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# NEWSLETTER

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# SECED & ICOMOS Special Issue: Cultural Heritage Structures

*In this issue*

# Foreword

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Earlier this year, Richard Hughes from ICOMOS approached SECED about collaboration between our two societies. ICOMOS is the International Council on Monuments and Sites, and is interested in conservation of cultural heritage worldwide. There is clearly a key role for the earthquake engineering profession to play in conservation, and often some of the most challenging seismic work is in satisfying safety requirements while being respectful of the existing building fabric. Linking the two communities together seemed to be of great mutual benefit.

Richard set up a discussion between us (Damian and Vasilis, representing SECED and ICOMOS, respectively), and this led to organising a co-badged evening talk at the Institution of Civil Engineers on 27th March 2019. The speaker was Federica Greco, and she presented a seismic retrofit project of historic earthen structures in Peru, carried out by the Earthen Architecture Initiative of the Getty Conservation Institute (GCI) in collaboration with the Ministry of Culture of Peru, the School of Science and Engineering of the Pontifical Catholic University of Peru, and the University of Minho. Federica's presentation was impressive in its breadth, but also provided enough detail for the technical folks in the audience to be able to appreciate the rigour of the work done. Federica provides a summary of the talk in this issue of the Newsletter.

In the spirit of continuing the collaboration kicked off earlier this year, we are happy to present this special issue of the Newsletter. Aside from Federica's contribution, we also include two other technical papers both first-authored by Vasilis Sarhosis. Vasilis is the Chair of the UK Scientific Committee on the Analysis and Restoration of Architectural Heritage (ISCARSAH-UK), i.e., a Scientific Committee within ICOMOS, and an Assistant Professor in Structural Engineering at the University of Leeds. The first of these papers highlights the strong role of advanced analysis methods (familiar in the earthquake engineering community) in seismic assessment of heritage projects. The second paper relates to damage caused by induced seismicity in the Groningen region of the Netherlands. This has been particularly topical in the last 7+ years, and prompted several technical papers at September's SECED conference in Greenwich. Again, this work highlights a strong overlap in the interests of ICOMOS–ISCARSAH and SECED.

We are continuing to look for other opportunities for our two organisations to collaborate. We are happy therefore to announce that the SECED Young Members' Subcommittee is organising a co-badged event with ICOMOS, visiting the University of Leeds and the Marsh Lane Project site, for early 2020. Details will be disseminated to the SECED membership when they are finalised.


(a) (b)

**Snapshots from the SECED–ICOMOS March evening lecture: (a) (from left to right) Damian Grant, Vasilis Sarhosis, and Federica Greco; (b) Federica Greco at the rostrum (© Richard Hughes).**

# Seismic Retrofitting of Historic Earthen Structures in Peru

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## 1. Introduction

The Seismic Retrofitting Project is one of the projects developed by the Earthen Architecture Initiative in the Getty Conservation Institute in collaboration with the Ministry of Culture of Peru, the Pontifical Catholic University of Peru, and the University of Minho. The project aims at characterizing the vulnerability of traditional earthen architecture in Peru, and the design of seismic retrofitting solutions using traditional construction techniques and materials, verified with advanced engineering methods. Finally, working closely with the Peruvian authorities and universities, the project intends to provide guidance for local and international professionals working in the field.

## 2. Prototype buildings

Four building typologies were identified in the first phase of the project as representative of the earthen architecture in Peru. Cancino et al. (2013) conducted a campaign to inspect earthen buildings in different parts of the country, and finally selected a case study for each typology (Figure 1).

Figure 2 shows the construction typologies of the selected prototype buildings (Cancino et al., 2013). Ica Cathedral represents the typology of the 'coastal cathedral': large buildings consisting of an external masonry envelope and an internal complex timber frame. The external masonry envelope consists of adobe walls sitting on brick masonry course and rubble stone masonry foundations. The church of Kuño Tambo represents the typology of the 'Andean church'. The church is a single rectangular space defined by massive adobe masonry walls sitting on rubble stone masonry foundations and base course. Two additional volumes are attached on one side, housing the baptistery and

**Figure 1: Case studies: (a) Ica Cathedral; (b) Kuño Tambo; (c) Hotel El Comercio; (d) Casa Arones (© J. Paul Getty Trust).**

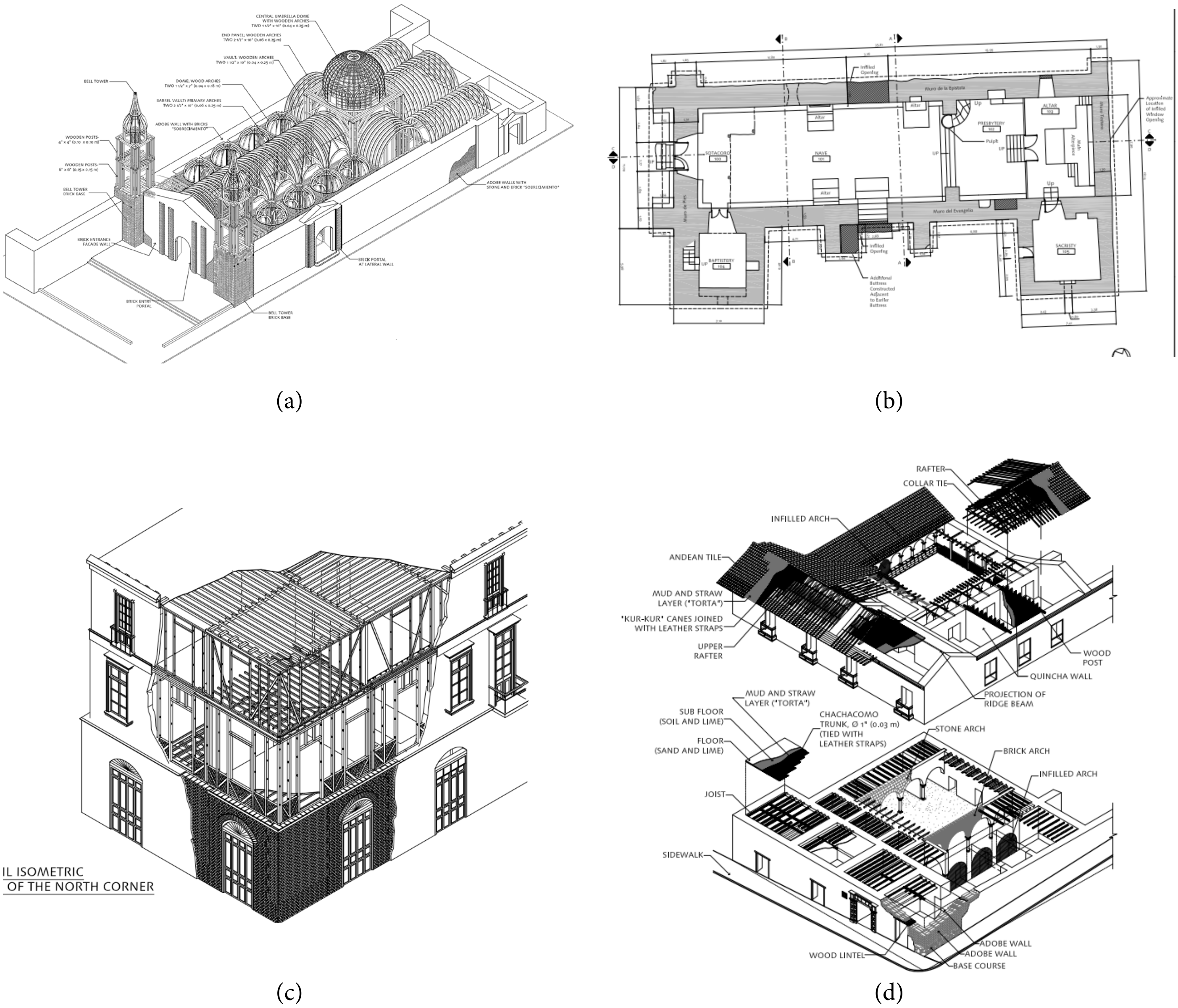


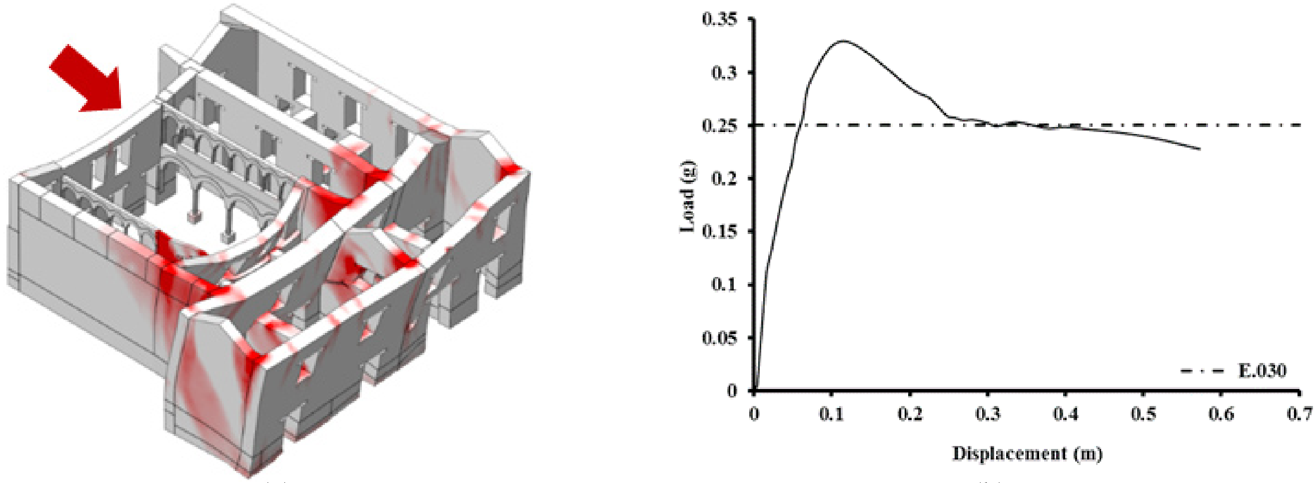

**Figure 2: Different construction typologies of the prototype buildings: (a) Ica Cathedral; (b) Kuño Tambo; (c) Hotel El Comercio; (d) Casa Arones (© J. Paul Getty Trust).**

the sacristy. The Hotel el Comercio represents the typology of 'coastal residential buildings'. Its large structure consists of masonry walls at the ground floor, whereas the first and second floor walls are made of a traditional timber system called *quincha* (wattle and daub). Finally, the typology of '*casona*' (i.e., vernacular house mostly common in the interior regions of the country) is represented by the Casa Arones building. This vernacular architecture is shaped around an internal courtyard, generally made of stone and brick masonry. Walls are made of adobe masonry sitting on rubble masonry foundations.

## 3. Analysis and testing

The seismic assessment of the buildings in their current configuration was evaluated using final element modelling, following the general recommendations on advanced modelling of historic earthen sites provided by Lourenço and Pereira (2018). Mass proportional pushover analysis was performed to assess the capacity of the prototype buildings under seismic loading in their main directions, since Endo et al. (2017) has demonstrated the compatibility of results obtained from nonlinear static and time history analysis in similar structures. A macro-modelling approach was used for the masonry walls, combined with a total strain rotating crack model with softening in tension and compression (Barontini and Lourenço, 2018; Ciocci et al., 2018; Karanikoloudis and Lourenço, 2018). Capacity, damage progression and failure modes of the prototype buildings were analysed (Figure 3); also, the peak ground acceleration (PGA) corresponding to the maximum lateral capacity was compared to the PGA provided by the Peruvian Code (MHCS, 2016).

Validation of the numerical modelling was done against in-situ and laboratory testing (Torrealva et al., 2018).




(a) (b)

**Figure 3: Pushover analysis results in the case of Casa Arones building: (a) principal crack width (the red arrow indicates the direction of the earthquake load); (b) load–displacement diagram (the dotted line represents the PGA requirement provided by the Peruvian Code, MHCS, 2016).**

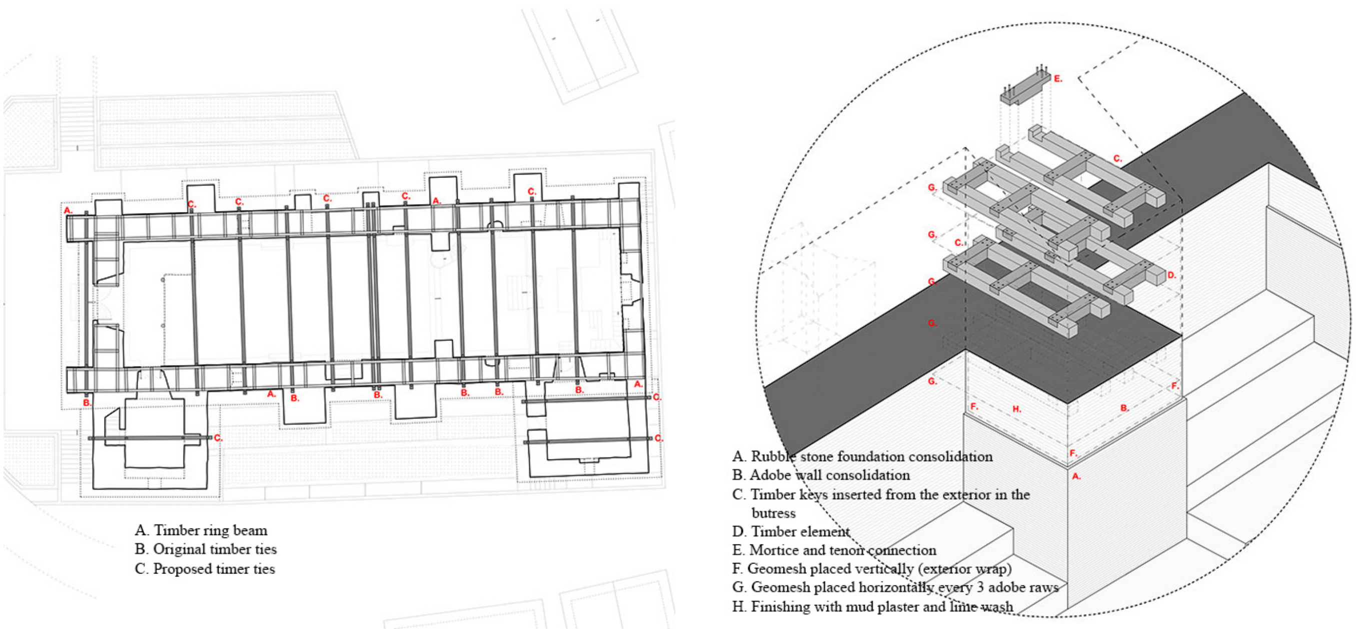
Several in-situ testing techniques were used, including thermographic camera imaging, sonic testing, and dynamic structural identification testing using ambient vibrations. The testing campaign aimed to: (*i*) characterise the morphology of the structure when this was not visible or accessible, (*ii*) estimate the mechanical properties of the main structural materials or systems, (*iii*) better understand the extent of damage observed, and (*iv*) define the global structural behaviour.

Preliminary analyses were performed on partial models of the buildings, and additional analyses were performed to verify the sensitivity of models to assumptions regarding material properties, geometry and connections.

Finally, response predictions from the models were validated also in terms of correlating crack patterns with damage observed in the existing buildings.

## 4. Retrofitting design

Seismic retrofitting design was performed for the Ica Cathedral and the Kuño Tambo Church. In both cases, the adopted retrofitting strategy aimed to preserve the elements of the highest significance, and use traditional techniques as well as state-of-the-art methods for the intervention, trying to minimise the loss of historic fabric. A




(a) (b)

**Figure 4: Examples of implemented seismic retrofitting solutions: (a) Tie beams with anchor keys connecting longitudinal walls, ring beam at roof level, and orthogonal timber corner keys located at different levels along the height of masonry walls; (b) Adobe masonry buttresses on rubble stone masonry base course and foundation, anchored to the existing walls using timber anchors (© J. Paul Getty Trust).**


(a)


(b)

**Figure 5: Construction site visit during a seismic retrofitting workshop: (a) Lecture on non-destructive testing; (b) participants trying one of the mortar mixes for repointing a sample panel (© J. Paul Getty Trust).**

detailed description of the retrofitting design is provided by Lourenço et al. (2019).

One of the main objectives of the design was to improve the out-of-plane capacity of masonry walls, which was achieved by (*i*) improving or re-establishing the connection between different parts of the structure (introducing elements, such as timber corner keys, tie beam with anchor keys, timber ring beam, etc.), and (*ii*) reducing the span of the long masonry walls (introducing traditional elements such as buttresses or brick masonry piers) (Figure 4).

## 5. Implementation and dissemination

The final phase of the project included the implementation of the seismic retrofitting solutions in the prototype buildings, and dissemination to the professional community. In this context, a series of workshops on seismic retrofitting of earthen constructions took place in Cusco, Peru. Participants were asked to attend a two-day course, which included activities such as proposing and evaluating different available seismic retrofitting techniques, and discussing decision-making processes for seismic retrofitting design that weight architectural and engineering principles. Participants visited the construction site at the Kuño Tambo Church to observe some of the discussed retrofitting details (Figure 5). The site visit included two lectures on available non-destructive testing techniques and on repointing mortar specifications.

Furthermore, a series of research reports is provided on the Seismic Retrofitting Project webpage summarising the methodology, findings and conclusions at different stages.

## Acknowledgments

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# Numerical Analysis of Multi-Drum Columns: The Case Study of the Two-Storey Colonnade of the Ancient Forum in Pompeii

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***Abstract***

*There is much to learn from the forgotten architectural and structural principles developed by ancient builders. Novel structural analysis tools extending traditional methods could allow engineers to understand the mechanisms that have allowed surviving structures to avoid structural collapse and destruction during strong earthquakes. By better understanding the seismic performance of ancient structures, better decisions on the conservation and rehabilitation techniques could be made. This study aims to investigate the seismic vulnerability of the two-storey colonnade of the Forum in Pompeii. Software based on the Discrete Element Method (DEM) is used. The colonnade was represented by a series of distinct blocks connected by zero thickness interfaces which could open and close depending on the magnitude and direction of stresses applied to them. From analysis results, it was shown that the seismic behaviour of these structures exhibits rocking and sliding phenomena between the individual blocks. Specifically, the drums may rock either individually or in groups resulting in several different shapes of oscillations. Also, during strong earthquake excitations, the drums in ancient columns act as a dissipating mechanism, allowing relative deformations, and in most cases preventing collapse.*

## 1. Introduction

Understanding the seismic behaviour of ancient columns contributes to the rational assessment of potential proposals for their structural rehabilitation and strengthening, while it may also reveal some information about past earthquakes that they may have experienced. As ancient monuments have been exposed to a large number of strong seismic events throughout their life span, those that survived have withstood a natural seismic testing, and it is therefore useful to understand the mechanisms that allowed their survivability. Since analytical studies of such multi-block structures containing large number of drums and subjected to strong earthquake excitations is practically infeasible, and given that laboratory tests are very difficult and costly to perform, numerical methods (Pompei et al., 1998; Makris and Zhang, 2001; Manos et al., 2001; Mitsopoulou and Paschalidis, 2001; Psycharis et al., 2003) can be used to simulate their dynamic behaviour and seismic response.

Today, the most widely used approach to model the mechanical behaviour of masonry structures is the Finite Element Method (FEM). Although FEM can be used for the analysis of structures with some discontinuities, the method is not suitable for the analysis of discontinuous systems that are characterized by continuous changes of the contact conditions among the constituent bodies. On the contrary, DEM has been specifically developed for systems with distinct bodies that can move freely in space and interact with each other through contact forces. Research efforts towards using DEM in simulations of ancient structures have already shown promising results, indicating a potential for further utilization of this method. Recent research work based on commercial DEM software applications (Mouzakis et al., 2002; Papantonopoulos et al., 2002; Komodromos et al., 2008; Sarhosis, 2012; Sarhosis and Sheng, 2014; Sarhosis et al., 2015; Sarhosis et al., 2016) demonstrated that DEM can effectively be used for the analysis of such structures.

This article presents the results from a numerical model developed by DEM to investigate the seismic vulnerability of a block-based frame of architectural heritage in the ancient city of Pompeii, Italy, aiming to gain a better understanding of its overall structural behaviour.

## 2. The archaeological site of Pompeii

The archaeological site of Pompeii is a Roman town located near Naples, Italy. Pompeii was a busy commercial hub between Neapolis (Naples), Nola and Stabiae (Castellammare di Stabia); three main cities in southern Italy. The town was destroyed during a catastrophic eruption of the Vesuvius volcano in 79 AD. As a result of the eruption, the city was mostly destroyed and buried under 4 to 6 m of pumice ash. The site was covered in ash for about 1,500 years until its initial rediscovery in 1599, and a broader rediscovery almost 150 years later by a Spanish engineer in 1748 (Maiuri, 1942). The ancient city of Pompeii has been preserved throughout the centuries, due to the lack of air and moisture, and survived earthquakes, thus providing a complete picture of the city as well as of the daily life at that period. However, since its excavation, Pompeii has experienced many events endangering its conservation. For instance, during the Second World War the Forum area was struck by a bomb and suffered severe damages. Nowadays, the ancient town consists of many partially collapsed buildings.

Since 1997, the ancient city of Pompeii has been recognised as a world heritage site by UNESCO. Today, Pompeii is one of the most popular tourist attractions in southern Italy with millions of visitors every year. During its lifetime, the city suffered many earthquakes. Before the catastrophic eruption, a strong earthquake hit the town in 62 AD. However, no reconstruction works were carried out. After its rediscovery in 1748, several other earthquakes hit Pompeii. The last earthquake to hit the city, producing damages in the entire archaeological site, was in 1980. Weathering, erosion, light exposure, water damage and poor methods of excavation and reconstruction have further resulted in the deterioration of the archaeological structures in Pompeii.

## 3. Building practice in ancient Pompeii

During its lifetime, the city underwent many earthquakes. 'Innovative solutions' in the building practice of the time were conceived to improve the seismic performance of structures; this is evident from the execution speed and the overall economy in reconstruction works after the 62 AD earthquake. Analysing the partially collapsed structures, constructive wisdom can be identified in the structural details conceived by ancient builders in Pompeii. For example, analysis of the pillars of the Basilica in the Forum shows that mortar joints were aligned with an unprecedented staggered pattern able to increase the seismic performance of the structure (de Martino et al., 2006).

## 4. The archaeological site of Pompeii

The structure under investigation is a two-storey colonnade of the Forum in Pompeii (Figure 1) made of white


**Figure 1: (a) The Forum in Pompeii: (a) Aerial view of the southern part of the Forum showing the colonnade under investigation, denoted with a red circle; (b) two-storey colonnade with multi-drum columns and multi-blocks segmented trabeation.**

limestone. The colonnade was erected in the main square of the ancient town, where political, economic and religious events were taking place. To prevent the passage of carts, the pavement was raised with respect to the height of the adjacent roads by two steps. It is believed that initially (i.e., VII–VI century BC) the Forum had an irregular shape. Today, the Forum has a rectangular shape with dimensions 143 m long by 38 m wide. Probably at the end of the II century BC, a double row colonnade was built, made of tuff. Some years later in 79 AD, the colonnade was reconstructed, and tuff was replaced with white limestone. The columns of the second storey followed the Ionic order, while the columns of the lower storey followed the Doric order. In the second half of the 20th century, only a small part of the second storey was re-erected for educational and touristic purposes. In 1980, an earthquake caused damage to the colonnades of the Forum, and it was decided to remove the beam over the second storey.

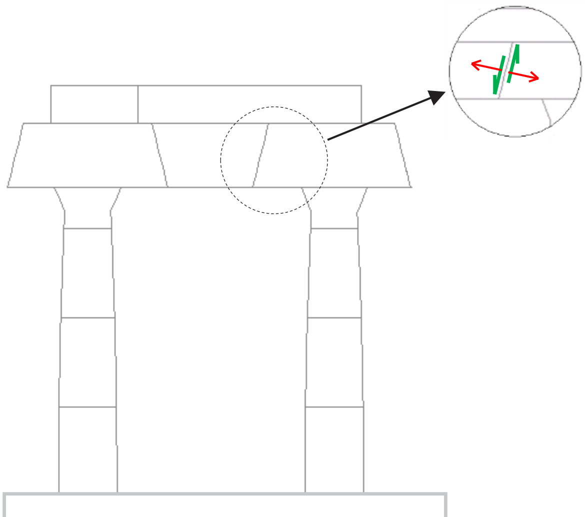
An 'innovative solution' was adopted for the construction of the trabeation. To avoid long-span beams over the columns, short segments were built up providing oppositely inclined pattern edges, i.e., a 'flat arch' configuration (Figure 2a). This solution was conceived to simplify construction phases and prevent lifting long-span heavy beams over the columns. Blocks mutually supported over inclined surfaces (keystones), induced a horizontal thrust along the trabeation capable of carring loads without any tensile strength. In a fully functioning structure, each keystone pushes over the two adjacent blocks, and this load is counteracted. Static problems may arise at the corners of the structure, where the absence of symmetric mutual interaction can lead to column overturning. To overcome the above hurdle, the builders avoided using reduced size blocks at the end of the colonnade. Instead, a long block (solid long beam in Figure 2b) was used. In this way, the horizontal thrust, which was not counteracted by the contiguous blocks, was counteracted by two columns, thus halving the horizontal thrust. The examination of the methods employed by the ancient builders revealed the continuous research and evolution in the design of structures against earthquakes (Adam, 1989).

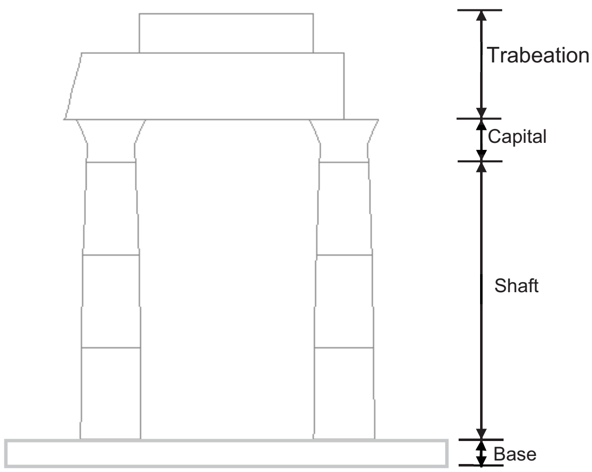
## 5. Development of the computational model

Geometric representations of the two-storey colonnade of the Forum in Pompeii were created in the computational model (Figure 3). Each stone unit of the monument (i.e., drum of the column and stone block of the trabeation) was represented by a deformable block separated by zero thickness interfaces at each joint. Zero thickness interfaces between each block were modelled using the elastic–perfectly plastic Coulomb criterion defined by the elastic normal stiffness ($JK_n$), the shear stiffness ($JK_s$), and the joint angle of friction ($J_{fric}$). The material parameters used in the computational model are shown in Tables 1 and 2. Also, a sensitivity analysis on the frictional performance of joints was undertaken. The joint friction angle was varied from 14° to 36.8°. This was done to simulate potential joint degradation effects and/or possible water lubrication at the joint. Self-weight effects were assigned as gravitational load. At first, the model was brought into a state of equilibrium under its own weight (static gravity loads). Then, seismic loading was applied to the structure by means of increasing horizontal static-equivalent seismic loads/actions (non-linear static analysis). Horizontal displacements at the upper part of the colonnade (Point A in Figure 3) were recorded.

## 6. Response to static loading conditions

Figure 4 shows the displacement vectors in the colonnade when the joint friction angle is 15°. The displacement vectors clearly show the opening of the column joints in the lower storey, a result of the thrust of the segmented beam's central blocks. The block at the central bay slides down






(a) (b)

**Figure 2: Different construction methods for the trabeation of the Forum in Pompeii: (a) Flat arch (segmented beam); (b) solid long beam.**

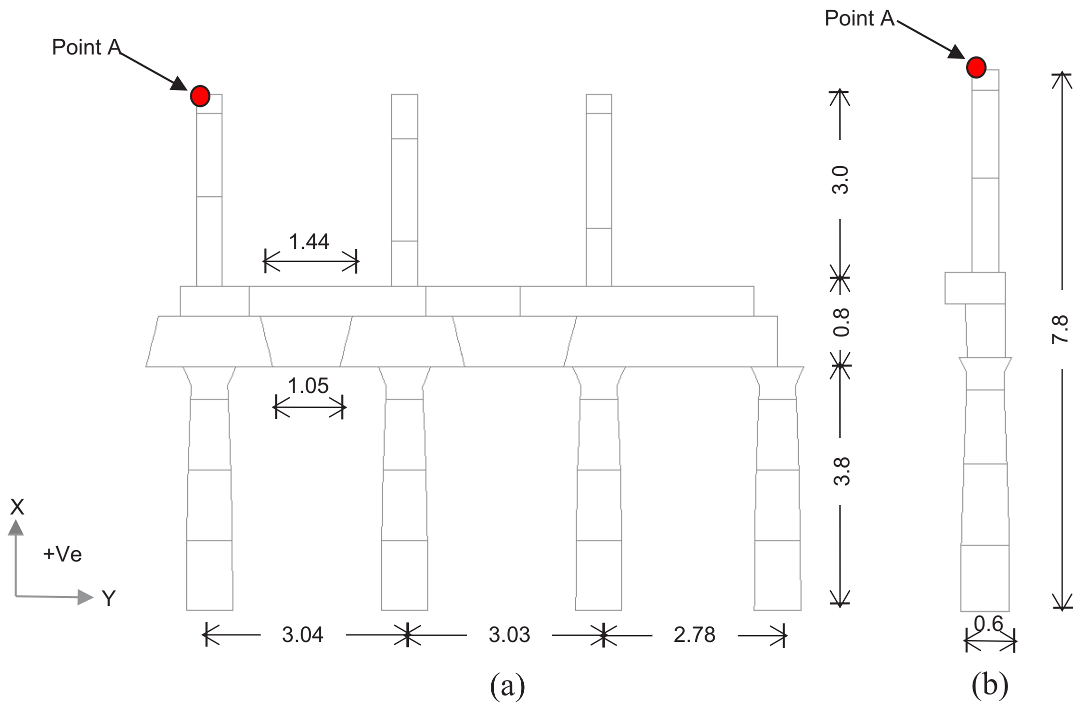



**Figure 3: Geometry of the two-storey colonnade of the Forum in Pompeii: (a) Front view; (b) side view.**

**Table 1: Properties of limestone blocks.**

| Unit Weight *d* (kg/m³) | Young Modulus *E* (GPa) | Shear Modulus *G* (GPa) | Bulk Modulus *K* (GPa) | Poisson's Ratio *v* (-) |
|---|---|---|---|---|
| 2680 | 40 | 16 | 27 | 0.25 |

**Table 2: Properties of joint interfaces (Angrisani et al., 2010; Kastenmeier et al., 2010).**

| Normal Stiffness $JK_n$ (GPa/m) | Shear Stiffness $JK_s$ (GPa/m) | Friction angle $J_{fric}$ (°) |
|---|---|---|
| 4 | 2 | 14–36.8 |

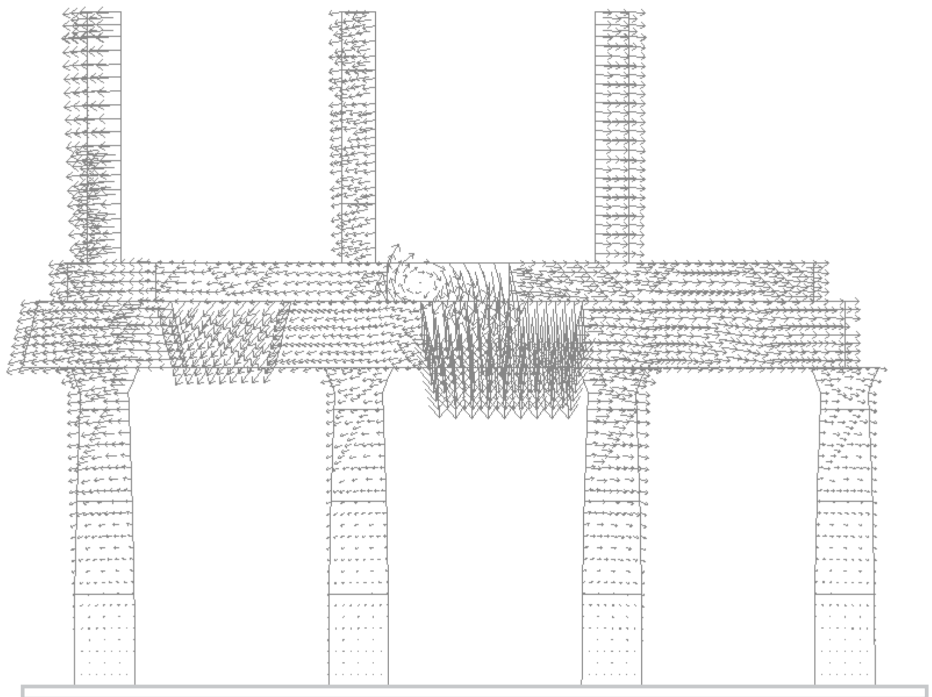


**Figure 4: Displacement vectors ($J_{fric}$ = 15°).**

and pushes the contiguous blocks apart. Due to the sliding, the upper block tends to rotate and further pushes down the former. The upper storey columns follow the sliding of the blocks of the beam. In this context, stability is mainly a geometrical problem since the loss of support is the reason for collapse. On the other hand, the stress level at the limestone blocks is negligible.

Figure 5 shows the location of opening joints for different values of friction angle. For a joint friction angle equal to 36.8°, the maximum opening at the joints is in the order of 0.085 mm (Figure 5a). For a joint friction angle equal to 15°, the maximum opening at the joints is 0.37 mm (Figure 5b). When the joint friction angle is 14° or below, equilibrium of the colonnade cannot be guaranteed, and failure occurs (Figure 5c). This is due to the fact that sliding of the short blocks over the two columns on the left is not able to hold in place the central block which is flowing down. It is worth mentioning that the angle of the inclined faces of the segmented blocks in the trabeation is also 14°.

Figures 6a and 6b show the principal stress distributions when the joint friction angle is equal to 36.8° and 15°, respectively. In Figure 6, stress intensities reduce as the joint friction angle increases. When the joint friction angle is equal to 36.8°, the maximum principal compressive stress in the structure is 0.27 MPa and the minimum principal tensile stress is 0.11 MPa. Similarly, when the joint friction angle is equal to 15°, the maximum principal compressive stress in the structure is 0.54 MPa and the minimum principal tensile stress is 0.24 MPa. Considering the continuous beam (left part of the structure), high tensile stresses develop at the bottom of the beam while the top of the overlapped elements is under compression. Conversely, the left layout provides a 'flat arch', hence almost negligible tensile and widespread compressive stresses are developed. Since joint degradation implies a reduction of mechanical properties, both joint opening and stresses tend to increase, further aggravating this phenomenon.

## 7. Nonlinear static (pushover) analysis

In order to evaluate the seismic vulnerability of the structure, a non-linear static analysis was performed. The analysis was undertaken under constant gravity loads and uniform monotonically increasing horizontal static-equivalent seismic loads (non-linear static analysis) until the structure was no longer in equilibrium (e.g., in-plane loss of stability due to sliding and/or rocking failure of blocks). Based

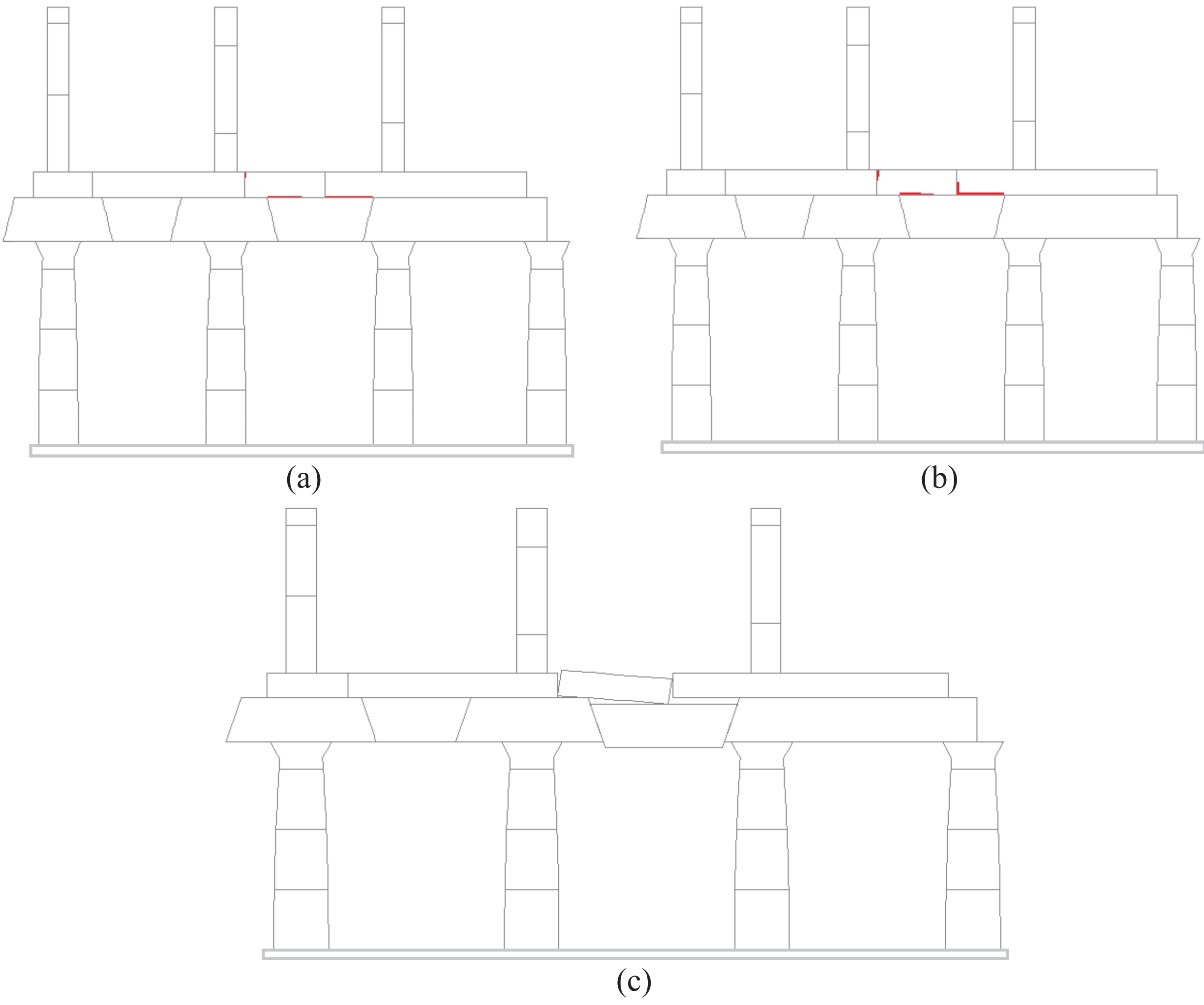



**Figure 5: Joint opening in the structure: (a) $J_{fric}$ = 36.8° (max joint opening 0.085 mm); $J_{fric}$ = 15° (max joint opening 0.37 mm); (c) $J_{fric}$ = 14° (collapse state).**

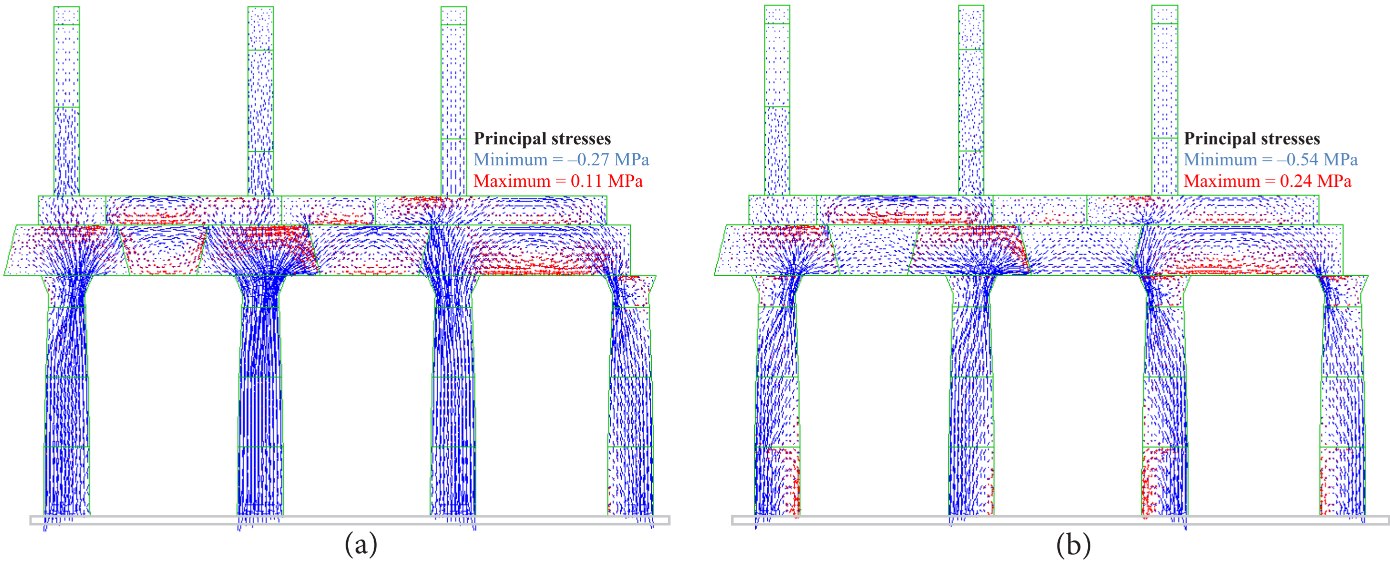



**Figure 6: Principal stress distributions (tensile: red; compression: blue) for different values of joint friction: (a) $J_{fric}$ = 36.8°; (b) $J_{fric}$ = 15°.**

on the ITACA seismic database (Luzi, 2008), peak ground accelerations were expressed in terms of probability of exceedance in a 50-year return period according to Eurocode 8–Part 1 (CEN, 2004). Preliminary modal analyses were undertaken, and the dynamic response of the colonnade in its elastic phase was obtained (see Lignola et al., 2015). The free vibration problem was analysed in the in-plane and out-of-plane directions, and the higher modes involved almost negligible participating mass. The first three modes involved translation in the longitudinal direction and the other three in the out-of-plane direction. Table 3 shows the natural frequencies and periods for the first six vibration modes of the colonnade in the in-plane and out-of-plane directions. Clearly, the natural period is longer for the out-of-plane direction, where the structure is slenderer, hence prone to overturning. Conversely, the first natural period in the in-plane direction is much shorter because the structure is much stiffer in this direction. Nevertheless, higher mode periods are almost comparable in both in-plane and out-of-plane directions. To improve the knowledge about the seismic vulnerability of the structure, it is crucial to include the nonlinear behaviour of the interfaces and especially their 'no tension' behaviour. The out-of-plane response of the colonnade is not investigated further in this study.

The seismic vulnerability was investigated according to Eurocode 8–Part 3 (CEN, 2006) by means of a nonlinear static (pushover) analysis under constant gravity loads and monotonically increasing horizontal loads at the location of the masses in the model. The relation between the base shear and the displacement at a control point (i.e., Point A in Figure 3) of the multi-degree-of-freedom system gives the 'capacity curve' of the single-degree-of-freedom (SDOF) system. Since the structure is not symmetric, both positive and negative lateral increasing static-equivalent seismic loads were considered. The structure was pushed (positive direction) and pulled (negative direction) up to failure, i.e., when loss of equilibrium was observed. The equivalent SDOF system was determined according to Eurocode 8–Part 1 (CEN, 2004) assuming a bilinear curve that provides the idealised elastic–perfectly plastic acceleration against displacement relationship.

**Table 3: Values of frequencies and periods for the first six in-plane and out-of-plane modes of vibration.**

| Mode | In-plane | | Out-of-plane | |
|---|---|---|---|---|
| | Frequency (Hz) | Period (sec) | Frequency (Hz) | Period (sec) |
| 1 | 18.1 | 0.055 | 7.4 | 0.135 |
| 2 | 25.3 | 0.040 | 27.4 | 0.036 |
| 3 | 25.5 | 0.039 | 71.5 | 0.014 |
| 4 | 27.6 | 0.036 | 104 | 0.010 |
| 5 | 104 | 0.010 | 159 | 0.006 |
| 6 | 111 | 0.009 | 202 | 0.005 |

Figures 7 shows the capacity curves of the SDOF systems in terms of acceleration and displacement. Each colour refers to a different friction angle. In addition, thick lines represent the actual capacity curve while thin lines represent the corresponding idealised bilinear curves. Similarly, the seismic demand is expressed in terms of an equivalent SDOF system. A spectral displacement demand represents a target displacement for the structure. For structures in the short-period range (i.e., lower than the upper corner period of the constant acceleration region of the elastic spectrum), the elastic spectral demand is increased, while for structures in the medium and long-period range, the target displacement is the displacement of the elastic response spectrum (CEN, 2004). Finally, the structure is considered safe when the displacement capacity exceeds the displacement demand. Assuming different friction angles for the joints, the behaviour is clearly different, and variations are evident in each direction.

Horizontal in-plane actions yield dissimilar behaviours depending on the friction angle of the joint. In the positive direction (left to right), low values of joint friction angle (i.e., 15°) yield a linear behaviour up to failure; for high values of joint friction angle (i.e., 36.8°), a short nonlinear plastic behaviour is observed having almost the same strength (Figure 7). Yielding, according to the idealised bilinear curve, occurs at an acceleration slightly lower than $a_g/g = 0.2$. In the negative direction (right to left), even though the strength is almost similar (i.e., when the friction angle is equal to 36.9°, the maximum acceleration is also close to $a_g/g = 0.2$), the nonlinear behaviour is remarkably different. In the case of a friction angle equal to 15°, yielding occurs at $a_g/g = 0.10$.

Figure 8 shows the pushover curves when the joint friction angle is equal to 26.6°. Dotted vertical lines represent the SDOF target displacements for different levels of ground motions (expressed in terms of return period and probability the ground motion will be exceeded in an interval of 50 years). The colonnade is able to withstand a design earthquake with a return period of 200 years or 22% exceedance probability in 50 years, both in the positive and negative directions, and this performance is provided while the structure responds in the elastic range (Figure 8). In this sense, the capacity curves overlap in the two directions in the elastic range. Conversely, in the negative direction, where the structure exhibits a ductile behaviour, the spectral displacement is properly amplified to account for the limited strength of the colonnade, hence considering its displacement capacity after yielding in the nonlinear field.

Figure 9 shows a highly magnified deformed shape, indicating the development of the failure mechanism; central blocks of the beam lose their support and sliding occurs resulting in the in-plane failure condition. In the positive direction, the mutual rocking and sliding of the blocks is counteracted by the long beam lying on two columns; the long beam represents a stiffer restraint compared to the negative direction where all the small blocks are less constrained given their size and the fact that each of them lies on a single column. The effect of seismic actions (return period of 200 years) at the peak (target) displacement is shown in Figure 10. From the point of view of stress levels, compression stresses are lower than 0.8 MPa (in all considered cases). Even though stresses exceed those under static conditions (Figure 6b), they are still lower than the expected material strength, and therefore do not cause particular concern. Similarly, tensile stresses are lower than 0.4 MPa. This means that no internal block failure is expected apart from sliding and rocking.

## 8. Conclusions

This article presented results from a numerical model developed using DEM to investigate the seismic vulnerability of a block-based frame of architectural heritage of the ancient city of Pompeii in Italy, and thus gain a better understanding of its overall structural behaviour.

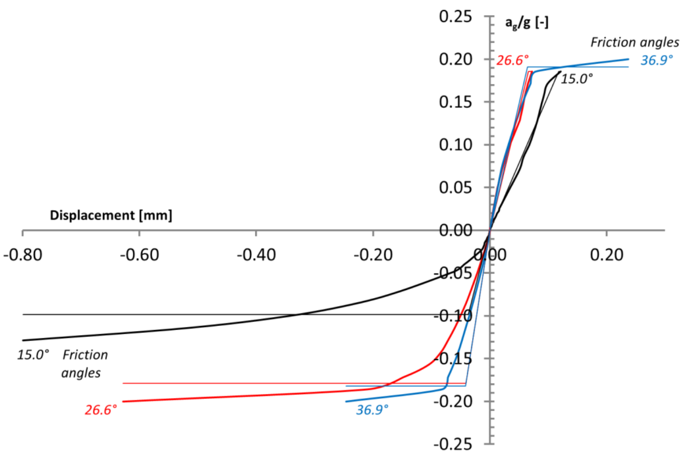



**Figure 7: Actual (thick) and bilinear (thin line) capacity curves derived from pushover analysis in the in-plane positive and negative directions for different values of joint friction angle.**

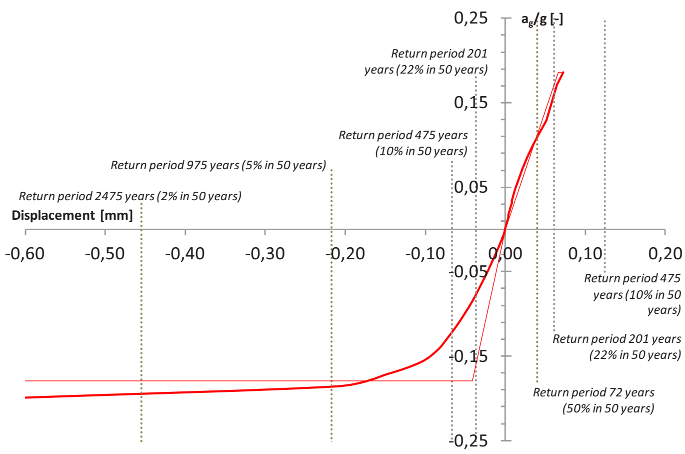



**Figure 8: Comparison of the actual and bilinear capacity curve (average $J_{fric}$ = 26.6°) with expected seismic demands (vertical dotted lines).**

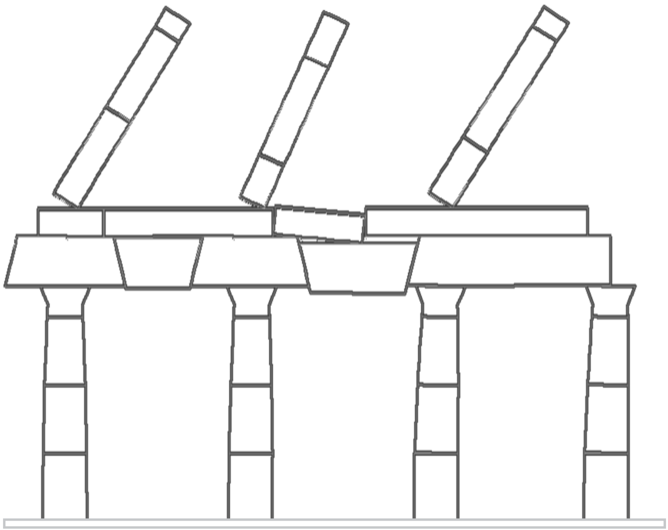


**Figure 9: Magnified (x200) deformed shape under seismic actions in the in-plane positive direction ($a_g$/g = 0.18).**

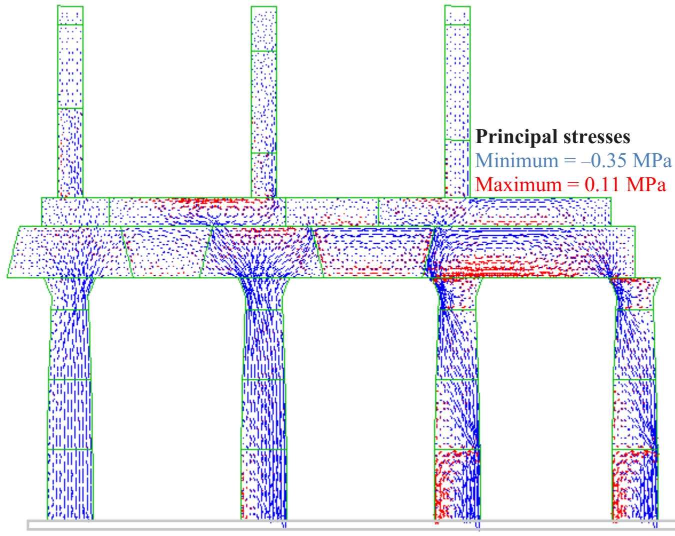



**Figure 10: Principal stress distributions (tensile: red, compression: blue) for $J_{fric}$ = 15°.**

The two-storey colonnade of the Forum in Pompeii was erected in the main square of the ancient town, made of white limestone. An 'innovative solution' was adopted for the construction of the trabeation. To avoid long-span beams over the columns, short segments were built up providing oppositely inclined pattern edges, i.e., a 'flat arch' configuration. It is believed that this solution was conceived to simplify construction phases and prevent lifting long-span heavy beams over the columns. The blocks are mutually supported over inclined surfaces inducing a horizontal thrust in the structure. Each block pushes over the other two contiguous blocks, and this load is counteracted. However, at the corners of the colonnade, where there is a lack of symmetry in the structural system, stability becomes critical. Such thrust could overturn a single extremity column. As a solution to this, the builders at that time used large blocks at the end of the colonnade allowing for the required horizontal thrust balance.

Joint openings are considered detrimental for the structure. Joint openings may lead to water leakage and lubrication of the potentially sliding planes. From the results of the analysis, it was found that if the joint friction angle is 14°, equilibrium of the structure cannot be achieved, and failure occurs. Also, the current condition of the partially collapsed colonnade presents higher vulnerability because the system of horizontal thrust is not perfectly balanced on one side, hence resulting in non-symmetric behaviour. From the results of the pushover analyses of this multi-drum colonnade, it was found that its capacity is adequate to withstand medium earthquakes expected in the area where it is built. It is worth mentioning that almost thirty years ago, in 1980, the structure successfully passed a natural test, withstanding with negligible damage an earthquake having an intensity comparable to an earthquake with a return period of 72 years or 50% probability of exceedance in 50 years. A potential strengthening intervention should involve improving the in-plane horizontal behaviour by balancing the horizontal thrust at the first column on the left-hand side.

## Acknowledgments

The author would like to thank IGI Global for allowing the partial reproduction of the book chapter 'Seismic Vulnerability of Ancient Colonnade: Two Story Colonnade of the Forum in Pompeii' (Sarhosis et al., 2015).

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# Impact of Induced Seismicity on Masonry Urban Infrastructure: The Case of Groningen, The Netherlands

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## 1. Introduction

Groningen is the largest gas field in Europe and the 10th largest worldwide. Due to the extensive gas extraction, induced earthquakes of relatively 'larger' magnitude have been recorded in the last decade. The building stock in the region comprises single- and two-storey unreinforced masonry (URM) houses constructed with no seismic considerations. The Groningen gas field has been exploited since 1963. However, an induced seismicity event in the field was first recorded in 1991 ($M_L$ 2.4). In subsequent years, there have been more than 1,300 registered small-magnitude earthquakes, the largest of which was of $M_L$ 3.6 in 2012 (Figure 1). In recent years, Groningen has been turned into the spearhead of research related to induced seismicity, as it is the most densely populated area worldwide with many induced earthquakes (Smyrou and Bal, 2019).

## 2. Liability and damage claim issues

Recursive induced earthquakes are often blamed for triggering structural damages in thousands of houses in Groningen. The liability of the exploiting company is related to the damages, and engineering firms and experts are asked to correlate the claimed damages with past earthquakes. Structures in the region present high vulnerability to lateral forces, soil properties are quite unfavourable for seismic resistance, and structural damages are present even without earthquakes. This situation creates a dispute, and the ambiguity of the 'damage vs earthquake' correlation is one of the main sources of the public unrest in the area until today. The damage claim procedure along with its social and political implications are discussed by Bal et al. (2019a).

## 3. Myths and fallacies in the Groningen earthquake problem

The main question, arising in the scientific community over the last years, is whether the existing state-of-the-art in the earthquake engineering discipline can be applied to regions in which induced seismicity events are taking place. Someone can interpret this question as to whether induced seismicity events can be called 'earthquakes' or are simply 'tremors'. Comparisons of such events are difficult to make, requiring understanding of the topic and correct interpretation of data. Although quick conclusions can be made that "*the Groningen earthquakes are different*", it is not possible to reach a definite answer thus far on whether natural and induced seismic events are different or similar. A discussion about the myths and fallacies around the Groningen earthquake problem is presented in Bal (2018)

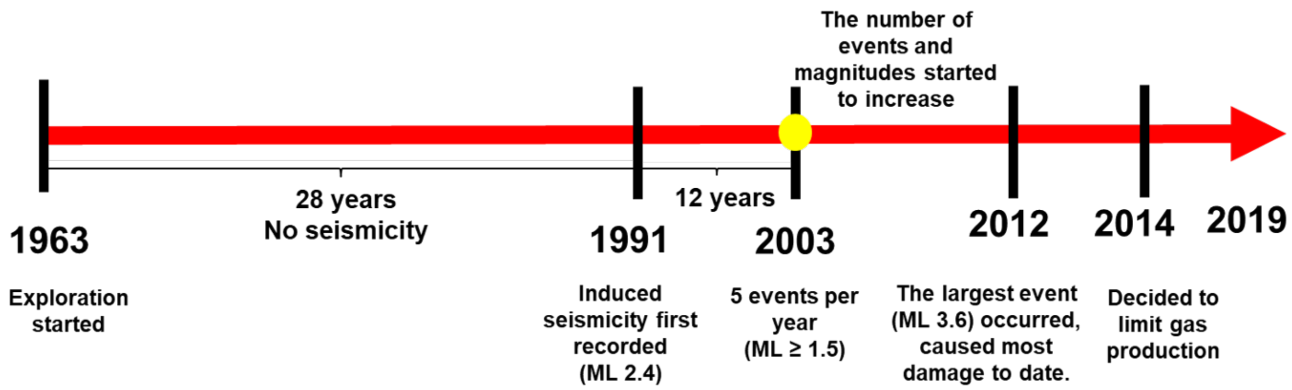


**Figure 1: Induced seismicity events in Groningen over time.**

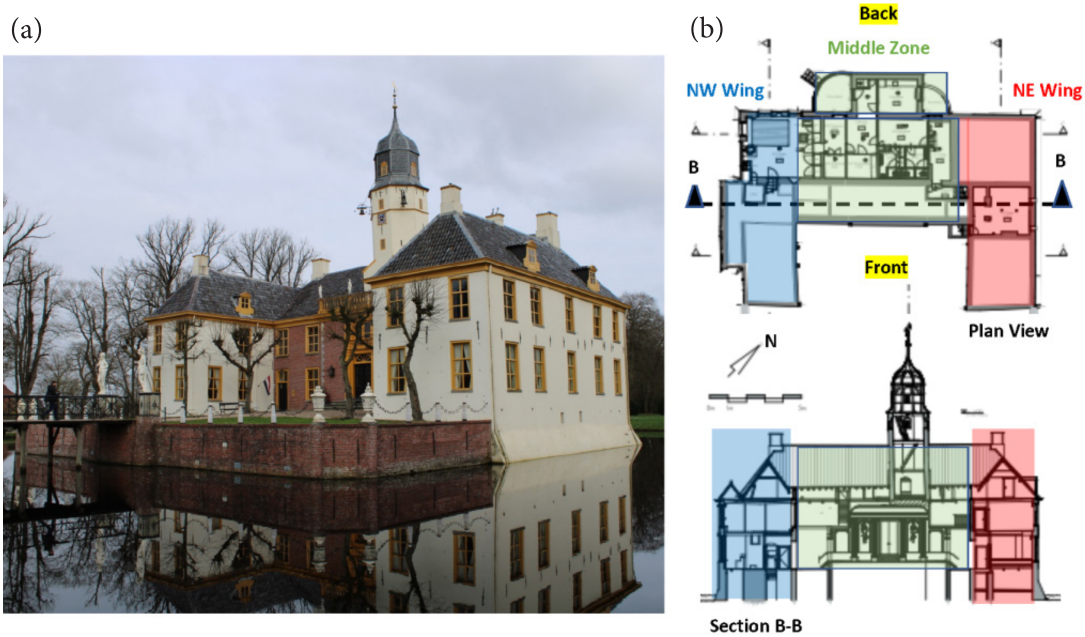


**Figure 2: Fraeylemaborg building: (a) Northeastern view; (b) plan view and cross-section.**

and Bal et al. (2018a; b).

## 4. Structural health monitoring – Fraeylemaborg

Despite the high concentration of historical buildings in the Groningen gas field along with their seismic vulnerability and past damages, there is only one historical masonry building in the region, i.e., the Fraeylemaborg, where a standard seismic structural health monitoring (SHM) system was developed and applied.

Fraeylemaborg (Figure 2) is the most emblematic historical building in the Groningen region. The structure was firstly built in 1300 and sits on an artificial island surrounded by water channels, rendering the problem of earthquake response even more complicated (Dais et al., 2019). During the past induced seismic activity, Fraeylemaborg suffered extensive damage. Two big restoration works took place recently at the end of 2015 and at the beginning of 2017 (Figure 3). Monitoring results and particularities in the case of induced earthquakes, as well as the usefulness and need of various monitoring systems for similar cases, are discussed by Bal et al. (2019b). 'Weak' soil properties dominate the structural response in the region. Thus, ground water monitoring and the interaction

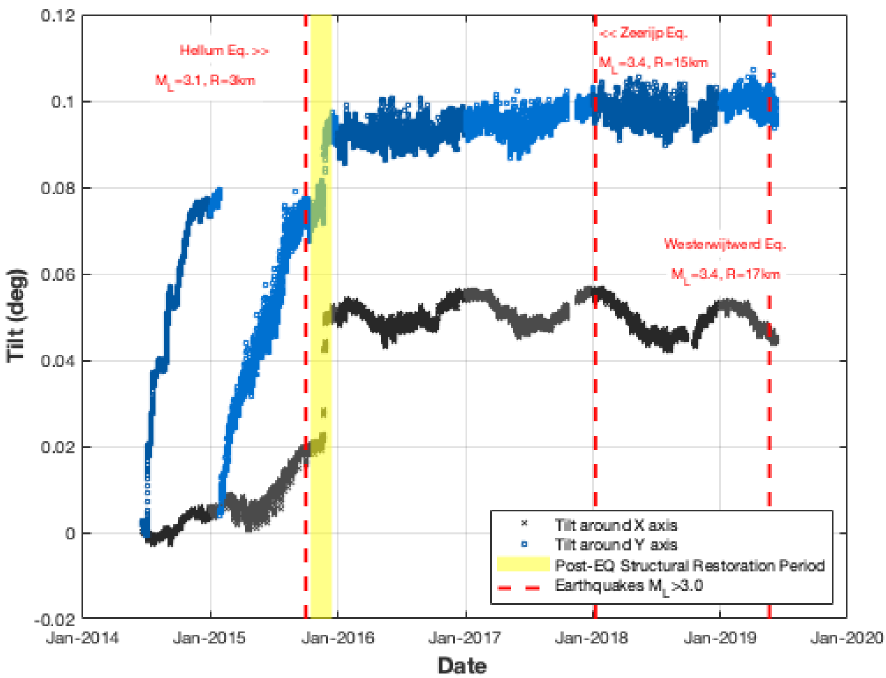


**Figure 3: Tilt meter measurements since 2014, together with significant earthquakes and restoration periods for Fraeylemaborg.**

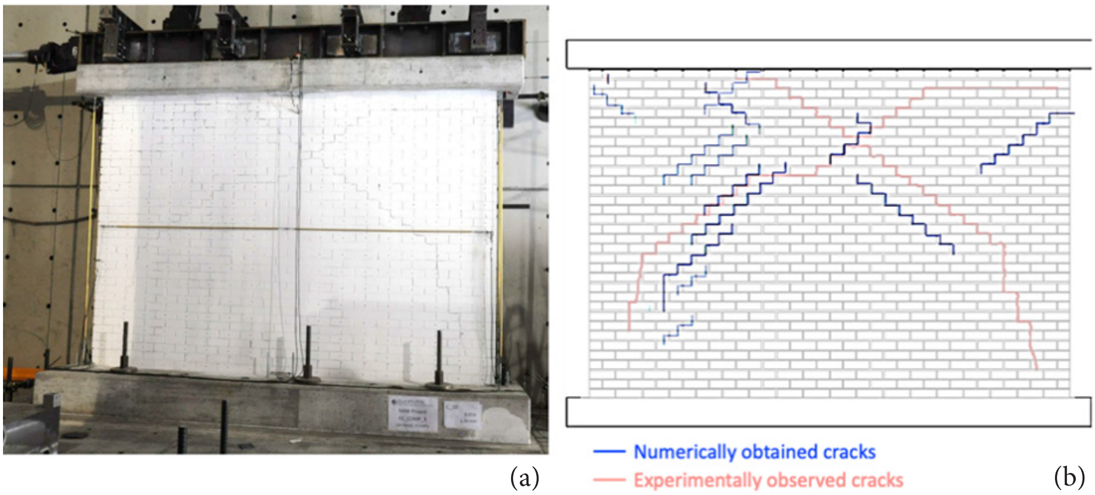


**Figure 4: (a) Calcium-silicate URM (resembling typical walls found in the Groningen region) tested under cyclic horizontal load, and crack pattern at 0.05% drift (Graziotti et al., 2018); (b) comparison of experimental against numerical results in terms of observed crack patterns.**

of soil movements with the structural response have also been scrutinized. Measurements produced from the SHM system of Fraeylemaborg are reported and accessible in Bal and Smyrou (2019a; b).

## 5. Damage accumulation and computational modelling

Evidence of cumulative damage on URM structures from available experimental and numerical data are reported in the literature (Sarhosis et al., 2019b). Specifically, available modelling tools were scrutinized in terms of their pros and cons in modelling cumulative damage in masonry. Previous efforts of modelling cumulative damage in single-degree-of-freedom (SDOF) models show that such simplified models are not able to accurately capture the accumulation of damage when sequential time-history analyses are performed, unless stiffness and strength degradation issues are properly addressed (Dais et al., 2017). The overall results of numerical models, such as SDOF residual displacement or floor lateral displacements, may be misleading in understanding damage accumulation. On the other hand, detailed discrete-element modelling was found to be more consistent in terms of providing insights into actual damage accumulation (Sarhosis et al., 2019b). Sarhosis et al. (2019a) attempted to quantify the cumulative damage of URM subjected to induced seismicity. A numerical model based on the discrete element method (DEM) was developed, able to represent masonry wall panels with and without openings, found in domestic houses in the Groningen gas field (Figures 4 and 5). The numerical model was

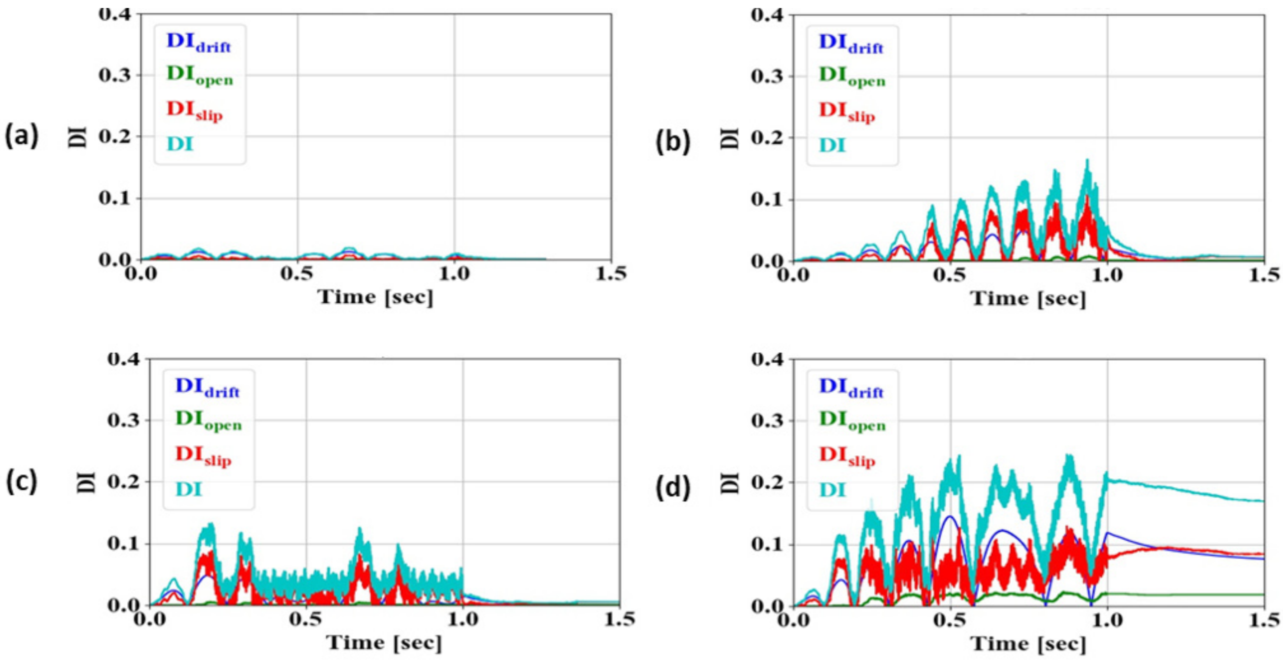


**Figure 5: Evolution of damage over time in masonry wall panel with symmetric opening for different acceleration amplitudes and frequencies: (a) 0.025g, 3 Hz; (b) 0.025g, 5 Hz; (c) 0.100 g, 3 Hz; (d) 0.100g, 5 Hz.**

validated against experimental data reported in the literature. A damage index (DI) equation was proposed which includes the following damage parameters: (*i*) the length of opened joints, (*ii*) the length of joints at shear sliding, and (*iii*) the relative drift at the top of the wall. Different masonry wall panels (in terms of geometrical characteristics) with openings were considered. The walls were subjected to harmonic loadings of different acceleration amplitude and frequency. From the analysis of the numerical results, it was shown that the frequency of a harmonic excitation is critical for the extent of damage in the wall panel. When the wall panel vibrates at a non-resonant frequency, damage is limited even under excitations of moderate acceleration amplitude. On the other hand, when the wall panel is harmonically excited at its natural frequency, there is potential for damage to occur even under low acceleration amplitudes. The obtained residual DI is expected to be relatively small when the frequency of the harmonic load diverges from the natural period of the wall.

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# Notable Earthquakes
# May 2019 – August 2019

## Reported by British Geological Survey

**Issued by: Davie Galloway, British Geological Survey, November 2019.**
**Non British Earthquake Data supplied by: United States Geological Survey.**

| Year | Day | Mon | Time UTC | Lat | Lon | Dep km | Magnitude ML | Mb | Mw | Location |
|---|---|---|---|---|---|---|---|---|---|---|
| 2019 | 04 | MAY | 00:19 | 51.16N | 0.24W | 2 | 2.5 | | | NEWDIGATE, SURREY |
| Felt Newdigate, Surrey, and in surrounding towns, villages and hamlets (4 EMS). | | | | | | | | | | |
| 2019 | 06 | MAY | 21:19 | 6.98S | 146.45E | 146 | | | 7.1 | PAPUA NEW GUINEA |
| 2019 | 14 | MAY | 12:58 | 4.05S | 152.60E | 10 | | | 7.6 | PAPUA NEW GUINEA |
| 2019 | 14 | MAY | 15:37 | 49.04N | 2.11W | 9 | 2.4 | | | JERSEY, CHANNEL ISLANDS |
| Felt Jersey (3 EMS). | | | | | | | | | | |
| 2019 | 17 | MAY | 02:56 | 52.58N | 1.02W | 3 | 1.6 | | | GREAT GLEN, LEICESTERSHIRE |
| 2019 | 26 | MAY | 07:41 | 5.81S | 75.27W | 122 | | | 8.0 | LORETO, NORTHERN PERU |
| Two people killed in Peru, 30 others injured (15 in Peru and 15 in Ecuador), and many buildings, streets and bridges were severely damaged in the epicentral area. | | | | | | | | | | |
| 2019 | 27 | MAY | 05:35 | 48.53N | 7.08W | 15 | 2.9 | | | CELTIC SEA |
| 2019 | 28 | MAY | 12:57 | 54.29N | 0.10W | 13 | 1.8 | | | FILEY, NORTH YORKSHIRE |
| 2019 | 30 | MAY | 09:03 | 13.20N | 89.31W | 57 | | | 6.6 | EL SALVADOR |
| 2019 | 08 | JUN | 06:11 | 56.08N | 3.94W | 7 | 1.6 | | | BANNOCKBURN, STIRLING |
| 2019 | 15 | JUN | 22:55 | 30.64S | 178.10W | 46 | | | 7.3 | KERMADEC ISLANDS |
| Small tsunami with a wave height of 5 cm was observed on Raoul Island. | | | | | | | | | | |
| 2019 | 17 | JUN | 14:55 | 28.41N | 104.93E | 6 | | | 5.8 | SICHUAN, SW CHINA |
| Thirteen people killed, at least 250 others injured, and over 150,000 buildings and houses were either damaged or destroyed in Changning and Gongxian. | | | | | | | | | | |
| 2019 | 18 | JUN | 13:22 | 38.64N | 139.48E | 12 | | | 6.4 | HONSHU, JAPAN |
| At least 36 people injured and over 100 homes damaged on Honshu, and a 10 cm tsunami was observed at Niigata. | | | | | | | | | | |
| 2019 | 18 | JUN | 19:53 | 53.29N | 4.91W | 10 | 1.7 | | | IRISH SEA |
| 2019 | 22 | JUN | 14:51 | 55.51N | 1.28W | 10 | 1.8 | | | CENTRAL NORTH SEA |
| 2019 | 24 | JUN | 02:53 | 6.41S | 129.17E | 212 | | | 7.3 | BANDA SEA |
| 2019 | 30 | JUN | 23:22 | 59.27N | 1.49E | 14 | 2.7 | | | NORTHERN NORTH SEA |
| 2019 | 01 | JUL | 22:16 | 56.37N | 5.13W | 7 | 1.8 | | | KILCHRENAN, ARGYLL & BUTE |
| Felt Kilchrenan, Kilmelford, Fasnacloich and Lismore (3 EMS). | | | | | | | | | | |
| 2019 | 04 | JUL | 07:29 | 52.19N | 3.01W | 12 | 1.5 | | | KINGTON, HEREFORDSHIRE |
| 2019 | 04 | JUL | 17:33 | 35.71N | 117.50W | 11 | | | 6.4 | RIDGECREST, CALIFORNIA |
| Twenty people injured and several buildings damaged in the Ridgecrest-Trona area. | | | | | | | | | | |

| | | | Time | | | Dep | Magnitude | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Year | Day | Mon | UTC | Lat | Lon | km | ML | Mb | Mw | Location |
| 2019 | 06 | JUL | 03:19 | 35.77N | 117.60W | 8 | | | 7.1 | RIDGECREST, CALIFORNIA |
| Five people injured and over 50 homes structurally damaged in the Ridgecrest-Trona area. Many cracks and landslides occurred on California State Route 178. Damage, from this earthquake and the sequence of earthquakes to occur in the region during July 2019, estimated in excess of $US100 million. | | | | | | | | | | |
| 2019 | 07 | JUL | 15:08 | 0.51N | 126.19E | 35 | | | 6.9 | MOLUCCA SEA |
| 2019 | 11 | JUL | 10:52 | 54.00N | 3.19W | 3 | 2.4 | | | IRISH SEA |
| 2019 | 14 | JUL | 05:39 | 18.22S | 120.36E | 10 | | | 6.6 | OFF WESTERN AUSTRALIA |
| 2019 | 14 | JUL | 09:10 | 0.59S | 128.03E | 19 | | | 7.2 | HALMAHERA, INDONESIA |
| Fourteen people killed, 129 others injured and over 2,700 homes damaged on Halmahera. A tsunami with a wave height of 20 cm observed at Labuha. Damage estimated at $US16 million. | | | | | | | | | | |
| 2019 | 27 | JUL | 00:04 | 53.83N | 1.68W | 12 | 1.6 | | | CALVERLEY, WEST YORKSHIRE |
| 2019 | 31 | JUL | 15:02 | 16.20S | 168.00E | 181 | | | 6.6 | VANUATU |
| 2019 | 01 | AUG | 18:28 | 34.24S | 72.31W | 25 | | | 6.8 | O'HIGGINS, CHILE |
| 2019 | 02 | AUG | 12:03 | 7.28S | 104.79E | 49 | | | 6.9 | OFF JAVA, INDONESIA |
| Eight people killed, several others injured, and over 500 buildings damaged in West Java. | | | | | | | | | | |
| 2019 | 05 | AUG | 19:55 | 61.29N | 4.26E | 24 | 3.0 | | | NORWEGIAN SEA |
| 2019 | 08 | AUG | 16:52 | 50.08N | 5.18W | 6 | 2.3 | | | HELSTON, CORNWALL |
| Felt in towns, villages and hamlets within approximately 25 km of the epicentre with the majority coming from residents in Helston, Porthleven, Constantine, Falmouth, Breage, Wendron, Mullion and Penryn (4 EMS). | | | | | | | | | | |
| 2019 | 17 | AUG | 18:49 | 54.96N | 1.63E | 18 | 1.7 | | | CENTRAL NORTH SEA |
| 2019 | 21 | AUG | 19:46 | 53.79N | 2.96W | 3 | 1.6 | | | BLACKPOOL, LANCASHIRE |
| Felt Westby, Weeton, Wrea Green, Blackpool and Lytham St Annes (3 EMS). | | | | | | | | | | |
| 2019 | 22 | AUG | 12:48 | 52.83N | 2.62W | 7 | 1.9 | | | HODNET, SHROPSHIRE |
| 2019 | 22 | AUG | 15:23 | 53.79N | 2.96W | 2 | 1.0 | | | BLACKPOOL, LANCASHIRE |
| Felt Westby (2 EMS). | | | | | | | | | | |
| 2019 | 23 | AUG | 23:46 | 53.84N | 1.02W | 3 | 1.7 | | | RICALL, NORTH YORKSHIRE |
| 2019 | 24 | AUG | 22:01 | 53.79N | 2.96W | 2 | 2.1 | | | BLACKPOOL, LANCASHIRE |
| Felt Westby, Weeton, Great Plumpton, Peel, Blackpool, Wesham, Lytham St Annes and surrounding areas (4 EMS). | | | | | | | | | | |
| 2019 | 26 | AUG | 07:30 | 53.79N | 2.96W | 2 | 2.9 | | | BLACKPOOL, LANCASHIRE |
| Felt Westby, Weeton, Great Plumpton, Peel, Blackpool, Wesham, Lytham St Annes and surrounding areas (6 EMS). | | | | | | | | | | |
| 2019 | 26 | AUG | 21:18 | 53.79N | 2.96W | 3 | 1.0 | | | BLACKPOOL, LANCASHIRE |
| Felt Westby (2 EMS). | | | | | | | | | | |
| 2019 | 27 | AUG | 06:55 | 53.79N | 2.96W | 2 | 0.5 | | | BLACKPOOL, LANCASHIRE |
| Felt Westby (2 EMS). | | | | | | | | | | |
| 2019 | 27 | AUG | 23:55 | 60.22S | 26.58W | 16 | | | 6.6 | SOUTH SANDWICH ISLANDS |

# Forthcoming Events

## Evening Lectures

**Soil–Structure Interaction and Optimum Seismic Design of Onshore and Offshore Energy Projects**
Prodromos Psarropoulos
29 January 2020 (6:00 pm) at the Institution of Civil Engineers, London

### Synopsis

Since society demands increased availability and reliability of energy supply, together with improved environmental standards, the structural design of any onshore or offshore energy project (including its foundation) may be very demanding, depending on the circumstances. It is evident that in the case of long energy projects that traverse remote regions with extreme terrains and/or seabeds such as a gas pipeline or a cable, the design may be more challenging due to the variety of geotechnical conditions and the potential geohazards along the routing. Nevertheless, in areas that are characterized by moderate or high seismicity the design of energy projects may be more complicated due to the various types of seismic loading. The seismic loading may be either dynamic due to the inertial forces developed on the mass of the structure(s) and/or quasi-static due to the permanent ground deformations (PGDs) caused by various earthquake-related geohazards, such as active-fault ruptures, slope instabilities, and soil liquefaction phenomena. The current presentation tries through case studies to shed some light on these interesting issues of geotechnical earthquake engineering from a structural and a geotechnical perspective. The first part of the presentation focuses on the impact of local site conditions (i.e., soil stratigraphy, bedrock geomorphology, and/or surface topography) on the ground surface motion that will dominate the dynamic structural response. In the second part, emphasis is given on the quantitative assessment of the earthquake-related geohazards and the realistic estimation of the PGDs that will actually determine the soil–structure interaction and the structural response/distress. Finally, the third part of the presentation is devoted to remote sensing and early-warning systems that are required for the safe operation of energy projects.

### Prodromos Psarropoulos

Dr Prodromos Psarropoulos is a Structural and Geotechnical Engineer with a balanced scientific and professional experience in the analysis and design of various structures and geostructures for almost 25 years. After his PhD in Geotechnical Earthquake Engineering at the National Technical University of Athens (NTUA), he conducted advanced research in various institutes in Greece and Italy, while he has been an adjunct Associate Professor of Geophysics & Earthquake Engineering in the Department of Infrastructure Engineering of the Hellenic Air Force Academy. In parallel, he has been involved in the design and construction of various challenging engineering projects in Greece and abroad. His expertise is in geotechnics, soil dynamics and earthquake engineering, mainly including: (a) problems of static and dynamic soil–structure interaction (regarding foundations, retaining structures, pipelines, etc.), (b) static and seismic stability assessment of dams, slopes and embankments, and (c) numerical simulation of dynamic soil response (i.e., local site effects and microzonation studies). Currently, he is teaching courses of geotechnical engineering and offshore engineering in the School of Rural & Surveying Engineering at NTUA, while he has been a lead member of the team of experts for the quantitative geohazard assessment and the seismic design of the upgrade of the main oil-refinery in Greece and two major high-pressure gas pipelines in south-east Europe (IGI–Poseidon and TAP).

### Further information

This evening meeting is organised by SECED and chaired by Dr Stavroula Kontoe (Imperial College London). Non-members of the society are welcome to attend. Attendance at this meeting is free. Seats are allocated on a first come, first served basis. Tea, coffee and biscuits will be served from 5:30 to 6:00 pm.

For up-to-date details and further information on events organised by SECED, visit the SECED website or contact Shelly-Ann Russell (020 7665 2147, societyevents@ice.org.uk)