultimate displacement/drift obtained from quasi-static tests (about 4% of the span), using the same reduced scale vault of Rossi et al. (2016). This aspect allows to
conclude that, in general, the monotonic approach is more conservative than the dynamic approach. This is according to the expected, but the novelty of the paper is that this statement was studied only in case of walls or simpler structures, but it was never experimentally investigated before for groin vault structures.

The capacity in terms of base shear was similar in both tests. The differences in terms of collapse between the Emilia earthquake and the artificial seismic input are influenced by the spectral acceleration of the Emilia input that is about 0.17 g above the spectral acceleration of the Italian Code (0.43 g), or the first period of the test specimen. This stresses the fact that, according to the most recent codes, a significant number of records must be selected in order to allow a better description of the safety of the structure (Lagomarsino and Cattari 2015).

Finally, this study, complemented by the dynamic identification tests, provides useful data for the development and calibration of reference numerical models, adopting different strategies, which then allow to evaluate the seismic behaviour of vaults with different geometry and seismic action. Moreover, this research provides a step for further investigations, which may contain more realistic boundary and loading conditions, such as the presence of the lateral wall, the presence of filling material, and the three components of the seismic actions.

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Declaration

Conflict of interest The authors certify that they have no affiliations with or involvement in any organization or entity with any financial interest, or non-financial interest in the subject matter or materials discussed in this manuscript.
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