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UNIVERSITY OF SPLIT

**FACULTY OF CIVIL ENGINEERING,
ARCHITECTURE AND GEODESY**

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**The effect of some parameters on behaviour and
bearing capacity of multi-drum stone columns under
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D i s s e r t a t i o n

Split, 2019.

Ante Buzov, mag.ing.aedif.

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This work I dedicate to my family.

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The effect of some parameters on behaviour and bearing capacity of multi-drum stone columns under static load and earthquake

Abstract:

In this thesis, the effect of several important parameters on the behaviour and bearing capacity of multi-drum stone columns loaded by short-term static loads and earthquakes was investigated. These parameters have not been investigated to date or have been investigated only sporadically. All of the experiments were conducted on small-scale column models using modern equipment for testing and measuring. Dynamic testing of the models was performed using an established earthquake platform. The effects of the column block number and joint type were investigated, as well as several parameters for the bolts used in the column joints (material, length, bolt diameter and the diameter of the bolt hole). Static tests were conducted separately for centric compressive loads and for bending loads with longitudinal compression. During the dynamic tests, the samples were exposed to the acceleration of three different earthquake types with a successive increase in acceleration to failure. During the action of all loads, the characteristic displacements, accelerations and strains of the model were recorded. The motion of the model was recorded with a precision camera.

The aim of this research was to contribute to the development of science in the subject area, which has been accomplished through the publication of five scientific papers in relevant worldwide journals. The purpose of this research is to provide conclusions regarding the investigated effects of important multi-drum stone column parameters, and the results should find practical application in the restoration/strengthening of existing multi-drum stone columns and the creation of new multi-drum stone columns. The most important conclusions of the research are listed below.

Increasing the number of blocks (joints) in the multi-drum column reduces the column's stiffness and increases its deformability. The consequence of this is a significant reduction in the column's bearing capacity for static loads, as well as for longer duration earthquakes, which entered great energy in the construction. For impact-type earthquakes, increasing the number of blocks in a column can result in a greater bearing capacity because the column stiffness is reduced, and smaller earthquake forces are generated in the column.

Soft joints (dry, joints made from stone powder and weaker mortar), compared to rigid joints and single-block column, result in a softer column with a lower bearing capacity for static loads and long lasting earthquakes with greater generated energy. However, for impact-type earthquakes, column with soft joints can have a greater bearing capacity than columns with rigid joints or single-block columns.

Bolts in multi-drum stone columns significantly contribute to their bearing capacity during earthquakes. The bolt material has no major impact on the column bearing capacity during earthquakes if the column collapse is predominantly followed by bending and less by shear, i.e., if the failure is caused by the bolts pulling out and not by shear failure.

The diameter of the bolt has no major effect on the column bearing capacity when it is loaded with smaller transverse forces. In that case, a thick bolt can be unfavourable for some earthquakes, as a thick bolt increases the stiffness of the column, and pull on it larger earthquake forces. In this case, the column failure is followed by the

bolt pulling out but not by its failure. When the column is subjected to large shear with respect to bending, increasing the bolt diameter increases the column capacity. The bolt length does not have a greater effect on the column bearing capacity when the bolt has no adhesion with the block, i.e., when the bolt cannot withstand tensile forces. Increasing the bolt hole diameter in relation to the bolt diameter and having a bolt that cannot withstand tensile forces decreases the column bearing capacity during an earthquake.

With the same maximum amplitude, the type of earthquake (duration and predominant period) has a considerable effect on all column parameters considered in this research. Therefore, in practice, it is crucial to know what type of earthquake can be expected at the considered location.

Keywords: multi-drum stone column, experiment, block height, joint type, bolts parameters, bearing capacity, static load, earthquake

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Utjecaj nekih parametara na ponašanje i graničnu nosivost višedijelnih kamenih stupova pri statičkom opterećenju i potresu

Sažetak:

U okviru teme ove doktorske disertacije, istražen je utjecaj nekoliko važnih parametara na ponašanje i nosivost višedijelnih kamenih stupova opterećenih kratkotrajnim statičkim opterećenjima i potresom, a koja do sada nisu bila istraživana ili su istraživana samo sporadično. Sva istraživanja su provedena eksperimentalno na umanjenim modelima stupova, koristeći suvremenu opremu za ispitivanja i mjerenja. Dinamička ispitivanja modela su provedena pomoću provjerene potresne platforme. Između ostalog, istražen je utjecaj broja blokova i tipa sljubnica u stupu, te više parametara trnova u sljubnicama stupa (materijal, dužina i promjer trna, te promjer rupe za trn). Statička ispitivanja su provedena zasebno za centrično tlačno opterećenje i zasebno za savijanje s uzdužnom tlačnom silom. Kod dinamičkih testova, uzorci su bili izloženi ubrzanju triju različitih tipova potresa, sa sukcesivnim povećanjem ubrzanja do njihova sloma. Pri djelovanju svih opterećenja, praćeni su karakteristični pomaci, ubrzanja i deformacije modela, te snimano gibanje modela preciznom kamerom.

Cilj istraživanja je doprinos razvoju znanosti u predmetnom području, koji je ostvaren objavom pet znanstvenih radova u relevantnim svjetskim časopisima. Svrha istraživanja je donošenje zaključaka u pogledu istraženih utjecaja razmatranih važnih parametara višedijelnog kamenog stupa, koji bi trebali naći svoju primjenu u obnovi/ojačanju postojećih i izradi novih višedijelnih kamenih stupova u praksi. Najvažniji zaključci istraživanja su niže navedeni.

S povećanjem broja blokova (sljubnica) u višedijelnom stupu, smanjuje se njegova krutost i povećava njegova deformabilnost. Posljedica toga je značajno smanjenje njegove nosivosti za statička opterećenja, kao i za potrese dužeg trajanja koji unose veliku energiju u konstrukciju. Za potrese kratkog udarnog djelovanja, povećanje broja blokova u stupu može rezultirati njegovom većom nosivosti jer se smanjenjem krutosti stupa u njemu generiraju manje potresne sile.

Meke sljubnice (suha, od kamenog brašna i slabijeg morta), u odnosu na krute sljubnice i stup iz jednog bloka, također rezultiraju mekšim stupom i njegovom značajno manjom nosivosti za statička opterećenja i potrese dugog trajanja s većom generiranom energijom. Međutim, za potrese kratkog udarnog djelovanja, stup s mekom sljubnicom može imati veću nosivost nego stup s krutim sljubnicama ili iz jednog bloka po visini.

Trnovi u sljubnicama višedijelnog kamenog stupa značajno doprinose njegovoj nosivosti pri potresu. Vrsta materijala za trn nema većeg utjecaja na nosivost stupa pri potresu ako je slom praćen dominantno savijanjem i manje posmikom, tj. ukoliko dolazi do čupanja trnova a ne do njihovog posmičnog popuštanja.

Promjer trna nema većeg utjecaja na nosivost stupa za slučajeve kada je opterećen manjim poprečnim silama. U tom slučaju debeli trn može biti nepovoljan za neke potrese jer se njime povećava krutost stupa i na njega navlače veće potresne sile, a pri tome je slom stupa praćen izvlačenjem trna a ne njegovim posmičnim otkazivanjem. Kada je stup opterećen velikim utjecajem posmika u odnosu na savijanje, povećanje promjera trna će rezultirati većom nosivosti stupa. Dužina trna nema većeg utjecaja na nosivost stupa u slučaju kada on nema prionjivost s blokom, tj. kada ne može nositi vlačne sile. S povećanjem promjera rupe za trn u odnosu na promjer trna i kada trn ne može nositi vlak, smanjuje se nosivost stupa pri potresu.

Uz istu maksimalnu amplitudu, tip potresa (trajanje i dominantni period) ima vrlo veliki utjecaj na sve razmatrane parametre u ovom istraživanju. Zato je u praksi ključno znati kakvi se tipovi potresa mogu očekivati na razmatranoj lokaciji.

Ključne riječi: višedijelni kameni stup, eksperiment, visina bloka, tip sljubnice, parametri trnova, nosivost, statičko opterećenje, potres

List of published papers and my contributions

The thesis is based on five papers (see Appendix), referred by Roman numerals in the text of the thesis:

- [I] *Buzov, A., Radnić, J., Grgić, N., Baloević, G.: Effect of the drum height on the bearing capacity of composite multi-drum column under static load. Composites Part B: Engineering 148, 243-251 (2018).*
- [II] *Buzov, A., Radnić, J., Grgić, N., Baloević, G.: Effect of the drum height on the seismic behaviour of a free-standing multi-drum column. Advances in Materials Science and Engineering (2018). <https://doi.org/10.1155/2018/5729068>.*
- [III] *Buzov, A., Radnić, J., Grgić, N., Baloević, G.: Effect of the joint type on the bearing capacity of a multi-drum column under static load. International Journal of Architectural Heritage 12, 137-152 (2018).*
- [IV] *Buzov, A., Radnić, J., Grgić, N., Baloević, G.: Effect of the joint type on the seismic behaviour of a free-standing multi-drum column. Construction and Building Materials, revised manuscript submitted.*
- [V] *Buzov, A., Radnić, J., Grgić, N.: Effects of several bolt parameters on the bearing capacity of a composite multi-drum stone column under an earthquake. Composites Part B: Engineering 162, 250-258 (2019).*

In all above mentioned papers, my contributions are listed below:

- Participation in the design of the research and testing programmes.
- Making all the small-scale testing models.
- Participation in the realization of all experimental tests (e.g., model installation, sensor installation, measurements, and data collection).
- Design of drawings, diagrams and tables based on the experimental results.
- Participation in the analysis of the tests results.
- Participation in the development of the research findings/conclusions.
- Participation in composing the paper for journal publication.

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1. Introduction

Historical stone buildings, especially important and monumental ones, represent a small number of today's buildings worldwide. The most significant stone buildings in the world are located only in a small number of states and are related to certain historical periods. Numerous historic stone buildings have an invaluable cultural and architectural importance, many of which are protected monuments and architectural heritage. The most remarkable and impressive elements of stone buildings are usually the stone columns.

Free-standing or top-connected stone columns date back to the pre-historic period of humanity. Columns in historic stone structures are important, not only for their structures but often for their design elements. The characterization of particular styles in the history of architecture is very clearly manifested in the design of columns, especially their capitals and bases.

As a structural element in historical stone buildings, columns are less present as the only supporting elements and are typical in combination with stone walls. Columns can be free-standing (console) or interconnected by beams and arches (vaults).

In terms of the structural importance in stone building, columns can be secondary or extremely important and the only bearing elements.

Columns can be smaller in height in the case when they are made from a single block or larger in height in the case when they are made from multiple blocks (i.e., multi-drum columns).

Multi-drum stone columns are complex composite structures and are a specific form of masonry structure. These columns are formed of stone blocks with different properties interconnected in joints in different manners. Specifically, the joints may be dry or filled with such materials as stone powder or various mortars. Possible bolts in the joints make multi-drum column structures even more complicated. Multi-drum stone columns are a complex assembly of different materials, whereby the manner of building has a great effect on the column's ultimate bearing capacity.

The behaviour and ultimate bearing capacity of multi-drum stone columns are affected by many important parameters, such as the following:

- Geometry, slenderness and boundary conditions of the column.
- Solutions of the top (capital) and the bottom of the column.
- Geometry, material properties, precision of stone block manufacturing, and precision of the blocks connecting surface treatment.

- Type and properties of the material in the joints between the stone blocks.
- Parameters of the bolts in the joints.
- Types of load (static, dynamic, impact) and their characteristics.
- Interaction with other construction bearing elements (e.g., walls, beams, vaults, and foundations).

Investigations related to the behaviour and bearing capacity of stone columns under load are still very rare. The majority of researchers are the Greeks and the Italians, which is logical because at present, the most numerous and monumental buildings are from the period of Athens and Rome. Other researchers are mainly from the Mediterranean area, where stone buildings are also a historic tradition and often of exceptional beauty and importance. It is therefore logical that there is also interest along the Croatian coast, where there are numerous famous and protected stone structures from Roman times, and in this area, there is a need for studies related to stone columns.

As the research subject of this dissertation is multi-drum stone columns, some better known and more important studies related to such columns are shown below, but only briefly because they are not or are only sporadically related to the issue investigated in this work. Most of the studies are numerical, and a smaller number are experimental. A smaller number of studies refer to the static loads on the columns, and a larger number refer to earthquakes.

Contemporary numerical studies of multi-element column earthquake behaviour are mainly based on discrete element methods. Using this method, Minafo et al. [M.3] in an analytical-numerical study analysed the behaviour of multi-drum columns using a sinus type dynamic excitation. Pulatsu et al. [P1.6] investigated the non-linear behaviour of ancient multi-drum stone columns in the Mediterranean area. Among other things, these authors concluded that irregularities in stone free-standing columns have a significant effect on the bearing capacity at the lateral load. Pitilakis et al. [P.11] numerically investigated the structural characteristics and the boundary stability of monolithic and multi-drum classical columns with reference to the Ionic Column on the Acropolis of Lindos (Rodos, Greece). Numerical model control was performed through a series of tests of a multi-drum column scaled model on an earthquake platform.

Sorace et al. [S.3] analysed the seismic response of the freestanding marble column, including the rocking and sliding effects. These authors concluded that the column had a high seismic vulnerability, as identified by severe damage of the design at a basic earthquake level, and an overturning-related, near-collapse response at the maximum earthquake level applies. In a parametric seismic response study of ancient rocking columns, Kawadias et al. [K.1] showed that

the velocity and frequency of the ground motion intensity exhibited a strong correlation with the seismic behaviour of free-standing rocking columns. In a seismic and restoration assessment of monumental masonry structures, Asteris et al. [A.8] presented a new computational stochastic framework for the earthquake-resistant design of a masonry structural systems that was based on the probabilistic behaviour of crucial parameters, such as the material strength and seismic characteristics, and this model utilized fragility analysis based on different failure criteria for the masonry material.

Sarhosis et al. [S.1] utilized a discrete element method to study the static and dynamic behaviour of a typical section of the two-storey colonnade of the Forum in Pompeii. The construction method used in ancient times, such as multi-block segmented trabeations and a solid block beam, was shown to have a significant impact on the mechanical response of the structure. Pappas et al. [P.9] describes the creation of a seismic vulnerability assessment form for free-standing columns that is based on discrete element method simulations, and this assessment can be used for rapid, preliminary seismic vulnerability assessment without the need for further sophisticated calculations. These authors analysed four ancient columns that had survived earthquakes of known acceleration amplitude to apply, test and validate the proposed assessment form.

By modelling the dynamics of masonry structures, Lemons [L.1] provided a realistic representation of the behaviour of many types of masonry structures under static and dynamic loads. Although the emphasis is on the seismic behaviour of block stone masonry, the treatment is general to cover other types of masonry. Makris and Vassiliou [M.1] investigated the planar rocking response and provided a stability analysis of an array of free-standing columns capped with a freely supported rigid beam with the goal of creating a seismic design concept allowing framing systems to uplift and rock along their plane to limit bending moments and shear forces. Lignola et al. [L.2] analysed the influence of short segments in the trabeation with opposing inclined edges on the seismic vulnerability of marble block colonnades in the Pompeii archaeological site. These authors showed the seismic vulnerability of a colonnade to understand how this world cultural heritage site can be preserved.

Fossetti et al. [F.3] analysed effect of steel collars on the seismic behaviour of axially cracked historical stone columns subjected to rocking motion due to pulse-type ground shaking. These authored showed effectiveness in reducing the overturning risk. Papadopoulos and Vintzileou [P.1] numerically investigated the response to earthquake actions of five multi-drum columns from ancient-Greek temples with each column having a different size and slenderness

and having various numbers of drums. These authors showed that the number of drums did not significantly alter the columns' dynamic response and that the effect of the column size on the instability threshold is of great significance, as larger columns had the strongest resistance to deformation and collapse despite that these columns are more slender than other columns.

Asteris et al. [A.7] provided methodology for an earthquake-resistant design seismic vulnerability assessment of historical masonry structural systems, and this methodology was verified via analyses of existing masonry buildings in three countries (Greece, Portugal and Cyprus), with different seismicity levels influencing the risk to the masonry structures. Voyagaki et al. [V.2] analysed the classical problem of rocking of a rigid, free-standing block with respect to earthquake ground shaking containing distinct pulses. These authors showed that the non-linear equations of motion yield a more stable response than their linearized counterparts. Ambraseys and Psycharis [A.2] presented a numerical analysis of the seismic vulnerability of statues mounted on monolithic or multi-drum columns. Emphasis was given to the procedure that should be followed for the assessment of the stability to future earthquakes, and this procedure was presented using a case study of the statue of Apollo at the Academy of Athens. Stefanou et al. [S.4] analysed the dynamic response of reinforced masonry columns in classical monuments to investigate the impact of fractures to the overall stability of the structure.

Pitilakis and Tavouktsi [P.12] numerically analysed the seismic response of multi-drum and monolithic columns of ancient Greek temples by investigating the “chaotic” response of these complex structures and by evaluating the critical acceleration of the column's collapse using appropriate parametric analyses. Papaloizou and Komodromos [P.3] studied the behaviour of multi-drum columns and colonnades during earthquakes using a discrete element method. The numerical simulations showed that earthquakes with relatively low predominant frequencies appeared to endanger both standalone columns and colonnades more than earthquakes with higher predominant frequencies. Komodromos et al. [K.2] performed a simulation of the response of ancient multi-drum columns under harmonic and earthquake excitations using a discrete element method. These authors performed a large number of parametric studies to investigate the effect of certain parameters on the response of the columns. The simulations indicated that multi-drum columns are significantly more vulnerable to collapse at low-frequency excitations, which do not commonly occur in the Eastern Mediterranean region where most of these monuments can be found.

Asteris [A.6] presented general methodology for the seismic protection of historical structures and monuments. This author determined the possibility of a building to be damaged

beyond a specified damage level during strong ground motions and investigated several different seismic scenarios with different repair/strengthening decisions. Argyriou et al. [A.4] numerically analysed the seismic response of ancient multi-drum columns that corresponded to the Hellenistic portico of the Lindos acropolis. Extensive time domain parametric analyses have been performed to examine the behaviour of a column subjected to seismic motion having different peak ground acceleration values. Additionally, three different column drums connecting systems have analysed to examine their influence on the seismic response of the column. These authors presented a procedure to estimate the dominant period of a monolithic, multi-drum, and two multi-drum columns connected with an architrave. This work was motivated by seismic events that have caused damage or collapse of invaluable historical buildings, further demonstrating the vulnerability of unreinforced masonry (URM) structures to earthquakes. De Lorenzis et al. [D.1] analysed the failure of masonry arches, which are typical components of URM historic structures, subjected to horizontal ground acceleration impulses. These authors described the dynamic behaviour of the arch and predicted the combinations of impulse magnitudes and durations that would lead to the arch's collapse.

In a numerical study of the seismic response of a multi-drum stone column, Konstantinidis and Makris [K.3] numerically investigated the seismic response of multi-drum classical columns, especially the level of ground shaking that classical multi-drum columns can survive and the possible advantages and disadvantages of retrofitting multi-drum columns with metallic shear links that replace the wooden poles that were installed in ancient times. These authors concluded the following: a) typical classical multi-drum columns can survive ground shaking from strong ground motions recorded near the causative faults of earthquakes with magnitudes $M_w = 6.0 - 7.4$; b) in most cases, multi-drum classical columns that are free to dislocate at the drum interface exhibit a more controlled seismic response than monolithic columns of the same size and slenderness; c) the shear strength of the wooden poles has a marginal effect on the sliding response of the drums; and d) a stiff metallic shear link between the column drums may have an undesirable role on the seismic stability of classical columns and should be avoided.

Using nonlinear numerical analysis, Ziyaeifar et al. [Z.2] presented a methodology in which different possible techniques for rehabilitation of any particular historical building can be weighed and compared based on their risk mitigation added value, their reduction of the archaeological, architectural and aesthetical values and their predictable costs. The proposed methodology offers a rational process in which weighing coefficients defined independently by different disciplines (e.g., structural engineers, architects, archaeologists, and financial planners)

contribute to a decision making process for building rehabilitation. Psycharis et al. [P.14] presented a numerical study of the seismic behaviour of a proposed restoration of the Parthenon Pronaos. The column-architrave classical structure was represented by a discrete element model with the assumption of rigid blocks and frictional joints. A proposed reinforcement with titanium connections was analysed, and the architrave connections generally were found to improve the response by decreasing the permanent displacements, while titanium dowels between the column drums did not significantly improve the behaviour, and in some cases, these rods may be unfavourable to the safety of the structure.

Papantonopoulos et al. [P.8] used the distinct element method to predict the earthquake response of a multi-drum marble model of a classical column. These authors compared their result with experimental data for an ‘identical’ specimen under the same excitation. They showed that the distinct element method can capture quite well the main features of the response, in spite of the sensitivity of the response to even small perturbations of the characteristics of the structure or the excitation. Psycharis et al. [P.15] numerically analysed the seismic response of free-standing classical columns through implementation of the distinct element method. Typical sections of two ancient temples were modelled and studied parametrically to identify the main factors affecting the stability and to improve understanding of the earthquake behaviour of such structures. These authors concluded that around three cycles of monochromatic excitation at the predominant period of the expected earthquake motion led to a gross prediction of the stability of a classical column during an earthquake.

Pompei and Scalia [P.13] analysed the problem of a rigid block above the foundation subjected to a horizontal ground motion. They show that disregarding the sliding can cause mistakes when studying the body rotation. Sarhosis et al. [S.2] performed three-dimensional modelling of ancient colonnade structural systems subjected to harmonic and seismic loading. A typical section of the two-storey colonnade of the Forum in Pompeii has been modelled and studied parametrically to identify the main factors affecting the stability and to improve the understanding of the earthquake behaviour of such structures. These authors concluded that high-frequency motion requires a large base acceleration amplitude to cause the collapse of the colonnade in a shear-slip mode between the drums, and low-frequency harmonic excitations are more prominent in the structural collapse of two-storey colonnades than the high-frequency excitations, which predominantly exhibit a rocking failure mode.

Asteris et al. [A.9] gave an extensive overview by numerical modelling historic masonry structures. These authors pointed how studying historic masonry structures is a challenging task

due to the difficulties encountered in the description of the complex geometry, morphology, material heterogeneity, material properties characterization, material variation due to weathering and deterioration effects and complex loading conditions. These researchers concluded that a combined experimental-numerical basis is the key to validate, extend and improve the existing methods for the analysis of the mechanical behaviour of historic masonry structures. Ascione et al. [A.5] numerically analysed the load carrying capacity of 2D FRP/strengthened masonry structures with an adaptive discontinuous finite element model to perform a limit analysis of masonry structures strengthened by fibre composites. This model is able to predict the effects of fracture damage and delamination on the load carrying capacity of the reinforced structures.

To reinstate the original timber pole and empolia at the interfaces between stone members, Papadopoulos and Vintzileou [P.2] numerically analysed new titanium connectors for the column capitals at the classical temple of Apollo Epikourios. The investigation showed that installation of a single pole at the interface between the capital and the uppermost drum leads to limited reduction of maximum and residual deformations of the column under strong earthquakes. Furthermore, poles located at every column interface ensure a significant limitation of the column residual deformation. The main aspects of the design of new titanium poles and empolias were verified through specially designed experiments. Contemporary experimental studies of multi-drum column behaviour are based on the use of earthquake platforms (shake-tables). Thus, Papalou [P.4] examined the dynamic response of short free-standing classical columns by investigating effect of their geometrical features on their dynamic behaviour. This author noted how numerical models based on the finite element method can provide reasonable estimates of the dynamic response of classical columns but cannot predict all the attributes of their actual dynamic behaviour.

The protection of a classical column in areas of seismic activity using passive control techniques was studied by Papalou and Strepelias [P.7]. These authors experimentally investigated the effectiveness of particle dampers in reducing the vibrations of multi-drum ancient columns. They found that the particle damper, when properly designed, can significantly reduce the motion of a multi-drum column without altering the external appearance of the column. Papalou [P.5] investigated the preservation of the original architectural features of historic structures during their restoration and strengthening. This author concluded that a simple way to preserve the appearance of ancient classical columns and to simultaneously protect them from earthquakes is to replace damaged or missing parts with a system that absorbs the incoming seismic energy. Papalou and Strepelias [P.6] investigated the protection of classical columns in

areas of seismic activity using passive control techniques on small-scale multi-drum column models with a damaged bottom drum or inclined base under random excitation. These researchers found that the damper, if properly designed, can effectively reduce the motion of classical columns (reaching levels of up to 40 %) without altering their overall appearance.

Drosos and Anastasopoulos [D.3] provided shake-table testing of free-standing multi-drum columns composed of stones resting on top of each other without any connection, inspired by the form of ancient Greek temples. These researchers explained how the complexity increases exponentially with the number of drums, which is directly associated with the number of drum-to-drum interfaces, and how this number increased probability of interface imperfections. Drosos and Anastasopoulos [D.2] experimentally studied the seismic performance of reduced-scale free-standing columns to model the remnants of Greek temples. These authors stated that the tested multi-drum columns were proven to be very earthquake resistant, and when subjected to the strongest motions ever recorded in Greece, their permanent deformation was minimal.

Pena et al. [P.10] analysed the dynamical behaviour of free-standing block structures under seismic loading using a comprehensive experimental investigation to study the rocking response of four single blocks with different geometries and associations of two and three blocks. These authors carried out 379 tests on a shaking table to address the repeatability of the results and the stability of the rocking motion response. Based on their results, a significant understanding of the rocking motion mechanism is possible from high quality experimental data. Mouzakis et al. [M.4] experimentally investigated the earthquake response of a marble model of a classical column using a 1: 3 scale replica of a column from the Parthenon on the Acropolis of Athens made from the same material as the original. These authors found that the column might undergo large deformations during the shaking, and these deformations are not necessarily reflected by the residual displacements at the end of the shaking. Thus, the response is very sensitive even to small changes in the geometry or the input motion parameters.

Manos and Demosthenous [M.2] studied the dynamic and earthquake behaviour of a model of ancient columns and colonnades subjected to horizontal base motions. These authors stated how the seismic response mechanisms that develop on this solid block structural system during strong ground motions can include sliding and rocking. Krstevska et al. [K.5] experimentally investigated a prototype and model of the Antonina column in Rome to provide clearer column behaviour and resistance. The general conclusion from the shake-table tests was that low frequency content earthquakes may cause serious damage to the column. Alecci et al. [A.1] performed experimental investigations of brick columns confined with a CFRP composite. The

experimental results were shown to not fit the formulas provided in the literature, and the reasons for the discrepancies were investigated in the mechanical process that produced the failure.

Brodsky et al. [B.5] experimentally investigated the behaviour of the interfaces between the masonry wall units and joints on the global behaviour and on the local infill-frame interaction. The results show that the joint properties have a significant effect on the ultimate load, the initial stiffness and the energy dissipation. Cascardi et al. [C.1] experimentally investigated a new generation of strengthening of masonry columns. These authors analysed the experimental results on FRCM-confined masonry columns subjected to a centred compression test. Kouris and Triantafillou [K.4] investigated state-of-the-art strengthening techniques on masonry structures that are the most vulnerable to earthquakes. These authors provided insight into relevant experimental procedures and provide results from retrofitted masonry elements to illustrate the effectiveness of textile-reinforced mortar for enhancing strength and deformation capacity.

Witzany et al. [W.1] experimentally investigated strengthening and stabilizing masonry columns. These authors concluded that the confinement of a masonry column using carbon fibre fabrics with a high modulus of elasticity and high tensile strength relieves the transverse tensile stresses in the masonry and increases the ultimate deformation and ultimate load. Vintzileou and Toumbakari [V.1] experimentally investigated the dowel action of titanium bars connecting marble fragments at different angles. These authors investigated the diameter of the bars and the angle between a plane perpendicular to the dowel's longitudinal axis and the marble surface to determine the failure mode and to evaluate the dowel shear strength as a function of the angle between the dowel and marble surface.

Zambas et al. [Z.1] experimentally tested titanium reinforcement for the restoration of marble architectural members of the Acropolis monuments to solve the problem of structural degradation. Foraboschi [F.1] experimentally investigated the effectiveness of novel methods to increase the FRP-masonry bond capacity. Additionally, Foraboschi and Vanin [F.2] experimentally investigated new methods for bonding FRP strips onto masonry structures. The experimental results proved that the application technique influences both the load-carrying capacity and the ultimate horizontal displacement of the perforated masonry walls that were tested.

In a global review of the aforementioned research, it can be concluded that other researchers have numerically and experimentally investigated the behaviour of multi-drum stone columns

under different loads but have not systematically explored the effects of various parameters that have a significant effect on the behaviour and bearing capacity of these columns.

As part of this dissertation, the effect of the important parameters mentioned below on the behaviour and ultimate bearing capacity of multi-drum stone columns under vertical and horizontal static loads and few different earthquake scenarios was investigated:

- Block height (number of joints),
- Joint type,
- Various parameters describing the bolts in the joints (material, diameter and length of the bolt, diameter of bolt hole).

All tests were conducted on free-standing multi-drum columns that carry their own weight alone or columns that have an additional weight (mass) at the top. All tests were conducted with successive load increments until the column failed. Based on the research results, some important conclusions have been made regarding the effect of the considered parameters. All tests were conducted on previously established equipment in the seismic testing laboratory at the University of Split, Faculty of Civil Engineering, Architecture and Geodesy where a number of similar studies [B.1, B.2, B.3, B.4, G.1, G.2, G.3, R.1, R.2, R.3] have been previously conducted.

The subject research is original and contributes to the development of science in the research area, which can be applied to multi-drum columns in practice. The confirmation of this is five scientific papers produced [I, II, III, IV, V], four of which have been published in the journals cited in the Current Contents database, and one is in the process of publishing (see Appendix).

2. Goal and purpose of the research

Some of the major parameters affecting the behaviour and bearing capacity of multi-drum columns are related to the number of blocks, i.e., the number of joints, the properties of the joints and the properties of the bolts used in the joints between blocks. As the effects of these parameters have not been investigated to date or have been investigated only sporadically and numerically, the primary goal of the research in the framework of this thesis is an experimental study of the effect of these mentioned parameters on the behaviour and bearing capacity of multi-drum stone columns under static loads and different earthquake types. Namely, the aim is to contribute to the development of science in the relevant area, which would result in the publication of scientific/research papers in relevant journals.

The purpose of the research is to make important conclusions based on the experimental study that was conducted, and the results will find practical application in the renewal/strengthening of existing multi-drum columns and the creation of new columns with the purpose of improving safety under static loads and earthquakes. Therefore, the basic purpose of this research is the application in the practice and the improvement of existing knowledge in the area of reconstruction and strengthening numerous historic stone buildings.

3. Research methodology

Systematically planned, well-prepared, correctly conducted and well-analysed experimental research with proven equipment and instrumentation, certainly has an advantage over numerical research (especially when numeric models are not sufficiently reliable and verified). Therefore, to achieve the planned objectives, experimental studies are preferred to investigate the effect of the considered parameters of multi-drum stone columns. The latest state-of-the-art static and seismic testing equipment for multi-drum column models, physical measurement equipment, and data collection and processing equipment were used. The experiments were conducted on the basis of detailed research plans and programmes, in accordance with contemporary achievements and knowledge in the implementation of experimental research.

In the adopted methodological approach, a separate study of the multi-drum stone columns parameters mentioned below was planned, whereby all of the other parameters were held constant (equal). This approach ensures high reliability of the results and conclusions obtained from small-scale models when applied to full-scale multi-drum columns in practice. For all models, the characteristic physical sizes were measured. Only the most important experimental results are shown.

I. Effect of the drum height on the bearing capacity of a composite multi-drum column under a static load.

This effect was investigated on small-scale columns with hinges at the top and bottom. The columns were approximately 2.5 m high and were formed from 2, 4, 6 and 12 blocks per column. The tests were conducted separately for the action of the axial compressive forces only, and separately for the action of the axial compressive forces and the horizontal lateral force at half the height of the column. The forces were progressively increased to column failure. The results of these studies were published in [1].

II. Effect of the drum height on the seismic behaviour of a free-standing multi-drum column

These investigations are a continuation of the foregoing. Free-standing multi-drum columns with a height of 1.2 m, formed from 1, 3 and 6 blocks per column, and having the same mass at the top were analysed. The columns were analogous to those in point I., but half as tall. All samples were exposed to the three different earthquake accelerograms, with a gradual

increase in the acceleration until column failure. The results of these studies were published in [II].

III. Effect of the joint type on the bearing capacity of a multi-drum column under static load

Columns analogous to those in point I. were used in this work, and the columns had six blocks per height. Four types of joints were used (dry, filled with stone powder, lead plate and epoxy). The loads and all other factors were analogous to that specified in point I. A detailed view of the research results is given in [III].

IV. Effect of the joint type on the seismic behaviour of a free-standing multi-drum column

This research is a continuation of the work in point III. A free-standing multi-drum columns analogous to those in point II. were used and had six blocks (joints) per height. Joints were varied as in point III. Samples were also investigated using three different earthquakes, as in point II. The results of the research were published in [IV].

V. Effects of several bolt parameters on the bearing capacity of a composite multi-drum stone column during an earthquake

A free-standing multi-drum column with a height of 1.2 m and made from 6 blocks was analysed; this is analogous to the columns in points II. and IV. Joints from stone powder were adopted. First, the behaviour of the columns without bolts was investigated. Columns with bolts from different materials (wood, lead, rubber, aluminium and stainless steel) were subsequently investigated. Then, for some materials, the effect of the bolt length, the bolt diameter and the diameter of the bolt hole was investigated. All samples were exposed to two different earthquakes. A detailed view of the research results is given in [V].

4. Tested samples

4.1. Static tests

In static tests [I, III], the multi-drum columns were considered according to Fig. 4.1. They represent free-standing columns with height h , which are horizontally retained with hinge on the top and bottom, or free-standing cantilever column with height $h/2$. The columns were first tested on the action of the V force only, where the force was successively increased to column failure or to the loss of column stability. Then, for several constant values of V force, the columns were additionally loaded with H force. This force was also successively increased to column failure. The effect of the block number was investigated in [I], and the effect of the joint type on the behaviour and bearing capacity of the multi-drum columns was studied in [III].

Fig. 4.1. Tested multi-drum columns under centric and eccentric compression [I, III]

4.2. Dynamic tests

In the dynamic tests [II, IV, V], the cantilevered multi-drum columns were considered according to Fig. 4.2. The columns were loaded with self-weight (Fig. 4.2.a), and with self-weight and an additional mass at the top (Fig. 4.2.b). The effect of the height (number) of the blocks on the multi-drum column behaviour during an earthquake was investigated in [II], and the effect of the joint type was investigated in [IV]. The effect of various parameters of bolts in

joints on the bearing capacity of multi-drum columns was studied only at the column according to Fig. 4.2.a [V].

Fig. 4.2. Tested free-standing columns [II, IV, V]

5. Blocks used in the tested multi-drum columns

In historical multi-drum stone columns, stone blocks from different types of stone and marble were used, and the connecting surfaces have been processed using different methods. Therefore, this raises the question of how many types of stones to use in less extensive experimental tests. In this research, instead of stone blocks, concrete blocks were used (made from the same fresh concrete) for the following reasons:

- Avoiding the use of specific types of stone.
- Avoiding the effects on the column parameters due to probable local anomalies in the stone.
- Reducing the cost and duration of the research.

As concrete has mechanical characteristics that are very similar to the stone (so-called artificial stone), when the concrete aggregate is from that stone and the failure of all the tested columns in the dynamic tests did not occur due to block fracture but by loss of stability, this substitution is considered to have no practical effect on the conclusions. Concrete blocks with limestone concrete were used in this research, and the obtained results are considered to be the closest to the multi-drum limestone columns whose failure is not caused by the strength of the blocks.

To reduce the effect of eccentricity on the transfer of vertical stresses in the joints between blocks and to accelerate and simplify the fabrication of the columns, the connecting surfaces of the blocks were cut horizontally after concrete hardening. However, this did not exclude unevenness on the block contact surfaces and vertical eccentricity of the columns.

One of the important parameters that affects the mutual relative horizontal displacements (slipping) between the adjacent blocks, is the coefficient of friction between the blocks. This coefficient depends on the type of stone, the processing of the connect surfaces and the presence of any material in the joints (e.g., stone powder or mortar). Keep in mind that the coefficients of friction under static and dynamic loads are different. The friction coefficients between the concrete blocks are considered to be similar to friction coefficients between the limestone blocks (with the same contact surface treatment); this finding is very important because the effects of sliding and especially rocking in multi-drum columns during earthquakes are very common and significant.

6. Base accelerograms used in the dynamic tests

In the conducted dynamic tests [II, IV, V], the earthquake accelerograms shown in Fig. 6.1.a were used: artificial accelerogram (AA) was used on the elastic response spectrum [E.1] for type 1 and soil type A, the horizontal component N-S of the Petrovac earthquake (AP) [A.3], and the horizontal component N-S of the Ston earthquake (AS) [A.3]. The spectral values of the accelerograms are presented in Fig. 6.1.b. It was not the intent to analyse the impact of a large number of different accelerograms, and found the most unfavorable for the tested samples. However, a few accelerograms with a characteristic changes in the acceleration in time were selected: earthquakes with a long duration and a relatively long predominant period, earthquakes with a medium-long duration and a longer predominant period and earthquakes with a short duration and a small predominant period (so-called impact earthquakes). This finding was justified and sufficient because only the relative effects of some of the multi-drum column parameters were tested using the earthquake types considered in this study.

Fig. 6.1. Adopted accelerograms and their spectral values

Since, in all tested samples, nonlinearities occur even at low excitation accelerations and are pronounced at high accelerations (e.g., rocking and slipping), it is difficult to correlate the basic periods of free elastic oscillation of the samples with the spectral values of the excitation, as

shown in Fig. 6.1.b. Namely, it is difficult to define which of the adopted excitations and which a_{gmax} is the most unfavourable for each sample.

In this study [II, IV, V], the earthquake accelerograms from Fig. 6.1.a have been applied with a successive increase in the maximum acceleration amplitude, a_{gmax} , to column failure or loss of its stability. The adopted acceleration increment was 0.2 ms^{-2} for AA and AP, and $0,5 \text{ ms}^{-2}$ for AS.

7. Testing and measurement equipment

For the static tests of the columns under an axial compressive force and horizontal force, a steel frame was used (Fig. 7.1.). Calibrated hydraulic presses were used in the force applications. The accuracy of the measured vertical force was within 0.5 %, and the horizontal force was within 0.1 %.

Fig. 7.1. Steel frame with the test sample

For earthquake testing of the samples, a shake-table at the University of Split, Faculty of Civil Engineering, Architecture and Geodesy was used (Fig. 7.2.). Its main characteristics were the following: uniaxial horizontal acceleration, area of 4 x 4 m, maximum sample weight of 20 t, maximum acceleration of ± 3 g, maximum frequency of 30 Hz, maximum displacement of ± 15 cm, and a maximum sample height of 9 m. The shake-table can apply an acceleration function (accelerogram) with high precision.

Fig. 7.2. Shake-table at the University of Split, Faculty of Civil Engineering, Architecture and Geodesy

The static forces were measured with force gauges (KDB force transducer, HBM). Displacements were measured by displacement sensors PB-25-S10-NOS-10C (Uni Measure). The strains were measured with strain gauges (type 100/120 LY41, HBM). High-frequency piezo-electric accelerometers type 4610 (MS) were used for the acceleration measurements. Two precision cameras (Type Canon EOS M5) were used to record all tests. The Quantum-x mx 840 A system (HBM) was used for data collection from the sensors.

8. Problems with small-scale models in static and dynamic experimental tests

It is well-known that the sample size in static and especially dynamic experimental tests affects the results obtained. Therefore, if tests on real constructions are conducted on small-scale models, the application of the results of such tests on a full-scale structure (prototype) should be done so very carefully.

It is also well known that the static strength of a concrete sample having the same shape (cylinder, cube, prism) will depend on its dimensions. In this regard, the strengths obtained on bodies having smaller dimensions are generally greater than the strengths obtained on bodies with larger dimensions. This also applies to the dynamic strength of concrete, as well as the strength of many other materials (e.g., stone, brick, and wood). The reasons stated above are numerous, and one of the most important reasons is that the strength of a material cannot be examined at a “point-level“, but can be examined on a sample that represents the structure of a specific shape and dimensions. It is obvious that the failure of such structures will be caused by different strain/stress sizes. Therefore, failure of the $1/n$ ($1:n$) small-scale model and the prototype with n -times ($n>1$) larger dimensions ($1/n = \text{scale factor}$), made from theoretically the same material, will not have equal strength.

Fig. 8.1. shows a small-scale model of a column and its prototype with n -times larger dimensions. If a small-scale model and prototype use the same material, i.e., a material with equal mechanical properties, the relation between some important parameters with respect to the small-scale model and the prototype is shown in Table 8.1.

As a result of the n -times smaller dimensions, most all the important parameters of the small-scale model are smaller than the prototype (except the frequency of free oscillations). Some of these parameters will be briefly explained.

Normal stresses/strains of the gravity axial forces and the inertial load (earthquake) forces are n -times lower. It is obvious that in the small-scale model, the centric compression capacity is overrated in the prototype.

The shear stress/strain of the gravity transverse forces loads and inertial forces are also n -times lower. Thus, the small-scale model also has a greater bearing capacity with regard to the prototype.

The normal stresses/strains from the bending moments of the gravity load and inertial forces are n^2 times lower. It is obvious that the bending capacity of the small-scale model, with regard to the prototype, is substantially greater.

Fig. 8.1. Small-scale model and prototype

Table 8.1. Relation of models and prototype small-scale parameters ($n \geq 1$)

Static and dynamic tests	Dimensions (a, b, c)	n^{-1}
	Volume	n^{-3}
	Weight	n^{-3}
	Area	n^{-2}
	Moment of resistance	n^{-3}
	Moment of inertia (stiffness)	n^{-4}
	Stress from self weight:	
	Normal from longitudinal force	n^{-1}
	Normal from bending moments	n^{-2}
	Shear from transverse force	n^{-1}
Dynamic tests	Inertial force	n^{-3}
	Moment of inertial force to the base	n^{-4}
	Inertial force strain:	
	Normal from longitudinal force	n^{-1}
	Normal from bending moment	n^{-2}
	Shear from transverse force	n^{-1}
	Free oscillation period	$n^{-1/2}$
	Free oscillation frequency	$n^{1/2}$

As the frequency of free oscillations of the small-scale models is $n^{1/2}$ - times higher than the prototype, the same dynamic excitation on both structures will result in different and practically incomparable results. In the dynamic analysis of the small-scale model, this problem is most often solved by shortening the time domain of the real accelerometer using the coefficient $n^{1/2}$ (reduction of the predominant excitation period).

Considering the abovementioned problems in using simple small-scale models, it is obvious that these models cannot adequately simulate a number of important characteristics of the prototype, such as: the ultimate bearing capacity and stability, the effects of sliding and rocking and the beginning and distribution of nonlinearity in the structure.

To make the conclusions obtained by testing on a small-scale model as true as possible and applicable to a prototype, some "interventions" on small-scale models are often used (e.g., mass addition and change in the material properties). This approach has some other problems. To provide a more reliable application of the research to the small-scale models for practical structure, further intensive research in this area is necessary. The use of different materials for the small-scale model with regard to the prototype in cases of non-linearity in the material (if

especially pronounced) does not make sense because different materials cannot have the same nonlinear effects as a prototype material. In such cases, it is difficult to speak about the satisfactory similarity of small-scale model and prototype.

In the study of the relative effect of some parameters on a real structure using small-scale model, the issues related to small-scale model are significantly simple. Namely, with acceptable accuracy, the relative effect of some parameters on the prototype and the small-scale model can be assumed to be very similar. For example, with acceptable accuracy, the prototype bearing capacity for two different material strengths (considered parameter) can be assumed to be very close to the small-scale model bearing capacity for these two strengths.

Since the all conducted tests within this dissertation are related to the relative effects of some important parameters of multi-drum columns on their behaviour and ultimate bearing capacity under static load and earthquake, the results obtained on the adopted small-scale models can be applied to the full-size structures.

9. Effect of the drum height on the behaviour and bearing capacity of multi-drum stone columns

The research results of the effect of the drum height (joint number) on the behaviour and bearing capacity of multi-drum stone columns are presented in this chapter. The research results for static load were published in [I] and are briefly shown in point 9.1. Research results for earthquake load were published in [II] and are briefly shown in point 9.2. The results in points 9.1 and 9.2 are not discussed because they are shown in [I, II], but the main conclusions of the research have been presented. In point 9.3, some effects of the block height for static load and earthquakes are compared.

9.1. Research results for static load [I]

The columns with practically equal heights were tested with a different height with an equal number of blocks per column height (Fig 9.1.). Columns with two (C1), four (C2), six (C3) and twelve (C4) blocks per column height were considered. Joints with a thin layer of stone powder between the blocks were used. The parameters measured are shown in Fig. 9.2.

Fig. 9.1. Geometry of tested columns

Fig. 9.2. Measured quantities

The columns were first tested under axial compression only: force V , with a gradual increase to column failure. Then, the columns were tested for the action of the horizontal force H with the previously applied vertical force V (100, 200, 300, 500 and 800 kN). Force H was also successively increased to column failure. The ultimate bearing capacity of tested columns is presented in Fig. 9.3.

Fig. 9.3. Ultimate bearing capacity of the columns

The vertical displacement of the column bottom under centric force V is presented in Fig. 9.4., and the average axial strain is shown in Fig. 9.5.

Fig. 9.4. Vertical displacement of the column bottom under force V only [I]

Fig. 9.5. Average vertical strain ε_{1-2}^V under force V only [I]

The relation between the horizontal force H and the horizontal displacement in the middle of the column height is shown in Fig. 9.6.

Fig. 9.6. H - u relation under forces V and H [I]

The main conclusions of the study of the block number effect on the behaviour and ultimate bearing capacity of the multi-drum columns loaded under axial compression and bending with a longitudinal compressive force are the following [I]:

- With an increase in the number of drums (joints) per tested column height, there was a reduction in the columns' ultimate bearing capacity and flexural and shear stiffness, and there was an increase in the columns' horizontal and vertical displacements.
- The bearing capacity ratio of the tested columns under an axial force alone was 1 : 0.87 : 0.73 : 0.6, respectively, which proves that an increase in the block number significantly reduced the column bearing capacity.
- The cause of the issue mentioned above is the increase in the number of joints in the column, which is its weakest link. Namely, they softens the column, leads to a concentration of the stress in their plane, causes eccentricities in the transverse cross section and contributes to the deviations in the verticality. Normal tensile stresses cannot be transmitted in the joint plane, and the efficiency of the horizontal shear force transmission decreased.
- Under axial compression, the columns only underwent brittle failure, and columns loaded by compression and flexure had more ductile failure (especially in the case of highly eccentric axial forces).
- A general conclusion for practical application is that for columns loaded with static forces, a smaller number of blocks (joints) in a multi-drum column should be preferred.

The above conclusions are valid for multi-drum columns with dry joints and joint filled with stone powder.

Fig. 9.8. Measured quantities

The maximum base acceleration, a_{gmax} , in the limit state of the columns is presented in Fig. 9.9., where the solid line indicates a_{gmax} at column failure and the dotted line indicates a_{gmax} from the previous acceleration increment that preserved column stability.

Fig. 9.9. Maximum base acceleration a_{gmax} in the limit state of the tested columns [II]

The horizontal displacements u_1 and u_2 of the columns S1 and S1m for the artificial accelerogram (AA) at a given a_{gmax} are presented in Fig. 9.10. The horizontal displacements u_1 to u_6 of columns S3 and S3m for the Petrovac accelerogram (AP) at a given a_{gmax} are shown in Fig. 9.11. Fig. 9.12. shows the horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 and u_{12} of columns S6 and S6m for the Ston accelerogram (AS) at a given a_{gmax} .

Fig. 9.10. Horizontal displacements u_1 and u_2 of the tested columns for the artificial accelerogram (AA) at a given a_{gmax}

Fig. 9.11. Horizontal displacements u_1 to u_6 of the tested columns for the Petrovac accelerogram (AP) at a given a_{gmax}

Fig. 9.12. Horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 and u_{12} of the tested columns for the Ston accelerogram (AS) at a given a_{gmax}

The deformations of column S6 for the Ston accelerogram at $a_{gmax} = 6.0 \text{ ms}^{-2}$ are presented in Fig. 9.13., and the deformation of column S6m for the same accelerogram is presented in Fig. 9.14.

Fig. 9.13. Deformations of the column S6 for the Ston accelerogram (AS) at $a_{gmax} = 6.0 \text{ ms}^{-2}$

[II]

Fig. 9.14. Deformations of the column S6m for the Ston accelerogram (AS) at $a_{gmax} = 3.5 \text{ ms}^{-2}$
[II]

The main conclusions of the study are the following [II]:

- The number of blocks (joints) in the multi-drum column significantly affects its stiffness, resistance and bearing capacity under earthquakes.
- The effect of the number of blocks on the column bearing capacity depends on the earthquake type. For AA, the ratio between the column bearing capacities were S1 : S3 : S6 = 1 : 0.71 : 0.64 and S1m : S3m : S6m = 1 : 0.83 : 0.66, i.e., increasing the number of blocks in the column decreases the column's bearing capacity. For AP and AS, an increase in the number of blocks increased the column's capacity. So, for AP the bearing capacity ratio was S1 : S3 : S6 = 1 : 1.22 : 1.33 and S1m : S3m : S6m = 1 : 1.42 : 1.57, and for AS this ratio was S1 : S3 : S6 = 1 : 1.42 : 1.71 and S1m : S3m : S6m = 1 : 1.16 : 1.16.
- The results above explain why a larger number of blocks (joints) in the column decreases its stiffness. This reduces the earthquake forces generated in the column, but also reduces its resistance. AA characterized by long action, large displacements, and high earthquake energy, which explains the reduction in the column bearing capacity with an increase in

the number of blocks in it. AP has a shorter action than AA, and AS is characterized by a very short impact. The AP and AS accelerograms lowered the earthquake energy in the column. This is why the column bearing capacity increases with an increase in the number of blocks in the column, i.e., with the reduction in the column's stiffness.

The conclusions above were valid for joints with stone powder and probably for all soft joints (dry joints, joints with soft mortars).

Generally, increasing the number of blocks in a column can be concluded to increase the column's bearing capacity for impact earthquakes, which do not cause any major displacements and do not introduce more energy to the columns. Conversely, increasing the number of blocks in a column can reduce the column's bearing capacity to long lasting earthquakes, which inducing a greater column motion and introduces greater energy to the column.

For the practical application of these results, it is important to know which type of earthquake and the maximum accelerations that can be expected at the subject location.

9.3. Comparison of the research results from 9.1 [I] and 9.2 [II]

Based on the results of the research [I, II] conducted on the effect of the number of blocks in a multi-drum column in terms of the column's behaviour and bearing capacity, globally, the following can be concluded:

- By increasing the number of blocks (joints) in a multi-drum column, there is a decrease in its stiffness and resistance. These decreases reduce the column's bearing capacity and increase displacements under static loads and under earthquakes that introduce high energy into the column (which excites strong column motion).
- With a decrease in the blocks number in a multi-drum column, there is an increase in the column's stiffness and resistance. This increases the column's bearing capacity and reduces motion under static load and under short-term earthquakes that introduce low energy into the column (which do not excite strong column motion).

10. Effect of the joint type on the behaviour and bearing capacity of multi-drum stone columns

This section presents the research results of the joint type effect on the behaviour and bearing capacity of multi-drum stone columns. The research results for static loads, which were published in [III], are briefly presented in point 10.1. The research results for earthquake loads were published in [IV], and are briefly shown in point 10.2. In point 10.3, some of the results of the joint type effects for static and earthquake loads were compared. The test results are not discussed because they expanded upon in [III, IV], but the main conclusions of the research are listed.

10.1. Research results for a static load [III]

The columns tested are shown in Fig. 10.1.a, and are analogous to the columns tested in point 9.1. Columns with six blocks per height and four types of joints were used: dry joint (C-DJ), stone powder joint (C-SPJ), lead joint (C-PBJ) and epoxide joint (C-EPJ).

Fig. 10.1. Geometry of the tested columns and joint types

The columns were first tested for the action of the centric compressive force V only, and then for the simultaneous action of force V and the horizontal force H . These tests were performed on columns identical to those in point 9.1. The measured quantities are presented in Fig. 10.2. The bearing capacity of the tested columns is shown in Fig. 10.3.

Fig. 10.2. Measured quantities [III]

Fig. 10.3. Bearing capacity of the column

Vertical displacement of the bottom of the columns for force V only is presented in Fig. 10.4., and the average vertical strain ε_{1-2}^V for force V only is shown in Fig. 10.5. The average circumferential strains ε_{1-2}^P , ε_{3-4}^P of the columns for centric force $V = 500$ kN is presented in Fig. 10.6., and in Fig. 10.7. presents the $H-u$ relation for the tested columns. Fig. 10.8. shows a photo of column C-SPJ at failure under force V only.

Fig. 10.4. Vertical displacement of the bottom of the columns [III]

Fig. 10.5. Average vertical strain ε_{1-2}^V for force V only [III]

Fig. 10.6. Average circumferential strains ϵ_{1-2}^p , ϵ_{3-4}^p of the columns at $V = 500$ kN [III]

Fig. 10.7. Horizontal force H – horizontal displacement u relation for the tested columns

Fig. 10.8. Photo of column C-SPJ at failure under force V only

Based on the obtained experimental results, the following global conclusions can be made [III]:

- The type of joint has a large effect on the bearing capacity of multi-drum stone columns under static forces.
- Under axial compression, the ratios between the bearing capacities of the tested columns were C-EPJ : C-DJ : C-SPJ : C-PBJ = 1 : 0.68 : 0.59 : 0.51.
- Column C-EPJ with epoxy joints had the highest bearing capacity because the epoxide layer in the joint ensured equal transmission of the compression stresses through the blocks, allowed transmission of tension through the joint and reduced the horizontal strains of the block at the joint level. This column had a greater bearing capacity than the column made from a single block.
- Column C-PBJ with a soft lead sheet in the joints had the lowest bearing capacity because this joint cannot ensure equal transmission of the compressive stresses across the joint, and does not limit the lateral deformability of the blocks along the joint. Column C-SPJ with stone powder joints had a slightly greater bearing capacity than the column C-PBJ because the stone powder ensured more uniform vertical compression transmission and allowed slightly less lateral displacement of the blocks at the joint level.
- Column C-DJ with a dry joint had a greater bearing capacity than columns C-PBJ and C-SPJ. This joint ensures greater friction and lower lateral column deformation.
- The effect of the joint type on the bearing capacity of the columns simultaneously loaded by force V and force H is analogous to the columns under force V only.
- Generally, a softer layer resulted in a lower column bearing capacity, a lower stiffness, and greater column displacement.
- For multi-drum columns in practice, it is extremely favourable that blocks connecting surfaces be straight and rely more evenly, i.e., to eliminate as much compressive forces eccentricity transfer as possible and to increase the shear capacity of the joints.

10.2. Research results for earthquake load [IV]

The tested free-standing small-scale multi-drum columns (Fig. 10.9.) were analogous to those in point 9.2. The columns were formed from six identical blocks, with three types of joints. Columns C-PJ, C-DJ and C-EJ were loaded with their self-weight only, with joints from stone powder (C-PJ), dry joints (C-DJ) and epoxy joints (C-EJ). Columns Cm-PJ, Cm-DJ and Cm-EJ

were in all equal to columns C-PJ, C-DJ and C-EJ, but had extra weight $m = 500$ kg at the top. All columns were practically equal in height, as the joint thickness was approximately 1 – 3 mm.

Fig. 10.9. Geometry of the tested columns

The columns were also exposed to the base accelerations according to the point 6. Measured displacements u and accelerations a are shown in Fig. 10.10.

Fig. 10.10. Measured displacements u and accelerations a

The maximum base accelerations a_{gmax} at the limit state of the tested columns is shown in Fig. 10.11., where the solid line indicates the column failure and the dotted line indicates the previous acceleration increment that preserved column stability.

Fig. 10.11. Maximum base acceleration a_{gmax} in the limit state of the columns

The horizontal displacements of the blocks at some points in some columns during the base accelerations are shown in Fig. 10.12. and Fig. 10.13.

Fig. 10.12. Horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 and u_{12} of the tested columns C-PJ, C-DJ and C-EJ for the Petrovac accelerogram (AP) at a given $a_{g,max}$

Fig. 10.13. Horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 and u_{12} of tested columns Cm-PJ, Cm-DJ and Cm-EJ for the artificial accelerogram (AA) at a given a_{gmax}

The deformations of column C-DJ for AP at $a_{gmax} = 1.6 \text{ ms}^{-2}$ are presented in Fig. 10.14.

Fig. 10.14. Deformations of column C-DJ for AP at $a_{gmax} = 1.6 \text{ ms}^{-2}$

The main conclusions of the study are the following:

- Except for the earthquake type, the capacity of the multi-drum column significantly depends on the joint type.
- When the joints are stiffer, the column stiffness is greater. The relation of the fundamental period of the free oscillations for columns C-DJ, C-PJ and C-EJ was 3.2 : 2.8 : 1.0. A stiffer column has a greater resistance than a softer column, but they also pulling greater seismic forces.
- The bearing capacity of columns C-PJ, C-DJ, C-EJ were approximately 1.0 : 1.0 : 1.3 for AA, 1.3 : 1.0 : 1.1 for AP and 1.7 : 1.1 : 1.0 for AS.
- The bearing capacity of columns Cm-PJ, Cm-DJ, Cm-EJ were approximately 1.0 : 1.0 : 1.5 for AA, 1.6 : 1.7 : 1.0 for AP and 1.2 : 1.8 : 1.0 for AS.
- Based on the results mentioned above, it is obvious that the columns with rigid joints (C-EJ, Cm-EJ) had the greatest bearing capacity for AA, and columns with soft joints (C-PJ, Cm-PJ, C-DJ, Cm-DJ) had the greatest bearing capacity for AP and AS.
- Depending on the earthquake type and the joint type, a multi-drum column may have significantly smaller or even significantly greater bearing capacity than a single-block column.
- Soft joints (e.g., dry or with stone powder) are expected be the most unfavourable for earthquakes that cause great displacements and introduce a lot of energy to the system.
- Rigid joints (e.g., epoxy joint) and column formed by a single block are expected to be the most unfavourable for earthquakes that have a short duration and that introduce less energy.

10.3. Comparison of research results from 10.1 [III] and 10.2 [IV]

Based on the results of the research [III, IV] on the joint type effect in multi-drum columns with respect to its behaviour and bearing capacity, globally, the following can be concluded:

- Soft joints in a multi-drum columns, compared to stiff joints, result in a columns lower stiffness and greater deformability. The result is a significantly lower column bearing capacity when subject to vertical and horizontal static loads, than for stiff joints or columns formed from one block.
- Soft joints during earthquake, relative to the stiff joints, result in less induced earthquake energy. However, due to that column having less resistance and depending on the

earthquake type, a column with soft joints may have a smaller or greater bearing capacity than a column with stiff joints or a column formed from one block. Soft joints are generally more unfavourable than rigid during earthquakes, as the earthquakes induce a large amount of energy into the structure. Namely, rigid joints or columns from a monolithic block are generally more unfavourable at impact earthquakes, which induced lower energy.

11. Effects of several bolt parameters on the bearing capacity of multi-drum stone columns under an earthquake

This section briefly presents the research results of the effect of various bolt parameters on the multi-drum column bearing capacity during an earthquake; the results are presented in more detail in [V]. The effects of the bolt material, the length of the bolt, the bolt diameter and the diameter of the bolt hole were investigated.

The free-standing multi-drum column shown in Fig. 11.1. was tested. The column was formed from 6 equal blocks per column, with dry joints between the blocks. The column was exposed to horizontal acceleration of the artificial accelerogram (AA) and the Ston accelerogram (AS), according to Fig. 6.1. The adopted bolt geometry is shown in Fig. 11.2. The bolt material (wood, lead, rubber, aluminium, stainless steel), bolt diameter, bolt length and the diameter of the bolt hole varied. The appearance of the bolts used is shown in Fig. 11.3.

Fig. 11.1. Tested multi-drum column

11. Effects of several bolt parameters on the bearing capacity of multi-drum stone columns under an earthquake

Fig. 11.2. Adopted bolt geometry

Fig. 11.3. Photography of the bolts used

The effect of the bolt length (h_b) on the multi-drum column bearing capacity was tested using lead and stainless steel bolts ($\phi_b = 12\text{mm}$, $\phi_h = 14\text{mm}$). The bolts with length $h_b = 72\text{mm}$ (short bolt), $h_b = 120\text{mm}$ (medium long bolt) and $h_b = 168\text{mm}$ (long bolt) with the ratio $h_b / \phi_b = 6, 10$ and 14 , respectively, were considered.

The effect of the bolt diameter ϕ_b was also tested on the lead and stainless steel bolts with $\phi_b = 12\text{mm}$ and $\phi_b = 20\text{mm}$, i.e., with $\phi_b / d = 1 / 16$ and $1 / 9.6$, respectively.

The effect of the bolt hole diameter was also tested using the lead and stainless steel bolts, with $\phi_b = 12\text{mm}$ and $h_b = 120\text{mm}$. The bolt hole diameters were $\phi_h = 14\text{mm}$ and $\phi_h = 22\text{mm}$, with $\phi_h / \phi_b = 1.17$ and 1.83 , respectively.

11.1. Effect of the bolt material

The effect of the bolt material on the column bearing capacity is shown in Fig. 11.4. The horizontal displacements of the top of the column for AA at $a_{gmax} = 2.2 \text{ ms}^{-2}$ and for AS at $a_{gmax} = 4.0 \text{ ms}^{-2}$ are shown in Fig. 11.5. The appearance of the column without bolts and with stainless steel bolts for AA at $a_{gmax} = 2.2 \text{ ms}^{-2}$ is shown in Fig. 11.6.

Fig. 11.4. Effect of the bolt material on the column bearing capacity

$$(h_b = 120 \text{ mm}, \phi_b = 12 \text{ mm}, \phi_h = 14 \text{ mm})$$

a) AA at $a_{gmax} = 2.2 \text{ ms}^{-2}$

11. Effects of several bolt parameters on the bearing capacity of multi-drum stone columns under an earthquake

b) AS at $a_{gmax} = 4.0 \text{ ms}^{-2}$

Fig. 11.5. Horizontal displacement of the top of the column depending on the bolt material

a) Column without bolts

b) Column with stainless steel bolts

Fig. 11.6. Appearance of column without bolts and with stainless steel bolts for AA at $a_{gmax} = 2.2 \text{ ms}^{-2}$ during excitation

The main conclusions related to the effect of the bolt material are the following [V]:

- Compared to the columns with bolts, the same column without bolts had a lower bearing capacity.
- Except for the bolt material, the bearing capacity depends on the earthquake type. The bearing capacity of the column with a bolt relative to the same column without the bolt was 1.25 : 1.0 to 1.89 : 1.0.
- The contribution of the bolts to the column bearing capacity was significantly greater for AS than for AA, which is explained by the fact that the size of the shear is significantly greater for impact earthquakes (AS) than for earthquakes that cause greater displacements and structure bending (AA).
- The columns without bolts had far greater relative horizontal displacements between adjacent blocks than the column with bolts.
- The ratio between the bearing capacity for the columns with bolts made of wood, lead aluminium and stainless steel for AA was approximately 1 : 0.94 : 0.83 : 1.0 : 1.11, and approximately 1 : 0.93 : 0.93 : 1.07 : 1.13 for AS. It is obvious that in the considered problem the bolt material effect was small, which is explained by the fact that the bolt shear capacity was not exceeded in any case and collapse of all the columns occurred by the bolts pulling out. In the case of larger transverse seismic forces, the effect of the bolt strength can be significant.

11.2. Effect of the bolt length

The effect of the bolt length (h_b) on the bearing capacity of the tested columns ($\phi_b = 12mm, \phi_h = 14mm$) is presented in Fig. 11.7. It is obvious that the effect is very small because the adopted bolt primarily bears shear forces, with negligible bending moments. If the adhesion of the bolt and block is ensured, i.e., the bolt can withstand tension, the effect of the bolt length could be significant.

Fig. 11.7. Effect of the bolt length (h_b) on the bearing capacity of the column

($\phi_b = 12mm$, $\phi_h = 14mm$)

11.3. Effect of the bolt diameter

The effect of the bolt diameter on the bearing capacity of the tested columns is presented in Fig. 11.8. The results are unexpected and, at first glance, illogical. Namely, for AA the bearing capacity of the column with the bolt $\phi_b = 12$ mm was greater than with the bolt $\phi_b = 20$ mm (bearing ratio 1 : 0.76 for the lead bolt and 1 : 0.7 for the stainless steel bolt). This is explained by the fact that the bolt with a larger diameter resulted in a greater column stiffness and the generation of larger earthquake forces in the column, and the collapse of the column did not occur due to shear failure but by the bolts pulling out. For AS, the effect of the bolt diameter was practically negligible. This is explained by the fact that this accelerogram has an impact excitation and did not introduce greater energy into the column with the larger bolt diameter in relation to the column with a smaller bolt diameter.

Fig. 11.8. Effect of the bolt diameter (ϕ_b) on the column bearing capacity ($h_b = 120$ mm)

11.4. Effect of the hole diameter

Fig. 11.9. shows the effect of the bolt hole diameter in relation to the bolt diameter, which is significant. For AA, increasing the lead bolt hole diameter from 14 mm to 22 mm column reduced the bearing capacity by approximately 21 %, and the bearing capacity of the column with stainless steel bolts was reduced by approximately 33 %. For AS, the aforementioned reductions were approximately 17 % and 21 %, respectively. This is explained by the fact that the larger hole diameter allows a greater relative displacement between adjacent blocks, which results in greater earthquake forces and vertical force eccentricities in the multi-drum column. This conclusion would probably be different in the case of adhesion between the bolt and the block.

Fig. 11.9. Effect of the hole diameter on the bearing capacity of the tested columns

$$(h_b = 120 \text{ mm}, \phi_b = 12 \text{ mm})$$

12. Conclusions

Based on the results of the research conducted within this dissertation, which is related to the effect of the considered parameters, the main conclusions below can be made regarding the behaviour and bearing capacity of multi-drum stone columns exposed to static loads and earthquakes. One of the most important global conclusions is that the type of earthquake (duration and change of acceleration in time), with the same maximum acceleration, has a significant effect on the behaviour and bearing capacity of a multi-drum stone column.

12.1. Effect of the block height

- An increase of the column block number, i.e., reducing the height of the blocks and increasing the number of joints in the column, decreases the stiffness and resistance and increases the deformation of the column. The result is a reduction in the column bearing capacity when subject to centric and eccentric static compressive forces.
- Increasing the number of blocks in the column can reduce and increase the column bearing capacity during an earthquake, depending on the earthquake type. If the earthquake has a long duration with a long predominant period, which causes larger column displacements and introduces greater energy in the column, an increase in the number of blocks in the column reduces the column's capacity. An increase in the number of blocks in the column, increases its bearing capacity for short duration earthquakes and short predominant periods (so-called impact earthquakes). In this case, the effect of earthquake force reduction is more dominant because of the column stiffness reduction which is a result of the column resistance reduction.
- The column from a monolithic block undoubtedly has a significantly greater bearing capacity than a multi-drum column for static loads and for earthquakes that induce greater displacements and greater energy. However, a multi-drum column may have a greater bearing capacity for impact earthquakes.
- An increase in the column block number increases the number of joints in the column, which are the weakest links in the column body. The joints are a place not only of column softening, but also the location with unequal transmission of compressive forces through adjacent blocks, eccentricity of the longitudinal compressive force and reduced column shear resistance.

12.2. Effect of the joint type

- A decrease in the joint stiffness, reduces the overall stiffness and increases the deformability of the multi-drum stone column under static load and earthquake. Soft joints (e.g., dry, from stone powder, and low mortar) result in a lower column stiffness than rigid joints (e.g., from strong mortar or epoxy) and, therefore, a lower bearing capacity under static load.
- A decrease in the joint stiffness reduces the multi-drum column bearing capacity during an earthquake which introduces greater energy in the column, and increases the bearing capacity for impact earthquakes.
- It is extremely important that the vertical stresses between adjacent blocks are transferred uniformly in the joints, without concentration and eccentricity. This property is believed to be ensured in so-called rigid joints, in addition to the more efficient transmission of horizontal shear stresses.

12.3. Effect of the bolt parameters

12.3.1. General

- The columns with bolts in joints have a significantly greater bearing capacity than columns without bolts.
- The contribution of the bolt bearing capacity depends significantly on the earthquake type. Specifically, as bolts have an important function in the transfer of horizontal shear forces in a column, their role is more significant in impact type earthquakes, which generate greater transverse forces than bending moments.
- Columns without bolts have significantly greater relative horizontal displacements between adjacent blocks than columns with bolts.

12.3.2. Bolt material

- The effect of the bolt material (its strength and deformability) is of less importance when the columns are exposed to smaller transverse forces, i.e., earthquakes that generate less transverse forces and greater bending moments. In other words, this effect is smaller when the column failure occurs by the bolts pulling out, but not by bolt shear failure.

- When the columns are loaded with an earthquake that generates large transverse forces in the column, in relation to the bending moments, the type of bolt material and their shear strength can have a major effect on the column bearing capacity.
- Stiffer bolts resulted in stiffer columns, which resulted in greater earthquake forces in the column, but such columns also have a greater resistance.

12.3.3. Bolt length

- The length of the bolt has no practical significance when there is no adhesion with block, i.e., when the bolt cannot resist tension.
- When the bolt can resist the tensile force, its anchorage length in the block is likely to have a significant effect on the column bearing capacity.

12.3.4. Bolt diameter

- The bolt diameter does not have a major effect when the bolt shear stresses at the column failure are less than the bolt shear strength. In this case, a bolt with a larger diameter may be less favourable than a bolt with a smaller diameter, because this situation results in a column with a greater stiffness and greater earthquake forces, and the column collapses by the bolts pulling out.
- For earthquakes that generate large transverse forces in the column, the effect of the bolt diameter can be significant.

12.3.5. Bolt hole diameter

- An increase in the bolt hole diameter in relation to the bolt diameter, increases the horizontal deformability of the column (when there is no adhesion between the bolt and the block) and, thus, the earthquake force in the column, which is unfavourable. If adhesion between the bolt and the block is ensured, this conclusion is not valid.

13. Future research

Although relatively strong global conclusions have been presented, further research is needed to achieve the objective validation of these conclusions. In this content, the future research outlined below is required, some of which is planned to be addressed:

- Effect of results obtained on small-scaled models in their application to the multi-drum columns of actual size.
- Effect of different column geometries and edge conditions, and effect of different block materials.
- Effect of the vertical component of the earthquake accelerations
- Effect of additional earthquake types
- Effect of the block connecting surface flatness
- Effect of other joint types
- Effect of interaction with other structure elements.
- The development of numerical models, their testing on the conducted experiments, and their use for the analysis of here considered and other possible effects of multi-drum columns

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PAPER [I]

Effect of the drum height on the bearing capacity of composite multi-drum column under static load

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ABSTRACT

The results of an experimental study on the effect of the drum height on the bearing capacity of composite multi-drum column under axial compression force and lateral horizontal force were presented. Small scale columns with two, four, six and twelve drums were tested. It was concluded that an increased number of drums reduces bearing capacity, flexural and shear stiffness, and increases horizontal and vertical deformability of columns. The reason is the simultaneous increase of the joints number, which are the weakest point in column. The column loaded by axial force only had a brittle failure, and by combined axial force and lateral force more ductile failure.

1. Introduction

Stone columns are important structural elements in masonry buildings. They are primarily intended to carry the vertical load, but they can also carry a smaller horizontal load. Stone columns may be formed from monolithic block and from several stone blocks (so called multi-drum column). Therefore, various heights of stone blocks and different solutions of horizontal joints between the stone blocks (dry joint, stone powder, various mortars and similar) have been used. Multi-drum column is essentially a complex composite structure, formed of stiff stone blocks and soft/deformable joints. Joints are a weak point in the bearing structure of the column. The number of joints (blocks) and the interaction between blocks and joints is one of the most important factors that affect the behaviour and bearing capacity of composite multi-drum column under different loads. Some experimental and numerical studies which are related to the multi-drum stone columns and masonry are listed below. Since topics of those studies are only indirectly related to the experimental study presented in this paper, their results are briefly cited.

Sarhosis et al. [1] utilized a discrete element method to study the static and dynamic behaviour of a typical section of the two-storey colonnade of the Forum in Pompeii. It was shown that the construction method used in ancient times, such as multi-block segmented trabeations and a solid block beam, had a significant impact on the mechanical response of the structure.

The protection of a classical column in areas of seismic activity using passive control techniques were studied by Papalou and Strepelias [2]. The damper takes the form of a drum that looks exactly

like the rest of the drums, but it is hollow and contains particles. It was found that the damper could reduce the motion of the column up to 40%.

Drosos and Anastasopoulos [3] experimentally studied the seismic performance of a free-standing reduced scale multi-drum column using a shake table. The columns tested were proven to be very earthquake-resistant.

Drosos and Anastasopoulos performed an experimental study of an ancient Greek temple using a shake table [4]. Reduced scale models of a single multi-drum column and a portal were tested. The single columns exhibited a remarkable earthquake resistance.

Lazarević et al. [5], performed a systematic study involving laboratory testing and numerical modelling to determinate the effect of contact stresses on the bearing capacity of traditional stone columns.

Papaloizou and Komodromos [6] studied the behaviour of multi-drum columns and colonnades during earthquakes using a discrete element method. The numerical simulations showed that earthquakes with relatively low predominant frequencies seemed to endanger both standalone columns and colonnades more than earthquakes with higher predominant frequencies.

Komodromos et al. [7] performed a simulation of the response of ancient multi-drum columns under harmonic and earthquake excitations by a discrete element method. A large number of parametric studies have been performed to investigate the effect of certain parameters on the response of the columns.

Konstantinidis and Makris [8] performed a numerical study to simulate the seismic response of multi-drum classical columns. The numerical model used could capture the sliding, rocking and slide-rocking

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Fig. 1. Geometric data for the columns.

response of rigid-body assemblies with fidelity.

Psycharis et al. [9] numerically studied the seismic behaviour of a part of the Parthenon Pronaos by a discrete element method, with the assumption of rigid blocks and frictional joints.

Papantonopoulos et al. [10] made a numerical prediction of the earthquake response of classical columns using a distinct element method. Their results were compared with experimental data for an identical specimen under same excitation.

Mouzakis et al. [11] reported experimental results for the earthquake response of a marble model of a classical column using a shake table. The model was a 1:3 scale replica of a column of the Parthenon on the Acropolis of Athens.

Krstevska et al. [12] performed experimental dynamic testing of a prototype and model of the Antonina column in Rome.

Foraboschi [13–16] analysed a stone columns in framework of buildings of famous Venetian architecture, and emphasizes the importance of structural design that advanced and revolutionized the architecture.

Alecci et al. performed experimental investigation: of masonry arches strengthened with PBO-FRCM composite [17], brick work columns confined with CFRP composite [18] and concrete columns confined with CFRP wraps [19].

D'Ambrisi et al. [20] experimentally and analytically investigated bond between Carbon-FRCM materials and masonry.

Ascione et al. [21] numerically analysed load carrying capacity of 2D FRP/strengthened masonry structures.

C. Blasi and P. Foraboschi [22] presented analytical approach to collapse mechanisms of circular masonry arch.

Behaviour and ultimate bearing capacity of multi-drum stone columns under load depend on a large number of parameters, such as: column geometry, boundary conditions, load characteristics, number and characteristics of joints between stone blocks, stone block height (number of blocks per column height) etc. The effects of many parameters are usually interdependent. In research studies, the effect of only

a small number of parameters is usually investigated, while all other parameters remain constant because otherwise they would be complex and expensive.

This paper presents the results of an experimental study on the effect of the drum height (number of drums i.e. number of joints) on the bearing capacity of small-scale composite multi-drum column under vertical and horizontal static forces. According to the available literature, this issue has not been experimentally investigated yet under static load. Four column types were considered, with two, four, six and twelve drums in the column. The columns were firstly tested under axial compressive force, and then under combined effect of axial compressive force and horizontal force in the middle of their height. For different constant vertical force values, the horizontal force was successively increased until column failure. Other column parameters were kept constant in all tests. The tested columns were 2.5 m in height, with hinges at the top and bottom. They had a circular cross-section with a diameter of 192 mm. The applied forces and shortening of the column, as well as the horizontal displacement and the vertical strain in the middle of the height, were measured. Three identical specimens were tested for each column type and for each load, and mean values of the experimental results are shown. Diagrams of ultimate load-bearing capacity of tested columns in relation to the vertical force-horizontal force were presented, as well as diagrams of measured displacements and strains. The most important conclusions of the study are given.

The basic data for the tested columns and test setup are shown in Section 2. The test results are presented in Section 3, and main conclusions of the research are shown in Section 4.

The main conclusions of the research are that an increase of drums (joints) in multi-drum column reduce its bearing capacity, flexural and shear stiffness, and increase its deformability. It has been experimentally confirmed that the joints are the weakest point in multi-drum column. The columns under axial force had a more brittle failure than under axial force and lateral force. It is considered that this study can be potentially useful for researches, engineers and architects.

2. Basic data for the columns

2.1. Geometry of the columns and test equipment

The geometric data of tested 1:2 small-scale columns are shown in Fig. 1. Practically, the columns were geometrically equivalent and were formed from blocks with equal cross sections and different heights. Column C1 was formed from two identical 1200 mm high blocks, column C2 from four identical 600 mm high blocks, column C3 from six identical 400 mm high blocks and column C4 from twelve identical 200 mm high blocks. All blocks are formed by precision cutting of larger samples. It is believed that cutting operations little influence the test results. Each column had a circular cross-section with a diameter of 192 mm and hinges at the top and bottom. All joints between the blocks and between the blocks and metal plates were of same type with thickness of approximately 2 mm. The total height of the columns C1, C2, C3 and C4 with metal shoes at the edges was 2658 mm, 2662 mm, 2666 mm and 2678 mm, respectively. Practically it can be considered that all columns had the same height, with a slenderness of approximately 46. The columns were analogous to a fixed console column with twice the lower height.

The testing equipment with column C3 is shown in Fig. 2. A rigid steel frame was used for carrying two-axial forces. The vertical force was applied with a hydraulic press at the bottom of the column, and the horizontal force was applied with a hydraulic press in the middle of the column height. The horizontal force was transmitted over the metal plate in the joint to avoid possible lateral damage of the column.

2.2. Blocks and joints

In order to determine the reliable effect of the stone block height on the behaviour of multi-drum stone columns, it is crucial that all used blocks have the same properties. Otherwise, different block properties could significantly affect the measured values and presented conclusions. Unfortunately, it has always been difficult to ensure equal

Fig. 3. Stress (σ) – strain (ϵ) relationship of concrete in compression.

properties of all used stone blocks due to often presence of different stone anomalies.

As there was no guarantee that all the stone blocks required for this research will be with the same characteristics (without anomalies), concrete blocks were used instead of stone blocks - which were made of the same mixture, at the same time and used on the same way.

The used concrete was made from limestone aggregate with a maximal grain of 8 mm (1613 kg/m^3) and compressive strength of 810 MPa, white Portland cement CEM I 42.5 R (320 kg/m^3), stone powder (150 kg/m^3) and superplasticizer (6.4 kg/m^3), with a water/binder ratio of 0.44. All columns were tested during the period from 46 to 48 days after casting, as well as the compressive and tensile strength of concrete specimens.

The compressive strength of the concrete was tested on a concrete block with a diameter of 192 mm and a height of 400 mm, according to [23]. Fig. 3 shows the average stress-strain relationship for three tested samples. The average compression strength of the concrete was 53.55 MPa, the average modulus of elasticity was 33.11 GPa and the average compressive strain at the failure was -3.31% . The splitting tensile strength of the concrete was determined on a specimen with diameter of 192 mm and length of 200 mm according to [24], and measured as 5.99 MPa.

Joints between stone blocks of multi-drum columns in historic stone buildings at Croatia coastal area are mostly dry (direct contact of stone surfaces), filled with mortar and sometimes filled with stone powder. Dry joints were used only at precise processed stone blocks contact surfaces. Joints filled with mortar or stone powder enable reduction of stress concentration on non-precise processed stone blocks connection surfaces, as well as achieving precise verticality of a column. Since joint surfaces of used concrete blocks were not ideally flat, it was decided to use certain material in the joints. Joints with stone powder were adopted, which in relation to the joints with mortar accelerates the specimen production and shortens the research duration. There is usually a small difference in the compressive strength of conventional mortar and stone powder. A stone powder was used with a maximum grain of 0.063 mm and a thickness of about 2 mm (Fig. 4), obtained from stone aggregates used for production of concrete blocks. However, because of compressibility of these joints, it was practically impossible to avoid imperfections and stress concentrations at the no-ideal flat contact surfaces of the concrete blocks. The stone powder is also placed between the concrete blocks and metal plates. The construction of tested columns was performed in the same way as the construction of usual multi-drum columns in practice.

Legend:

- 1- Rigid steel frame
- 2- Hydraulic press for causing vertical force
- 3- Hydraulic press for causing horizontal force
- 4- Vertical force gauge
- 5- Horizontal force gauge
- 6- Displacement gauge
- 7- Vertical strain gauge
- 8- Steel hinged joint
- 9- Column
- 10- Steel sheet
- 11- Joint

Fig. 2. Testing equipment.

Fig. 4. Schematic presentation of the joint between the drums.

Fig. 5. The scheme of the columns load and assumed V-H relation.

Fig. 6. Parameters measured.

2.3. Load of the columns

The scheme of the column loads is presented in Fig. 5. Firstly, the

Fig. 7. Ultimate bearing capacity of tested columns in a V-H relationship.

Fig. 8. Vertical displacement of the column bottom for $V = 500$ kN.

Fig. 9. Vertical displacement of a column bottom at failure.

columns were tested to axial compression force V , with force increments of approximately 5 kN/s up to column failure. Thus, the maximum force V_{max} was determined, which column can carry. Subsequently, all the columns were tested for the combined action of V and H

Fig. 10. Vertical compressive strains for acting of vertical force alone.

Fig. 11. The average vertical strain ϵ_{1-2}^v axially loaded columns.

forces. It was not known from the available literature that the multi-drum stone columns was experimentally tested on simultaneous acting of forces V and H. In the performed test, for each adopted constant compressive force V (100 kN, 200 kN, 300 kN, 500 kN, 800 kN), the horizontal force H was applied with an increase of 0.5 kN/s up to column failure.

2.4. Measured parameters and measurement equipment

The measured parameters are shown in Fig. 6. Applied forces (V, H), horizontal displacement in the middle of the column (u) and vertical displacement at the bottom of the column were measured for all columns, and even the vertical strains (ϵ_1^v , ϵ_2^v) of a block in the middle of the axially loaded column. The forces were measured with force gauges (KDB force transducer, HBM), displacements with displacements gauges (type LWA, Novotechnik) and strains with strain gauges (type 100/120

LY41, HBM).

3. Experimental results

The mean values of the measured results for three identical tests are presented. The ultimate bearing capacity of columns in a V-H relationship is shown in Fig. 7.

3.1. Results for the acting of axial force only

For the bare acting of axial compressive force V, the ultimate bearing capacity of the columns C1, C2, C3, C4 was 1138.1 kN, 985.4 kN, 827.2 kN, 684.8 kN, respectively. The ratio of the limit compressive force for the columns was C1: C2: C3: C4 = 1: 0.87: 0.73: 0.60. It is obvious that increasing the number of blocks in the column reduces its bearing capacity. Therefore, in the test, the ultimate bearing

Fig. 12. Photography of the column C1 at failure under axially force.

capacity of the column with twelve blocks was only 60% of the capacity of the column with two blocks.

The reduction of the column's bearing capacity by increasing the number of blocks can be explained as follows. Larger number of blocks in the column creates a larger number of horizontal joints. Horizontal joints are usually the weakest part of the column. In the plane of adopted joints, there is no complete and equal transmission of stress from one block to another. Additionally, it is difficult to avoid the eccentricity of the axial force and the concentration of compressive stresses in the joints. Vertical compaction occurs in the joint and the horizontal deformability of a block near the joint is higher than in the rest of the block. By increasing the number of joints, the probability that the column will not be completely vertical is also increased i.e., it increases the probability of additional eccentricity of the axial force in the column.

Fig. 8 shows the vertical displacement at the bottom of a column with equal vertical force $V = 500$ kN. The large difference in the results is a consequence of the column's compaction in the joints. Columns with more joints (blocks) have higher shortening.

Fig. 9 shows the vertical displacement at the bottom of a column at failure. The displacement values are a consequence of joint compaction and shortening of the blocks due to the action of the vertical force.

The vertical compressive strains ε_1^v , ε_2^v in the columns are shown in Fig. 10. In an ideal state, they should be equal in all the columns for an equal force V . However, they are different in all columns already at the start of acting of the force V , which is caused by the uneven seating of the blocks and deviations from the verticality of the column axis, i.e., by the eccentricity of the axial forces (stresses) on the column axis. The larger strains of the column (ε_1^v ili ε_2^v) are on its concave side due to the lateral deformation. Depending on the imperfections in the joints, column under axial force can deform laterally to the left or right (Fig. 2). Some permanent strains occurred after the failure of all columns.

The average vertical strain of the columns $\varepsilon_{1-2}^v = 0,5(\varepsilon_1^v + \varepsilon_2^v)$ due to force $V = 500$ kN is shown in Fig. 11a, and the maximum strain at column's failure in Fig. 11b. It is evident that it is approximately same

for all columns under the same vertical force and that it is increased by decreasing the number of blocks in columns at failure, i.e., by increasing the failure force.

The failure of all columns was explosive (Fig. 12), without visually detectable lateral deformation. Failure mostly occurred in the middle of the columns' height (the position of the highest effect of slenderness and lateral displacement), and at the edges of some columns (stress concentration below the metal plate at the point of force V action). By increasing the compression force increases the shortening of the column and its compression strain. The yielding and crushing of materials appear just before the column failure. At the same time, there is an increase of transverse tensile deformation in blocks. There was a vertical splitting of some blocks at column failure, wherein the splitting plane was near the middle of the columns' cross section.

3.2. Results for combined action of vertical and horizontal forces

The curves for the ultimate bearing capacity of the columns during the simultaneous action of vertical compressive force V and horizontal force H are shown in Fig. 7. The bearing capacity of the columns under the horizontal force strongly depends on the level of the vertical force. It is evident that the columns can carry a small horizontal force in comparison to the vertical force. The V-H relationship is similar to the relationship for the reinforced concrete column, however the bearing capacity under the horizontal force is significantly reduced because tensile stresses cannot be transmitted at the level of joints. For the same vertical force, the bearing capacity under a horizontal force decreases with an increase in the number of blocks/joints in the column. Analogously to the bearing capacity under axial force only, column C1 endured the highest horizontal force and column C4 endured the lowest.

The relationship between the horizontal force (H) and the horizontal displacement (u) in the middle of the column height is shown in Fig. 13. The horizontal displacement of the column depends on its bending stiffness. In this study, it depended on the number of joints in the column. For the same vertical force V , the lowest bending stiffness and the highest horizontal displacements was for column C4 and the highest bending stiffness and lowest horizontal displacements for column C1. It is evident that bending stiffness of the columns increases by increasing the compression force V . For the same horizontal force, lower horizontal displacements occurred for columns with higher vertical force. By increasing the vertical force up to $V = 500$ kN, the ultimate bearing capacity of the column under horizontal force is increased, as well as the corresponding horizontal displacement. After reaching the maximum horizontal force, the measured horizontal displacements should be taken with a reserve.

The H- u relationship for the applied vertical force V is shown in Fig. 14, which is analogous to Fig. 13. It is evident that for all levels of the force V , the highest stiffness occurred in column C1, i.e. that stiffness of the column decreased with an increase of number of blocks/joints in it. Horizontal joint deformability also affected the column horizontal displacement, especially for low vertical force.

The collapse of the columns with the simultaneous action of vertical and horizontal force was considerably less brittle and explosive than under vertical force only, especially under lower vertical force (Fig. 15). The plane of the column splitting at failure was closer to the edge than to the middle of the column cross-section, especially for lower force V .

4. Conclusion

Based on performed experimental tests of composite multi-drum columns of equal height and different number of concrete blocks per column height (different height of drums i.e. different number of joints) under axial and horizontal static force, with thin layers of stone powder between drums, the following can be concluded.

Generally, an increase of number of drums per height of the tested multi-drum columns reduces their ultimate bearing capacity, flexural

Fig. 13. Horizontal force (H) - horizontal displacement (u) relationship.

and shear stiffness, and increases their horizontal and vertical deformability. Bearing capacity ratio of the tested column on the axial force with two, four, eight and 16 blocks per height was 1: 0.87: 0.73: 0.6, respectively.

The cause of the above mentioned is the simultaneous increase of the joint number in the column, which are the weakest points in it. Even with great attention in processing of the block connecting surfaces, joint formation and block installation, it is difficult to avoid stress concentrations and irregular transmission of compression through the joint, as well as to ensure the verticality of the blocks and of the entire column. The normal tensile stresses cannot be transmitted over the joint, and the efficiency of the transmission of horizontal shear stresses is reduced.

The columns loaded by bare vertical compressive force had a brittle failure. The columns loaded simultaneously by vertical and horizontal forces had more ductile failure, especially under low vertical force.

The presented conclusions may be applied on columns with dry joints. This is based on the results of experimental study of joint type effect on the behaviour and ultimate bearing capacity of multi-drum columns under static load, performed by Buzov et al. [25].

Since stone columns can carry small horizontal forces, stiffer construction elements (walls, vaults) should be used for this purpose in actual buildings.

The presented data of the performed experimental tests can be useful in verification of numerical models for static analysis of multi-drum stone columns.

In practice, engineers and architects should prefer multi-drum columns with the less number of blocks (joints) by their height if they are dominantly loaded by static load. However, according to the author's current research results, the conclusion regarding the effect of drum numbers on multi-drum column bearing capacity under earthquake depending on the earthquake type.

Fig. 14. Relationship $H - u$ for different levels of force V . Fig. 15. Photography of the column C3 at the failure under acting of V and H .

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PAPER [II]

Research Article

Effect of the Drum Height on the Seismic Behaviour of a Free-Standing Multidrum Column

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The results of a shake-table study on the effect of the drum height on the seismic behaviour and bearing capacity of small-scale free-standing multidrum columns are presented. Columns of equal height with one, three, and six drums through their height were considered for the case of their self-weight only and for the case with an additional weight on the top of the column. The columns were exposed to a horizontal base acceleration of three accelerograms by successively increasing the maximum acceleration to their failure. The characteristic displacements and accelerations of the column were measured. It was concluded that an increase in the number of blocks in the column can significantly increase or decrease its ultimate bearing capacity, depending on the type of the applied accelerogram. It is expected that the experimental database can be useful in the validation of nonlinear numerical models for the dynamic analysis of multidrum columns.

1. Introduction

Multidrum stone columns are important structural elements of historic buildings. Their behaviour during an earthquake depends on numerous parameters, such as the number of blocks, boundary conditions at the top and bottom of the column, the type and processing of joints, the level of gravitational stresses, stone quality, interaction with possible walls, slenderness, precision of construction, foundation deformability, local effects, and earthquake type.

Because of the great importance of historical buildings, the studies regarding the seismic behaviour of stone structures, especially multidrum stone columns, are still up to date.

Unfortunately, only a small number of numerical and especially experimental investigations of multidrum stone columns under an earthquake have been conducted to date. In doing so, only some parameters that affect their seismic response and bearing capacity have been investigated. Several of these studies are briefly listed below.

Contemporary numerical studies on the seismic behaviour of multidrum columns are mainly based on the discrete element method. Using this method, Sarhosis et al. [1] analysed

the colonnade of the Forum in Pompeii; Papaloizou and Komodromos [2] analysed multidrum ancient columns and colonnades; Komodromos et al. [3] investigated ancient columns; Konstantinidis and Makris [4] considered a multidrum classical column; Psycharis et al. [5] analysed part of the Parthenon Pronaos; Papantonopoulos et al. [6] analysed classical columns; Sarhosis et al. [7] performed three-dimensional modelling of ancient colonnade structural systems subjected to harmonic and seismic loading; and Asteris et al. [8] analysed historic masonry structures.

Contemporary experimental studies regarding the seismic behaviour of multidrum columns are based on shake-table testing. Thus, Drosos and Anastasopoulos [9] studied a free-standing reduced-scale multidrum column; Drosos and Anastasopoulos [10] investigated an ancient Greek temple; Mouzakis et al. [11] analysed a marble model of a classical column; and Krstevska et al. [12] analysed a model of the Antonina column in Rome. The above-mentioned numerical and experimental studies have not been described more closely because they are not considered in this article.

This paper presents the experimental study results regarding the effect of the number of blocks in a multidrum

FIGURE 1: Tested free-standing columns: (a) self-weight only and (b) self-weight and mass at the top.

column on its seismic behaviour. The behaviour of small-scale free-standing multidrum columns without and with the mass at the top was studied by shake-table testing. The columns were exposed to the horizontal base acceleration of three different accelerograms. Columns made from one, three, and six blocks per their height with stone powder joints were considered. In addition to the new insights of the effect of the drum height on the seismic behaviour of multidrum columns, one of the aims of this study was to create an experimental database for the possible validation of nonlinear numerical models for the seismic analysis of such columns. The main conclusions of the study are given at the end of the article. According to the authors' knowledge, such research has not been performed to date. It is believed that the given conclusions can be applied to multidrum columns in practice.

Summarized findings of the study are as follows:

- (i) An important effect of the number of blocks in the multidrum column on its seismic response was investigated, which has not been investigated yet.
- (ii) A reliable experimental approach to research considered problem with the shake-table has been adopted, which can be easily repeated.
- (iii) Column models were tested to the failure on different earthquake types, with and without mass at their top.
- (iv) The results of the research are applicable in practice.
- (v) Generated experimental database can be useful in verifying numerical models for seismic analysis of multidrum columns.

1.1. Tested Columns. The small-scale free-standing multidrum columns (Figure 1) with approximately equal height formed from one, three, and six blocks per column height were tested. The columns S1, S3, and S6 (Figure 1(a)) were loaded only by self-weight, and the identical columns S1m, S2m, and S6m (Figure 1(b)) were loaded with additional

TABLE 1: Fundamental free oscillation period T_1 of the tested columns.

	S1	S3	S6	S1m	S3m	S6m
T_1 (s)	0.054	0.072	0.152	0.179	0.232	0.507

weight at their top. The columns have a circular cross section with a 192 mm diameter. These columns were supported on a shake table indirectly over a 50 mm thick, rigid concrete plate fixed to the shake table. Thin joints (1–3 mm) from stone powder were formed between the blocks, between the block and the bottom plate, and between the block at the top of the column and mass (m). Such joints were used to reduce the stress concentration in the nonideal flat dry joint, that is, to ensure an equal adjustment of the contact surfaces between the blocks and an equal transmission of vertical and horizontal stresses in the joint, as well as to simplify the column construction. The type of the joint also affects the behaviour of the multidrum columns. This issue was investigated by Buzov et al. [13] for static loads. The height of the columns was approximately 1200 mm, and their slenderness l was approximately 35. The mass at the top of the column ($m = 500$ kg), which is centrally positioned in relation to its axis, was formed from a rigid concrete block with dimensions of $0.8 \times 0.8 \times 0.33$ m.

The columns S1 and S1m were formed from one 1200 mm high block, the columns S3 and S3m from three 400 mm high blocks, and the columns S6 and S6m from six 200 mm high blocks. The blocks were made of concrete with a compressive strength of 53.55 MPa, obtained on the cylinder according to [14]. Although the subject of this study is stone columns, concrete blocks were used instead of stone blocks for several reasons [13, 15]: (i) unreliability of the uniformity of the stone quality (the probable presence of anomalies in the stone), (ii) great similarity of stone and concrete properties (artificial stone), (iii) low level of maximum stress in the blocks (the collapse of a column is due to the loss of its stability), and (iv) lower research costs. This approach did not greatly influence the given conclusions of this study.

FIGURE 2: Applied horizontal base accelerations: (a) artificial accelerogram; (b) Petrovac accelerogram; (c) Ston accelerogram.

The experimentally determined first free oscillation period T_1 of the considered columns is shown in Table 1. The significant effect of the number of blocks in the column and the mass at the column top on T_1 is evident. By increasing the number of blocks, T_1 increases, that is, the column stiffness decreases. The mass at the top of the column also increases T_1 . The relation of the basic period for the columns S1, S3, and S6 is 1:1.33:2.82, and the relation for the columns S1m, S3m, and S6m is 1:1.30:2.8.

1.2. Applied Accelerograms. The tested columns from Figure 1 were subjected to a horizontal base acceleration of the three accelerograms according to Figure 2: (i) the artificial accelerogram (Figure 2(a)) generated from the elastic response spectrum in accordance with [16] for type 1 and soil type A; (ii) the N-S horizontal component of the Petrovac earthquake

[17], Montenegro (Figure 2(b)); and (iii) the N-S horizontal component of the Ston earthquake [17], Croatia (Figure 2(c)). The spectral values of the adopted accelerograms are presented in Figure 3. The artificial and Petrovac accelerograms are characterized by long-lasting action, where the predominant period of the Petrovac accelerogram is somewhat longer. The Ston accelerogram is characterized by a short impact action with a short predominant period. By considering the basic periods of the columns from Table 1 and the spectral values of the applied excitations from Figure 3, it is expected that the artificial and Petrovac accelerograms are more unfavourable than the Ston accelerogram. However, due to the possibility of the occurrence of nonlinearities in the joints (sliding and rotation), it is difficult to predict which accelerogram would be the most unfavourable for each column in Figure 1.

All the accelerograms in Figure 2 were applied with a successive increase in the maximum acceleration a_{gmax} .

FIGURE 3: Spectral values for the adopted accelerograms: (a) accelerations; (b) velocities; (c) displacements.

until the collapse of the column. For the artificial accelerogram and the Petrovac accelerogram, the increment in the acceleration $\Delta a_{g\max}$ was adopted as 0.2 ms^{-2} , and $\Delta a_{g\max}$ for the Ston accelerogram was employed as 0.5 ms^{-2} . Although only three different base accelerations were adopted to shorten the research, it is believed that this has no practical influence on the given conclusions.

1.3. Measured Quantities and Equipment. The horizontal displacements u_i of the top and bottom of the blocks (5 mm from their edges) for each tested specimen were measured according to Figure 4, as well as the horizontal acceleration a_i in the middle of the drum height. The motion of the columns during base excitation was recorded by a precise video camera.

The shake table at the University of Split, Faculty of Civil Engineering, Architecture and Geodesy, was used. A QuantumX MX840A system (HBM) was used to record the data from the sensors. Analogue displacement sensors PB-25-S10-NOS-10C (UniMeasure) were used for the measured displacements, and the accelerations were measured by a type 4610 piezoelectric high-frequency accelerometer (MS).

1.4. Test Results. Before the final tests were performed, the accuracy of the recurrent tests was measured to verify their reliability. Specifically, for the S3 and S3m columns, three identical tests were performed for the artificial and Petrovac accelerograms. The maximum difference in the measured

FIGURE 4: Measured quantities.

values for each test was up to 4.8%. Given the sensitivity of the problem and the possible effect of different parameters (column verticality, inhomogeneity in joints, mass eccentricity at the top of the column, precision of the acceleration application by the shake table, imperfections in the measurement equipment and measurements, etc.), it was suggested that these deviations are within acceptable limits and have no practical influence on the given conclusions. Therefore, because of the simple presentation of the measured values, the final tests were conducted on a single specimen.

Only some test results are graphically presented and briefly discussed. The maximum base acceleration a_{gmax} values for the columns S1, S3, and S6 in their limit state are shown in Figure 5, where the full line indicates a_{gmax} at the collapse of the columns and a dotted line indicates a_{gmax} from the previous acceleration increment with the preserved stability of the columns. It is evident that the column bearing capacity depends on the number of blocks in the column and the applied accelerogram. For the artificial accelerogram, by increasing the number of blocks in a column, its ultimate bearing capacity decreases. For the Petrovac and Ston accelerograms, by increasing the number of blocks in a column, its ultimate bearing capacity increases. Increasing the number of blocks in a column, that is, increasing the number of joints in the column, decreases not only its static stiffness but also its dynamic stiffness. Therefore, lower inertial (earthquake) forces are generated, but at the same time, the column also has a lower resistance. The reduction in the bearing capacity of the column by increasing the number of blocks for the artificial accelerogram can be explained by the fact that this excitation introduced a higher energy in the column. Specifically, this accelerogram has higher spectral displacements and velocities for a higher T_1 (Figure 3.) than those of the Petrovac accelerogram and especially the Ston accelerogram. The maximum bearing capacity for the Ston accelerogram can be explained by the fact that this excitation has a short impact activity and introduces the lowest energy of the earthquake in the columns; that is, it has the lowest spectral velocities and displacements for a higher T_1 .

For the columns S1m, S3m, and S6m with a mass at the top, the maximum base acceleration a_{gmax} in their limit state

FIGURE 5: Maximum base acceleration a_{gmax} in the limit state of the columns S1, S3, and S6. FIGURE 6: Maximum base acceleration a_{gmax} in the limit state of the columns S1m, S3m, and S6m.

is shown in Figure 6. The full line indicates a_{gmax} at the failure of the columns, and a dotted line indicates a_{gmax} from the previous acceleration increment with the preserved stability of the columns. By comparing Figures 5 and 6, a similar behaviour of the columns with a mass at their top and the columns without mass is notable. However, the bearing capacity of the columns S1m, S3m, and S6m is smaller due to the effect of the mass at their top (a significant increase in seismic forces but also an increase in the stability of the column). The similarity of the columns' behaviour for a particular accelerogram is also noticeable. Namely, for the artificial accelerogram, the bearing capacity of the column was also reduced with an increase in the number of blocks, while for the Petrovac and Ston accelerograms, an increase in the number of blocks increased the column's bearing capacity. The explanations are analogous to those for the columns S1, S3, and S6.

FIGURE 7: Horizontal displacements u_1 and u_2 of the column S1 for the artificial accelerogram at some a_{gmax} .

FIGURE 8: Horizontal displacements u_1 and u_2 of the column S1m for the artificial accelerogram at some a_{gmax} .

FIGURE 9: Horizontal displacements u_1 to u_6 of the column S3 for the Petrovac accelerogram at some a_{gmax} . FIGURE 10: Horizontal displacements u_1 to u_6 of the column S3m for the Petrovac accelerogram at some a_{gmax} .

FIGURE 11: Horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 , and u_{12} of the column S6 for the Ston accelerogram at some a_{gmax} .

The horizontal displacements u_1 and u_2 of the column S1 for the artificial accelerogram for some a_{gmax} are shown in Figure 7. It is evident that there were no major displacements in the column up to approximately $a_{gmax} = 1.6 \text{ ms}^{-2}$. By increasing a_{gmax} , there was ever-increasing reversible and irreversible displacements of the column. At $a_{gmax} = 2.8 \text{ ms}^{-2}$ (excitation before the column's collapse), the irreversible horizontal displacement of the column's bottom was approximately 60 mm and that of the column's top was approximately 50 mm (the column remained in an inclined position). The largest relative displacement of the column's top in relation to its bottom for $a_{gmax} = 2.0 \text{ ms}^{-2}$ was approximately 80 mm.

Figure 8 shows the horizontal displacements u_1 and u_2 of the column S1m for the artificial accelerogram for several a_{gmax} . At approximately $a_{gmax} = 0.6 \text{ ms}^{-2}$, the column did not demonstrate large displacements. For the larger a_{gmax} values, the column's top displacements were significantly increased. At $a_{gmax} = 1.2 \text{ ms}^{-2}$, there were small irreversible displacements of the column's top and bottom. The largest relative displacement of the column's top in relation to its bottom was approximately 40 mm.

The horizontal displacements of the column S3 for the Petrovac accelerogram for some a_{gmax} are shown in Figure 9. The column's displacements were small at approximately $a_{gmax} = 1.2 \text{ ms}^{-2}$. The maximum displacement of the column's

top for $a_{gmax} = 1.6 \text{ ms}^{-2}$ (approximately 100 mm) was larger than that for $a_{gmax} = 2.4 \text{ ms}^{-2}$ (approximately 70 mm). After the strongest excitation for which the column remained in balance ($a_{gmax} = 2.4 \text{ ms}^{-2}$), there were almost equal permanent displacements of all column blocks (approximately 10 mm), that is, the column remained almost vertical.

Figure 10 presents the horizontal displacements of the column S3m for the Petrovac accelerogram for some a_{gmax} . The large permanent displacements occurred already at $a_{gmax} = 0.4 \text{ ms}^{-2}$. During the excitation before the column's failure ($a_{gmax} = 2.0 \text{ ms}^{-2}$), the permanent displacement of the column's top was approximately 18 mm. Thus, the column's bottom remained in the initial position before testing.

Some horizontal displacements of the column S6 for the Ston accelerogram for some a_{gmax} are shown in Figure 11. The other displacements are not presented for the clear presentation of drawings. The column demonstrated large reversible displacements already at $a_{gmax} = 3.0 \text{ ms}^{-2}$ but practically without any permanent displacements. At $a_{gmax} = 6.0 \text{ ms}^{-2}$, the top of the column had a large permanent displacement of approximately 120 mm, and the tallest block exhibited large permanent displacements.

Figure 12 presents several horizontal displacements of the column S6m for the Ston accelerogram at some a_{gmax} . The permanent displacements of the column occurred

FIGURE 12: Horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 , and u_{12} of the column S6m for the Ston accelerogram at some a_{gmax} .

FIGURE 13: Continued.

FIGURE 13: Appearance of the column S6 for the Ston accelerogram at $a_{g\max} = 6.0 \text{ ms}^{-2}$ at different periods of time: (a) $t = 2.8 \text{ s}$, (b) $t = 3.4 \text{ s}$, (c) $t = 3.8 \text{ s}$, (d) $t = 4.9 \text{ s}$, (e) $t = 5.6 \text{ s}$, (f) $t = 6.1 \text{ s}$, (g) $t = 6.4 \text{ s}$, and (h) $t = 6.7 \text{ s}$.

FIGURE 14: Appearance of the column S6m for the Ston accelerogram at $a_{g\max} = 3.5 \text{ ms}^{-2}$ at different periods of time: (a) $t = 2.4 \text{ s}$, (b) $t = 4.5 \text{ s}$, (c) $t = 5.5 \text{ s}$, (d) $t = 6.2 \text{ s}$, (e) $t = 7.0 \text{ s}$, (f) $t = 7.9 \text{ s}$, (g) $t = 8.9 \text{ s}$, and (h) $t = 9.9 \text{ s}$.

already at $a_{g\max} = 1.0 \text{ ms}^{-2}$, which did not significantly increase in the following stronger excitation. The largest displacement of the column's top at $a_{g\max} = 3.5 \text{ ms}^{-2}$ was approximately 70 mm.

Some photographs of the column S6 for the Ston accelerogram at $a_{g\max} = 6.0 \text{ ms}^{-2}$ at a certain time according to Figure 11 are shown in Figure 13, and several photographs of the column S6m for the same excitations at $a_{g\max} = 3.5 \text{ ms}^{-2}$ at

a certain time in accordance with Figure 12 are shown in Figure 14. Notably, the relative displacements between the blocks were smaller for the column S6m than for the column S6.

2. Conclusions

Based on the results of the performed shake-table study on the behaviour and ultimate bearing capacity of the free-standing small-scale multidrum columns (S1, S3, and S6; S1m, S3m, and S6m) formed from one, three, and six blocks through their height with stone powder joints with variations in the columns gravity load (S1, S3, and S6: only self-weight; S1m, S3m, and S6m: self-weight and 500 kg mass at the top) and three different successively increased horizontal base accelerations (artificial, Petrovac, and Ston accelerograms), the main conclusions given below can be drawn.

The number of blocks in the multidrum column significantly affects its stiffness, resistance, and bearing capacity under an earthquake load.

The columns loaded with self-weight only (S1, S3, and S6) and the identical columns with a mass at their top (S1m, S3m, and S6m) had a similar effect to the number of blocks on their bearing capacity. However, the columns with the mass at their top had a lower bearing capacity because of additional seismic (inertial) forces caused by the mass.

The effect of the number of blocks in the column on its bearing capacity was dependent on the applied accelerogram (earthquake type). For an artificial accelerogram, the column's bearing capacity was reduced with the increase in the number of blocks in the column. The ratios of bearing capacity of the columns were approximately $S1:S3:S6=1:0.71:0.64$ and $S1m:S3m:S6m=1:0.83:0.66$. For the Petrovac and Ston accelerograms, by increasing the number of blocks in the column, the column's bearing capacity was increased. The ratios of the bearing capacity for the Petrovac accelerogram were $S1:S3:S6=1:1.22:1.33$ and $S1m:S3m:S6m=1:1.42:1.57$. The ratios of the bearing capacity for the Ston accelerogram were $S1:S3:S6=1:1.42:1.71$ and $S1m:S3m:S6m=1:1.16:1.16$.

The abovementioned relationships are related to the fact that increasing the number of blocks in a column reduces its stiffness and generates earthquake forces but also decreases the column resistance. The artificial accelerogram is characterized by long-lasting action with high spectral velocities and displacements for longer T_1 , that is, the great earthquake energy. This is considered the cause of decreasing the column's bearing capacity by increasing the number of blocks in it, that is, by decreasing its stiffness.

The Petrovac accelerogram has a shorter duration than the artificial accelerogram with lower spectral velocities and displacements for soft structures. The Ston accelerogram has a short impact action with the lowest spectral velocities and displacements, that is, with the lowest earthquake energy. The abovementioned is the reason why for the Petrovac and Ston accelerograms, an increase in the number of blocks in a column resulted in a greater bearing capacity.

We hope that the obtained experimental database can be useful for the validation of numerical models for the dynamic analysis of multidrum columns.

Although the aforementioned conclusions are based on the experimental study of small-scale models, such an approach may be accompanied with a number of problems and dilemmas. However, it is believed that this experimental approach can reliably be applied to similar full-scale columns because only the relative effect of a single parameter was considered (the number of blocks in a column), while all other influential parameters were kept equal in all the tests.

Further detailed and more comprehensive research of the subject matter is needed to produce more reliable conclusions. The effect of the drum height on the behaviour of multidrum columns exposed to static forces was performed by Buzov et al. [15].

Data Availability

All data underlying the findings of the study are presented in this paper.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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Effect of the joint type on the bearing capacity of a multi-drum column under static load

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Effect of the joint type on the bearing capacity of a multi-drum column under static load

Abstract

The results of an experimental study on the effect of the joints between the blocks on the ultimate bearing capacity of a multi-drum column loaded to centric vertical force and horizontal force in the middle of its height are shown. The column is approximately 2.5 metres high, with one hinge at the top and another hinge at the bottom. Four types of joints between the blocks were considered: a dry joint (column C-DJ) and joints with stone powder (column C-SPJ), lead (column C-PBJ) and epoxy (column C-EPJ). The applied vertical and horizontal forces, horizontal displacement, vertical strains and horizontal circumferential strains in the middle of the column height, as well as shortening of the column, were measured. Under axial compression, ratios between the ultimate load bearing capacities of tested columns were $C\text{-EPJ} : C\text{-DJ} : C\text{-SPJ} : C\text{-PBJ} = 1 : 0.68 : 0.59 : 0.51$. The bearing capacity of the tested columns with regard to the horizontal force depended on the level of the applied centric compression force. Columns with soft joints (C-PBJ, C-SPJ) had the largest shortening and the largest horizontal displacements for the equal forces.

Keywords: Multi-drum column, joint effect, static load, bearing capacity, deformability.

1. Introduction

Stone columns are important structural elements in historical stone buildings. Lower stone columns are usually built from a single monolithic stone block, and higher stone columns are usually built from several stone blocks through the height (so-called multi-drum columns). In multi-drum columns, different solutions of horizontal joints between blocks are used: dry joint, stone powder, various mortars, lead, etc.

In general, previous studies on the behaviour and numerical modelling of stone columns under different loads (static, dynamic) and actions (temperature changes) are still quite rare. The largest number of recent research studies is related to the effect of an earthquake on the stone columns in historic buildings. Some studies regarding multi-drum stone columns are shown below. Some of them are related to the experimental studies on the behaviour of columns under static and dynamic / seismic load. Also some studies consider numerical analysis of single-block and multi-drum columns under static and dynamic / seismic load.

Atalić et al., 2008. investigated the influence of the rotational stiffness between column elements on the global stability of historical constructions. Lazarević et al., 2010. experimentally and numerically analysed traditional column connections with the possible retrofit concept. Atalić et al., 2012. studied the influence of contact stresses on the bearing capacity of traditional stone columns. Foraboschi, 2017. and 2016. provided scientific contributions about the structural behaviour of stone columns under vertical loads and stone elements in masonry buildings. Those papers prove that past architects had the ability of choosing between brick and stone according to the specific conditions. Foraboschi and Vanin, 2014. studied the behaviour of masonry columns under vertical loads, and Foraboschi and Vanin, 2013. proposed a method for nonlinear static analysis

of masonry buildings based on a strut-and-tie modelling. Krstevska et al., 1996. performed experimental dynamic testing of a model of the Antonina column in Roma. Experimental investigation of the earthquake response of a model of a marble classical column was performed by Mouzakis et al., 2002. Drosos and Anastasopoulos, 2015. experimentally investigated the seismic response of classical temple columns. Drosos and Anastasopoulos, 2014. performed shaking table tests of multi-drum columns and portals. Papalou and Strepelias, 2015. analysed the dynamic response of classical columns with defects. Stability of slender masonry columns with circular cross-section under their own weight and eccentric vertical load was numerically analysed by Gurel, 2016. Seismic response parametric study of ancient rocking columns was performed by Kavvadias et al., 2017. Pappas et al., 2016. described the creation of a seismic vulnerability assessment form for free-standing columns based on a discrete element method simulations. Housner, 1963. described the behaviour of inverted pendulum structures during earthquakes. Numerical evaluation of the seismic response of single-block and multi-block monumental columns was performed by Mitsopoulou and Paschalidis, 2001. Numerical prediction of the earthquake response of classical columns using the distinct element method was described by Papantonopoulos et al., 2002. Psycharis et al., 2000. performed parametric investigation of the stability of classical columns under harmonic and earthquake excitations. Psycharis et al., 2003. numerically studied the seismic behaviour of a part of the Parthenon Pronaos. Seismic response analysis of a multi-drum classical column was studied by Konstantinidis and Makris, 2005. Komodromos et al., 2008. simulated the response of ancient columns under harmonic and earthquake excitations. Papaloizou et al., 2008. numerically analysed ancient multi-drum columns with epistyles under dynamic loadings. Planar investigation of the seismic response of ancient columns and colonnades with epistyles

using custom-made software was performed by Papaloizou et al., 2009. Sarhosis et al., 2016. studied the stability of colonnade structural systems under static and dynamic loading conditions.

Because the research presented in this paper is related to the study on the effect of some parameters on the behaviour of multi-drum columns and such research has not been investigated yet, there is no need to provide a detailed description of the results from the above-specified literature.

The type and characteristics of the joints between the blocks undoubtedly affect the behaviour of a multi-drum column under static load and earthquake. According to the available literature, this issue has not been investigated more closely and therefore further research on this effect is welcome. Foraboschi and Vanin, 2015. presented the results of an experimental study that analysed the effect of several joints between composite floor and vertical structures, but not between the blocks of a multi-drum column.

This paper presents the results of experimental studies considering the effect of four types of joints between the blocks on the behaviour and bearing capacity of multi-drum column under both vertical and horizontal static load. Dry joints and joints with stone powder, soft lead sheet and strong epoxy glue were considered. Each column was tested to the failure in different relations of vertical and horizontal force (centric and eccentric compression). Three identical specimens were tested for each joint type and load. In order to shorten the article and provide greater transparency of presented drawings, mean values of the measured quantities on three identical specimens are shown. In order to illustrate the dissipation of the experimental results, some measured values on three identical specimens for only one joint type are presented additionally. The most relevant conclusions of the study are listed at the end of the paper.

It is believed that the results of the study could help to better understand the effect of joint type on the behaviour of multi-drum columns under centric and eccentric compression, and that they may be useful to both scientists and engineers. It is also believed that the presented experimental database can be useful for verification of numerical models for nonlinear analysis of multi-drum columns with the considered joint types.

An investigation of the effect of joints between the blocks on the behaviour of freestanding multi-drum column under earthquake excitations by shake-table is in progress.

2. Tested column

2.1. Basic data

The basic data of tested column are presented in Fig. 1. Each column has a circular cross-section with diameter of 192 mm. It is formed of six equal blocks with 400 mm height, each of which has one steel hinge at the top and another steel hinge at the bottom. The height of the joints for each type of column varies from approximately 0 to 3 mm; thus, the height of the considered columns between the steel shoes varies from 2420 to 2436 mm. The total height of the column with steel shoes is approximately 2650-2666 mm, and the slenderness of the column is approximately 46 (column with middle slenderness). In the structural view, the column is analogous to the classical cantilever column with twice the lower height.

The number of joints, i.e., blocks in a multi-drum column, also has an effect on its ultimate bearing capacity. The effect of the number of blocks on the column behaviour under static forces is the subject of a separate experimental study, which is in progress.

Fig. 1. Basic data of a tested column

2.2.Blocks

It is always very difficult, especially today, to find stone blocks with the same properties in a quarry. In fact, often found are differences in the structure, strength, stratification and other characteristics of stone that is excavated even in the narrow location within the same quarry. To avoid the effect of possible anomalies in stone blocks and possible strength variation, concrete blocks were used instead. Namely, as the primary objective of this paper is to investigate the effect of joint type on the

behaviour of the multi-drum column, it is logical to exclude the effect of uneven quality of the material of the drums on the given conclusions. The uniformity of concrete quality is much more reliable than that of the stone. All concrete blocks were made of the same concrete. Limestone aggregate with maximal grain of 8 mm, white cement CEM I 52.5 R (320 kg/m^3) and stone powder (150 kg/m^3) were used. A superplasticizer was added to enhance the flowability of the concrete in the amount of 6.40 kg/m^3 , which resulted in water/binder ratio of 0.44. Testing of all columns was conducted in the period of 86-88 days after manufacturing of the blocks. In this period, the compression and tension strengths of the concrete specimens were tested according to HRN EN 12390-3, HRN EN 12390-6.

The compression strength of concrete was evaluated on a block with dimensions equal to those used in the columns. The average stress (σ) – strain (ϵ) relationship of three tested specimens is shown in Fig. 2. The average compression strength of the concrete blocks was 54.81 MPa, the average strain at the failure was -3.42 ‰ and the average initial modulus of elasticity was 33.42 GPa.

The tension strength of concrete was evaluated by splitting a concrete cylinder with a diameter of 192 mm and a length of 200 mm. The average tension strength of concrete was 6.09 MPa.

Fig. 2. Stress-strain relationship of concrete in compression

2.3. Joints

Four types of joints, i.e., four types of columns, were considered (see Fig. 3).

Fig. 3. Considered types of joints (columns)

Column C–DJ had a dry joint, where the blocks were in direct contact. As previously mentioned, even with great effort through processing of the blocks and the manufacturing of the columns, it is difficult to achieve equal adjustment of the contact surfaces between blocks and the verticality of the column. In joints where the blocks are in contact with the steel, a thin epoxy layer was previously drifted onto the contact steel surface on which stone powder is strewn to achieve approximately the same friction as that between blocks.

In column C-SPJ, the joints between the blocks and the joints between metal elements and blocks were filled with a layer of dry stone powder, having a maximal grain of 0.063 mm, in the thickness of approximately 2 mm.

In column C-PBJ, a lead sheet with the thickness of 2 mm was inserted in the joints between the blocks and the joints between the metal elements and the blocks, with uniaxial compressive strength of 10 MPa.

Column C-EPJ has all the joints connected using a two-component epoxy glue (*MAPEI* type “Epojet”) and a stone powder filler, in thickness of approximately 2 mm. In addition to the compression capacity, each joint has a tension capacity as well. The

tension and compression strengths of epoxy glue are significantly higher than those of the concrete.

2.4. Load

The columns were tested to the failure under the vertical axial compression force V and horizontal force H in the middle of the column height (Fig. 4). The columns are first tested under centric compression, with gradually increased force V to the failure of the column. Next, each type of column was tested to the eccentric compression, i.e., on the different relations of forces V and H . For each adopted constant value of force V (100 kN, 200 kN, 300 kN and 500 kN), which in the column produces centric compression (of approximately 3.46 MPa, 6.91 MPa, 10.37 MPa and 17.28 MPa, respectively), force H was gradually increased until the failure of the column. To avoid possible lateral damage of the column, force H was applied through a metal sheet in the joint. The application rates of forces V and H were approximately 5 kN/s and 0.5 kN/s, respectively.

Fig. 4. The scheme of columns' load and assumed V-H relation

Precision of vertical force V applying, which was applied for all the tests equally, was tested on a monolithic 2400 mm high column, with diameter and material properties as in the tested multi-drum columns. The top and bottom of this specimen were carefully horizontally cut and sanded. The specimen was multiple loaded by force V , with stresses up to approximately 40% of the concrete strength, with horizontal specimen rotation of 0° , 90° and 180° . Vertical strains were measured in the middle of the specimen's height, in four points at column circumference (in two orthogonal directions). The largest difference between measured strains on the opposite sides of the specimen was up to 7%.

By assuming the elastic strains (stresses) and their linear changes along the cross section (Fig. 5), for the ratio of minimum strain $\epsilon_{\min}$ and maximum strain $\epsilon_{\max}$ $\psi = \epsilon_{\min} / \epsilon_{\max}$ is valid:

$$e = \frac{W}{A} \cdot \frac{1 - \psi}{1 + \psi} \quad (1)$$

where e is the eccentricity of the force V , and W and A are the resistance moment and the area of the column cross-section, respectively. If the expression (1) is applied to the measured results of the tested monolithic column, where $\psi = 0.93$ and $W/A = 24 \text{ mm}$, it follows that $e = 0.87 \text{ mm}$. Such accuracy of force V applying has been adopted as acceptable since the core of the circular cross-section is $k = 192/8 = 24 \text{ mm}$. Because of the non flat surface of blocks at the top and bottom of the tested multi-drum columns, the force V was probably applied with greater eccentricities in some tests.

Fig. 5. Relation between column cross section edge strains $\varepsilon_{\min}, \varepsilon_{\max}$ and eccentricity of the force V

2.5. Measured values

Measured values are shown in Fig. 6. The applied forces (V, H), horizontal displacement in the middle of the column (u) and vertical displacement at the bottom of the column (v) were measured for all tested specimens. In addition, vertical strains ($\varepsilon_1^v, \varepsilon_2^v$) and horizontal circumferential strains (ε_1^p to ε_4^p) of the block near the middle of the column height were measured for centric loaded columns.

Fig. 6. Measured values

2.6. Instrumentation

The biaxial column test was conducted using a steel frame and calibrated hydraulic presses. The accuracy of the measured force is within 0.1%. Displacements were measured using displacements gauges (type LWA linear position transducer, Novotechnik) and strains with strain gauges of 100 mm length (type 100/120 LY41, HBM). The horizontal lateral strain of the block is equal to the measured horizontal circumferential strain.

3. Test results

Only some results of the performed tests are briefly discussed below. The bearing capacity of the tested columns in relation to the vertical force V and the horizontal force H is shown in Fig. 7.

Fig. 7. Bearing capacity of the tested columns

3.1. Column under vertical force only

For the case of columns under centric compression, column C–EPJ with epoxy joints showed the greatest bearing capacity (1421.6 kN), whereas column C–PBJ with lead sheet joints showed the lowest bearing capacity (724.8 kN). Column C–DJ with dry joints had the capacity of 967.5 kN, and the column with stone powder joints C–SPJ had the capacity of 835.6 kN. The ratio of the limit compression forces V for the tested columns C–EPJ : C–DJ : C–SPJ : C–PBJ was 1 : 0.68 : 0.59 : 0.51. It also represents the ratio of idealized computational limit centric compression stresses in the columns, which for columns C–EPJ, C–DJ, C–SPJ and C–PBJ were 49.1 MPa, 33.4 MPa, 28.8 MPa and 25.0 MPa, respectively. Obviously, there is a great difference in the ultimate bearing capacity of the tested types of columns, which will discuss below.

Column C–EPJ had the highest bearing capacity because of the epoxide layer in the joint, which ensured full overlap of blocks, i.e., equal transmission of compression stresses across the entire surface of the column's cross-section. A layer of epoxy in the joint forms a stiff tensile membrane that reduces the horizontal deformability of block along the joint. Fig. 8 shows the increased lateral deformation of the tested columns under centric compression for the equal longitudinal force $V = 500$ kN based on the measured circumferential strains (lateral strains). Reduction of the lateral deformability of a block causes lateral compression in it and increases the compressive strength of the material. The effect of epoxy joints is analogous to the effect of stirrups in a concrete column, which increases the compressive strength of concrete. It is obvious that even greater compressive strength of the column C–EPJ could be achieved with lower height of a block, i.e., with smaller vertical interval of epoxy joints. Therefore, it can be expected that a column made from a single monolithic block showed lower bearing capacity than a column with several stone blocks and epoxide joints. Three identical

centric pressure tests of a column with a monolithic block were performed to verify the previous statement. The mean value of the ultimate compressive force for that column was 1302.5 kN, which is approximately 8.4% less than the ultimate force of a column C-EPJ.

Fig. 8. Idealized deformation of tested columns under centric compression ($V = 500$ kN)

The lowest bearing capacity of column C-PBJ can be explained by the fact that the soft lead sheet produced a negative effect compared to the dry joint because it reduced the friction between the contact surfaces and increased horizontal lateral deformability of the blocks along the joint (see Fig. 8).

Column C-SPJ with joints filled with stone powder had a slightly greater bearing capacity than the column C-PBJ with a lead sheet in each of the joints. This difference is explained by the fact that the stone powder ensured more uniform transmission of vertical compression stress across the joint, and enabled greater friction and lower lateral deformability of a block. It can be expected that the use of mortar

instead of stone powder in the joints would increase ultimate bearing capacity of the column.

Column with dry joint C-DJ had greater bearing capacity than columns C-PBJ and C-SPJ. The probable reason for this was the direct contact of blocks in column C-DJ that ensured greater friction in the joints and lower horizontal lateral deformability of the blocks, where the effect of imperfections on contact via transmission of compressive stress was probably similar to the effect in columns C-DJ, C-PBJ and C-SPJ (for equal processing of block contact surfaces). Ultimate bearing capacity of the columns C-DJ, C-SPJ and C-PBJ, as well as their relationship highly depends on the quality of processing of the contact surfaces of blocks.

Vertical displacement of the bottom of columns for the equal vertical force $V = 500$ kN is shown in Fig. 9. The largest displacement was found for column C-PBJ, and the lowest was for column C-EPJ. The great difference in the results is a consequence of the large differences in the compressibility of the joints. While an epoxy joint is almost incompressible, joints with a lead sheet and stone powder are very compressible.

The vertical displacement of the bottom of columns at failure is shown in Fig. 10. The values of displacements are difficult to explain because they are the result of the compacting of the joints and axial deformation of the blocks.

Fig. 9. Vertical displacement of the bottom of columns for $V = 500$ kN

Fig. 10. Vertical displacement of the bottom of columns at failure

It should be emphasized that under the centric force V , some of three identical specimens for each column were deformed in one direction, and some of them in the opposite direction. This is confirmed in Fig. 11, where the vertical compressive strains $\varepsilon_1^v, \varepsilon_2^v$ of three identical specimens of column C-SPJ are shown. It is evident that ε_1^v is smaller than ε_2^v for the first and third specimen, which shows that these specimens were deformed to the left side. Strain ε_1^v is higher than ε_2^v for the second specimen, which shows that this specimen is deformed to the right side. Similar strain distribution was also for the remaining tested columns, but these results are not shown due to limited article length.

Fig. 12 shows the mean values of maximum and minimum vertical strains for all columns, regardless of the direction of columns' deformation, that are marked with $\varepsilon_{\max}^v, \varepsilon_{\min}^v$. A great difference in vertical strains was noticeable for columns C-DJ, C-SPJ and C-PBJ, while for column C-EPJ it was significantly lower. This difference is a consequence of the actual eccentricity of force V in the area of glued vertical strain gauges. By analysing the Fig. 12, it can be determined that the lowest relation

$\psi = \varepsilon_{\min}^v / \varepsilon_{\max}^v$ at failure was for column C-PBJ and was approximately $\psi = 0.6$. According to the expression (1), the eccentricity of force V in this cross section was $e = 0.125 \cdot 192(1 - 0.6) / (1 + 0.6) = 6 \text{ mm}$. Thus, the actual eccentricity of force V in that point is still deep inside the cross section core $k = 24 \text{ mm}$. Considering the expected negative common effects of the joints (higher extent) and initial eccentricity of force V at the place of application (less extent), it can be concluded that it is not too large, i.e. that the precision of the performed tests is satisfactory. For column C-EPJ, the actual eccentricity of force V in the middle of the column's height is significantly smaller due to very small effect of the epoxy joint. There is probably a greater effect of eccentricity at the point of force V acting than the effect of joint eccentricity.

Fig. 11 Vertical compressive deformation $\varepsilon_1^v, \varepsilon_2^v$ of three identical specimens of axially loaded column C-SPJ

Fig. 12. The average vertical compressive strain $\varepsilon_{\max}^v$, $\varepsilon_{\min}^v$ of centrally loaded columns

The average vertical strain $\varepsilon_{l-2}^v = 0,5(\varepsilon_{\max}^v + \varepsilon_{\min}^v)$ in the considered columns at the force $V = 500$ kN is shown in Fig. 13. These strains are similar for all tested columns, which shows that the type of joint does not have a great effect on the average longitudinal strain of the observed block.

The average vertical strain ε_{l-2}^v at the failure of the considered columns is shown in Fig. 14. The difference in their values is a result of the different longitudinal forces at the failure of the columns, as well as their lateral deformability and imperfections in the joints.

Fig. 13. The average vertical strain ε_{1-2}^v of centric loaded columns at $V = 500$ kN

Fig. 14. The average vertical strain ε_{1-2}^v of centric loaded columns at failure

Circumferential tension strains ε_1^p to ε_4^p for all three tested specimens of column C-SPJ are shown in Fig. 15. (for better transparency of the drawings, no backward strains are shown), which are analogous to vertical strains $\varepsilon_1^v, \varepsilon_2^v$ in Fig. 11. They can cause splitting of the column blocks in a vertical plane. Larger circumferential strains are on the side of larger vertical strains of the blocks, i.e. they also depend on the direction of lateral column deformation. Strains on the opposite sides of the column begin to differ more when the column approaches the failure. It may be effect of the occurrence of vertical cracks in the zone of glued circumferential gauge. The mean values of maximum and minimum circumferential tension strains for all tested columns are presented in Fig. 16, independent of the deformation direction, which are marked with $\varepsilon_{\max 1,2}^p, \varepsilon_{\min 1,2}^p$ (correspond to strains $\varepsilon_1^p, \varepsilon_2^p$) and $\varepsilon_{\max 3,4}^p, \varepsilon_{\min 3,4}^p$ (correspond to strains $\varepsilon_3^p, \varepsilon_4^p$). Great difference is noticeable in the circumferential strains for columns C-DJ, C-

SPJ and C-PBJ, while for column C-EPJ it was significantly lower. Explanations are analogous to those of the vertical compressive strains. Larger variations for higher forces V probably contain the effect of crack width on the measuring points.

Fig. 15. The circumferential tension strains ϵ_1^p to ϵ_4^p of three identical specimens of centrally loaded column C-SPJ

Column C-DJ

Column C-SPJ

Column C-PBJ

Column C-EPJ

Fig. 16. The average circumferential tension strains $\varepsilon^p_{\max 1,2}$, $\varepsilon^p_{\min 1,2}$, $\varepsilon^p_{\max 3,4}$, $\varepsilon^p_{\min 3,4}$ of centrically loaded columns

The average circumferential strains $\varepsilon^p_{1-2} = 0,5(\varepsilon^p_{\max 1,2} + \varepsilon^p_{\min 1,2})$ and $\varepsilon^p_{3-4} = 0,5(\varepsilon^p_{\max 3,4} + \varepsilon^p_{\min 3,4})$ of all columns at the force $V = 500$ kN are shown in Fig. 17. These strains are the lowest for column C-EPJ and the highest for column C-PBJ. It is obvious that the type of joint has an influence on the lateral deformation of column and thus indirectly affects its circumferential strain and ultimate bearing capacity.

Fig. 17. The average circumferential strains $\varepsilon_{1-2}^p, \varepsilon_{3-4}^p$ of columns at centric force $V = 500 \text{ kN}$

All tested columns had brittle explosive failure, without previous visual detectable horizontal deformations. A photograph of column C-SPJ at failure is shown in Fig. 18. The collapse of the columns dominantly occurred at their half height because the greatest influence of slenderness and horizontal displacements was there.

Fig. 18. Photograph of column C-SPJ at failure under centric compression

The collapse of the columns is followed by vertical splitting of some blocks, where the main plane of splitting was closer to the centre than to the edge of the column's cross-section. One of the tested columns after failure under centric compression is shown in Fig. 19.

Fig. 19. Photograph of one of the tested columns after the failure under centric compression

3.2. Columns under vertical and horizontal force

The bearing capacity of tested columns under horizontal force H is generally small and depends on the level of vertical force V (Fig. 7). In addition, it should be noted that the columns were firstly loaded by vertical force to the full amount, and then by horizontal force to their failure. Shape of bearing capacity diagrams of such column is analogous to those of a reinforced concrete column, wherein the bearing capacity of tested columns is significantly lower because it cannot transmit tension stresses at the level of joints (except for column C-EPJ) and due to limited tensile strength of a block. The effect of the joint type on limit bearing capacity of columns simultaneously exposed to forces V and H is analogous to the columns exposed to force V only. For the same vertical force V , the highest horizontal force at failure had a column C-EPJ, and the lowest column C-PBJ. For $V = 500$ kN, the ratio of the limit force H for the tested columns was C-EPJ : C-DJ : C-SPJ : C-PBJ = 1 : 0.92 : 0.85 : 0.70.

If the influence of lateral deformation of a column on the increase in the bending moments for the vertical force V is ignored and the blocks are assumed to be rigid, then

the criterion of theoretical maximal horizontal force ($H_{\max}$) can be defined in terms of overturning of the block in the joint by:

$$H_{\max} \leq 2 V d / h \quad (2)$$

where d is the diameter of the circular column, and h its height. If the geometric data of the tested column was included in equation (2), for $V = 500$ kN the maximal horizontal force would be $H_{\max} = 72.0$ kN. The measured limit horizontal forces of the tested columns are lower than $H_{\max}$ because of the increase of the bending moments from the horizontal displacements of the column and the limited compression strength of the block (before overturning the block, the failure of the material on the compression edge of its cross section occurs).

The horizontal displacements of the column depend on its bending stiffness, which depends on the type of joint between blocks (type of column). The highest bending stiffness and lowest horizontal displacements were found for column C-EPJ with the epoxy joint; thus, the highest horizontal force at failure is realized in it. The lowest stiffness and the highest displacements were found for column C-PBJ with the lead sheet in the joint; thus, the lowest horizontal force at failure is realized in it. The relationship between the horizontal force H and the horizontal displacement (u) of its action in the middle of the tested columns is shown in Fig. 20. By increasing the vertical force, bending stiffness of all tested columns is increased, i.e., for the same horizontal force, their horizontal displacements are reduced. In addition, by increasing the vertical force, the limit horizontal force is increased. The horizontal displacements should be taken with care after reaching the maximal horizontal force, and they are not important from the engineering point of view.

Fig. 20. Horizontal force (H)–horizontal displacement (u) relation for the tested columns

The relationship between the horizontal force (H) and the horizontal displacement (u) in the middle of the column height for an applied level of vertical force is shown in Fig. 21, which is analogous to Fig. 20. For all levels of the vertical force, the highest stiffness and bearing capacity are both found for column C–EPJ. Columns C–DJ, C–SPJ and C–PBJ had lower stiffness and bearing capacity values in that order.

Fig. 21. Horizontal force (H)–horizontal displacement (u) relationship for different levels of force V

The columns that are exposed to the axial compression and bending at the same time had significantly lower explosive failure than the columns exposed to the axial compression force only, especially at lower levels of the compression force. The columns had noticeable horizontal displacements before failure. With these loads, especially at lower vertical force, the plane of splitting at the failure of the column is approaching the compression edge of the column cross-section. Some columns after failure are shown in Fig. 22.

Fig. 22. The appearance of some columns exposed to the axial compression and bending after the failure

4. Conclusions

Based on the experimental test results of multi-drum columns under vertical and horizontal static forces, the following conclusions can be drawn.

The type of joints and processing of blocks has a great effect on the load bearing capacity of a multi-drum column. Even with much effort in the processing of the blocks and the production of the multi-drum columns, it is difficult to achieve equal adjustment of the contact surfaces between blocks and the verticality of a column.

Under axial compression, the ratios between the ultimate load bearing capacities of the tested columns with different types of joints were C-EPJ : C-DJ : C-SPJ : C-PBJ = 1 : 0.68 : 0.59 : 0.51.

Column C-EPJ had the highest bearing capacity because of the epoxide layer in the joint, which ensured equal transmission of compression through the blocks, allowed transmission of tension stresses through joints and significantly reduced the horizontal deformability of block at the joint level (the effect of rigid tension membrane - analogous to the effect of stirrups in concrete column). This column had higher bearing capacity than the column made from a single monolithic block.

Column C-PBJ with soft lead sheet in the joints had the lowest bearing capacity because the joint did not ensure equal transmission of compression through the blocks, and increased the lateral deformability of the column in the area of joints relative to the dry joint.

Column C-SPJ, with the joint filled with stone powder, had slightly greater bearing capacity than column C-PBJ because the stone powder ensured almost equal transmission of compression through joint and allowed slightly less lateral deformability of column at the level of joint. The load capacity of column would probably be higher if a mortar was used instead of the stone powder.

Column C-DJ, with a dry joint, had a higher bearing capacity than columns C-PBJ and C-SPJ, with vertical and horizontal deformable joints, respectively. Direct surface contacts of the blocks in the joints ensure higher friction and lower lateral deformability of the columns. The load capacity of such column probably would be significantly lower at greater roughness of contact surfaces between the blocks; in that case, the bearing capacity could be less than that for C-PBJ and C-SPJ.

The failure of the tested axial compressed columns was explosive and followed by vertical splitting of the blocks, with the splitting plane located closer to the centre than to the edge of the columns cross-section.

The bearing capacity of tested columns under lateral horizontal force H depends on the level of axial compressive force V and is generally small. For the adopted constant values of force V up to 500 kN, the limit force H generally was increased with the increase of force V . V - H diagrams of the bearing capacity of the tested multi-drum columns are analogous to those of a classical monolithic concrete columns, wherein, the bearing capacity of tested columns is significantly lower because they cannot carry tension in joints (except the column C-EPJ). The effect of the joint type on the bearing capacity of the columns simultaneously loaded by forces V and H is analogous to the columns under force V only. For $V = 500$, the ratio of limit force H for tested columns was C-EPJ : C-DJ : C-SPJ : C-PBJ = 1 : 0.92 : 0.85 : 0.70. The horizontal displacements of the columns depend on their bending stiffness, which depends on the joint type. The highest stiffness and smallest displacements were found for column C-EPJ, and smallest stiffness and highest displacements were found for column C-PBJ. The columns that are simultaneously exposed to axial compression and bending had a far lower explosive failure than the columns under axial compression force only. The plane of blocks splitting at the columns failure was near the compression edge of their cross-section (especially at lower force V).

In progress is the study of effect of joints on the behaviour of freestanding multi-drum columns under earthquake using a shake-table.

5. Literature

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EFFECT OF THE JOINT TYPE ON THE SEISMIC BEHAVIOUR OF A FREE-STANDING MULTI-DRUM COLUMN

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Abstract:

The results of a shake-table study of the effects of the joint type on the seismic behaviour of free-standing small-scale multi-drum column are presented. Columns with three types of joints, loaded only with their own weight and with additional weight on the top, were exposed to three different accelerograms. The acceleration amplitude was successively increased to the columns failure. It was concluded that both the joint type and the type of accelerogram have great effects on the behaviour and bearing capacity of a multi-drum column. It is believed that the experimental database can be useful in validating numerical models for the seismic analysis of multi-drum columns.

Keywords: Multi-drum column, effect of joint type, shake table

1 **1 Introduction**

2 Multi-drum stone columns are often structural elements of historic stone structures. The
3 application of several blocks at the column height is especially relevant for tall columns. Such
4 a concept simplifies, accelerates and reduces the cost of the construction of columns. Thereby,
5 the logical question of comparing the behaviour and the bearing capacity of monolithic and
6 multi-drum columns exposed to the equal loads is posed. This also raises the question of how
7 each type of joint in a multi-drum column (dry joint, joint from stone powder, joint from
8 different mortars, epoxide joints, etc.) affects its behaviour and bearing capacity. There are
9 still no satisfactory answers to these questions, and there is no understanding of the influence
10 of a number of other parameters that affect the seismic behaviour of multi-drum columns.
11 Therefore, these and similar studies that contribute new understandings of the real behaviour
12 of multi-drum columns during an earthquake are welcomed.

13 Some contemporary shake-table and numerical studies related to multi-drum columns are
14 only briefly described below, because they have not considered the issue of the presented
15 study.

16 Using a shake-table, Mouzakis et al. [1] experimentally investigated the earthquake
17 response of a marble model of a classical column as 1: 3 scale replica of a column of the
18 Parthenon on the Acropolis of Athens, made from the same material as the original. They
19 were found that the column might undergo large deformations during the shaking, which are
20 not necessarily reflected by the residual displacements at the end of it, response is very
21 sensitive even to small changes of the geometry or the input motion parameters. Krstevska et
22 al. [2] experimentally investigated the prototype and the model of the Antonina column in
23 Rome to provide more clear column behaviour and resistance. The general conclusion from
24 the shake-table testing is that low frequency content earthquakes may cause serious damage to

1 the column. Drosos and Anastasopoulos [3] experimentally studied the seismic performance
2 of the reduced scale free-standing columns as the form of the remnants of Greek temples.
3 They stated that the tested multi-drum columns were proven to be very earthquake-resistant,
4 even when subjected to the strongest motions ever recorded in Greece, their permanent
5 deformation was minimal. Also, Drosos and Anastasopoulos [4] provide shake-table testing of
6 free-standing multi-drum columns composed of stones, resting on top of each other without
7 any connection, as the survived form of the ancient Greek temple. They pointed how
8 complexity increases exponentially with the number of drums, being directly associated with
9 the number of drum-to-drum interfaces and increased probability of interface imperfections.
10 Papalou [5] examines the dynamic response of short free-standing classical columns by
11 investigating effect of their geometrical features on their dynamic behaviour. She notes how
12 numerical models based on the finite element method can provide reasonable estimates of the
13 dynamic response of classical columns but cannot predict all the attributes of their actual
14 dynamic behavior. The protection of a classical column in areas of seismic activity using
15 passive control techniques were studied by Papalou and Strepelias [6]. They experimentally
16 investigated the effectiveness of particle dampers in reducing the vibrations of multi-drum
17 ancient columns. They found that the particle damper when properly designed can reduce the
18 motion of a multi-drum column significantly without altering the external appearance of the
19 column. Manos and Demosthenous [7] studied dynamic and earthquake behaviour of the
20 model of ancient columns and colonnades subjected to horizontal base motions. They stated
21 how the seismic response mechanisms that develop on this solid block structural system
22 during strong ground motions can include sliding and rocking. Alecci et al. [8] performed
23 experimental investigations of brick work columns confined with CFRP composite.

24 In numerical study of the seismic response of a multi-drum stone column, Konstantinidis and
25 Makris [9] numerically investigated seismic response of multi-drum classical columns,

1 especially the level of ground shaking that classical multi-drum columns can survive, and the
2 possible advantages or disadvantages of retrofitting multi-drum columns with metallic shear
3 links that replace the wooden poles that were installed in ancient times. They concluded that:
4 a) typical multi-drum classical columns can survive the ground shaking from strong ground
5 motions recorded near the causative faults of earthquakes with magnitudes $M_w = 6.0 - 7.4$; b)
6 in most cases multi-drum classical columns free to dislocate at the drum interfaces exhibit
7 more controlled seismic response than the monolithic columns with same size and
8 slenderness; c) the shear strength of the wooden poles has a marginal effect on the sliding
9 response of the drums; and d) stiff metallic shear links in-between column drums may have an
10 undesirable role on the seismic stability of classical columns and should be avoided.
11 Papaloizou and Komodromos [10] studied the behaviour of multi-drum columns and
12 colonnades during earthquakes using a discrete element method. The numerical simulations
13 showed that earthquakes with relatively low predominant frequencies seemed to endanger
14 both standalone columns and colonnades more than earthquakes with higher predominant
15 frequencies. Sarhosis et al. [11] utilized a discrete element method to study the static and
16 dynamic behaviour of a typical section of the two-storey colonnade of the Forum in Pompeii.
17 It was shown that the construction method used in ancient times, such as multi-block
18 segmented trabeations and a solid block beam, had a significant impact on the mechanical
19 response of the structure. Komodromos et al. [12] performed a simulation of the response of
20 ancient multi-drum columns under harmonic and earthquake excitations by a discrete element
21 method. They performed a large number of parametric studies to investigate the effect of
22 certain parameters on the response of the columns. The simulations indicate that multi-drum
23 columns are significantly more vulnerable to collapse at low-frequency excitations, which do
24 not commonly occur in the Eastern Mediterranean region where most of these monuments can
25 be found. Papantonopoulos et al. [13] using the distinct element method predict the

1 earthquake response of a multi-drum marble model of a classical column. They compared
2 result with experimental data for an ‘identical’ specimen under the same excitation. They
3 show that the distinct element method can capture quite well the main features of the
4 response, in spite of the sensitivity of the response to even small perturbations of the
5 characteristics of the structure or the excitation. Psycharis et al. [14] present numerical study
6 of the seismic behaviour of a proposed restoration of the Parthenon Pronaos. The column-
7 architrave classical structure was represented by a discrete element model, with the
8 assumption of rigid blocks and frictional joints. A proposal of reinforcement with titanium
9 connections was analysed, and it was found that architrave connections generally improve the
10 response by decreasing the permanent displacements, while titanium dowels between the
11 column drums do not improve significantly the behaviour and in some cases they may be
12 unfavourable to the safety of the structure. Kawadiaz et al. [15] in seismic response
13 parametric study of ancient rocking columns show that the velocity based as well as the
14 frequency content ground motion intensity measures exhibit a strong correlation with the
15 seismic behaviour of free-standing rocking columns. Ascione et al. [16] numerically analysed
16 the load carrying capacity of 2D FRP/strengthened masonry structures with an adaptive
17 discontinuous finite element model for limit analysis of masonry structures strengthened by
18 fiber composites, which is able to predict the effects of fracture damage and delamination on
19 the load carrying capacity of the reinforced structures. Lignola et al. [17] analysed influence
20 of short segments in the trabeation with opposing inclined edges on the seismic vulnerability
21 of the marble blocks colonnade in the archaeological site of Pompeii. They show the seismic
22 vulnerability of a colonnade, in order to understand how this World cultural heritage site can
23 be preserved. Fossetti et al. [18] analyzed effect of steel collars on the seismic behaviour of
24 axially-cracked historical stone columns subjected to rocking motion due to pulse type ground
25 shaking. They show their effectiveness in reducing the overturning risk. D’Ambrisi et al. [19]

1 experimentally and analytically investigated bond between Carbon-FRCM materials and
2 masonry. Brodsky et al. [20] experimentally investigate the behaviour of the effect of the
3 interfaces between the masonry wall units – joints, on the global behaviour and local infill –
4 frame interaction. The results show that the joint properties have a significant effect on the
5 ultimate load, the initial stiffness and the energy dissipation. Cascardi et al. [21]
6 experimentally investigate a new generation of strengthening of masonry columns. They
7 analysed the experimental results on FRCM–confined masonry columns subjected to centred
8 compression test. Kouris and Triantafillou [22] investigate state-of-the-art on strengthening of
9 masonry structures which are most vulnerable to earthquakes. They give an insight into
10 relevant experimental procedures and results of retrofitted masonry elements to illustrate the
11 effectiveness of textile reinforced mortar for enhancing strength and deformation capacity.
12 Witzany et al. [23] experimentally investigate strengthening and stabilisation of masonry
13 column. They concluded that the confinement of a masonry column with fabrics of carbon
14 fibres with a high modulus of elasticity and high tensile strength relieves the transverse tensile
15 stresses in the masonry and increases the ultimate deformation and the ultimate load. Sarhosis
16 et al. [24] performed three dimensional modelling of ancient colonnade structural systems
17 subjected to harmonic and seismic loading. A typical section of the two-storey colonnade of
18 the Forum in Pompeii has been modelled and studied parametrically, in order to identify the
19 main factors affecting the stability and to improve understanding of the earthquake behaviour
20 of such structures. They conclude that the high-frequency motion requires large base
21 acceleration amplitude to lead to the collapse of the colonnade in a shear-slip mode between
22 the drums, and low-frequency harmonic excitations are more prominent to cause structural
23 collapse of the two-storey colonnade than the high-frequency ones with predominant rocking
24 failure mode. Asteris et al. [25] gave an extensive overview through numerical modelling of
25 historic masonry structures. They pointed how the study of historic masonry structures is a

challenging task due to the difficulties encountered in the description of the complex geometry, morphology, material heterogeneity, material properties characterization, material variation due to weathering and deterioration effects and complex loading conditions. They conclude how a combined experimental-numerical basis is the key to validate, extend and improve the existing methods for the analysis of the mechanical behavior of historic masonry structures. Sarhosis et al. [26] conducted a numerical study (using Distinct Element Method) to investigate the seismic vulnerability of the two storey colonnade of the Forum in Pompeii. They represent colonnade as an assemblage of distinct blocks connected together by zero thickness interfaces which could open and/or close depending on the magnitude and direction of stresses applied to them. Also, they performed a sensitivity study to investigate the effect of frictional resistance of the joints on the structural response of the colonnade to simulate potential joint degradation effects and/or possible water lubrication at the joint. Asteris et al. [27] performed the numerical simulation of displacement response history of a real-life system to known seismic excitation, using the finite element method. The result obtained demonstrate that the earthquake response of the systems is significantly affected by the behaviour at the interfaces between contacting materials. Mohebkah et al. [28] present the development of a three dimensional numerical model based on the discrete element method of analysis which was used to investigate the seismic behaviour of the Cube of Zoroaster tower. The behaviour of the tower to different ground shaking motions is discussed and the possible failure modes for each case are explored.

As can be seen from the aforementioned, to date the researchers have almost not or only sporadically analyzed the effect of joint type on the behaviour and bearing capacity of the multi-drum columns at the earthquake.

This paper presents the results of the shake-table study of the effect of the joint type on the behaviour of a free-standing, small-scale multi-drum column exposed to three different

1 accelerograms. Three types of joints are varied: a dry joint, a joint filled with stone powder,
2 and an epoxy joint. A column with self weight only was analysed, and the same column with
3 a mass on the top was also analysed. The excitations were applied with a successive increase
4 of the maximum acceleration until the failure of the column. Characteristic displacements and
5 accelerations of some blocks are measured, and the motions of the columns during the
6 dynamic excitations are recorded. It was concluded that the joint type and accelerogram type
7 have great influences on the behaviour and the bearing capacity of a multi-drum column
8 during an earthquake. It was also explained why some types of joints and some excitations
9 resulted with the measured values of tested multi-drum columns. It is believed that the
10 presented experimental results can be useful in the valorisation of numerical models for
11 seismic analysis of multi-drum columns. To the best of the authors' knowledge, such research
12 has not yet been conducted.

13 Namely, main novelties and highlights of the presented study are:

- 14 - It was conducted a systematic experimental study of the effect of several joint types on
15 seismic response of multi-drum column.
- 16 - Testing of the adopted samples is provided by several different types of earthquakes, with
17 an increase of their acceleration from low values to the column failure.
- 18 - Free-standing columns with self weight only and with self weight and mass on the columns
19 top were considered.
- 20 - It was concluded that the type of joints in multi-drum column has a great effect on its
21 stiffness (free- oscillation period) and bearing capacity.
- 22 - With a softer joint (dry, stone powder etc.) the stiffness of the multi-drum column
23 decreases, and with a stiff joint (strong mortar, epoxy) and column with one monolithic
24 block the stiffness of the multi-drum column increases.

- Multi-drum column with soft joints has a lower earthquake resistance than a column with rigid joints or column from a monolithic block, but they pull to itself lower earthquake forces.
- Earthquakes with long duration and larger predominant period, which entered higher energy in the structure, are usually the most unfavourable for multi-drum columns with soft joints.
- Earthquakes with short impact action, which entered lower energy in the structure, are usually the most unfavourable for multi-drum columns with stiff joints and columns from one monolithic block.
- The conclusions of the study can be useful for engineers in practice.

2 Tested samples

The tested free-standing small-scale multi-drum columns are presented in Fig. 1. The columns are formed from six identical blocks, each with a circular cross section of 192 mm diameter and a height of 200 mm (approximate slenderness 35). The thickness of the joints in the column are approximately 1-3 mm. Thus, all the columns are practically equal (approximately 1210 mm). Columns C-PJ, C-DJ, and C-EJ are loaded with self weight only, and equal columns Cm-PJ, Cm-DJ, and Cm-EJ have an additional mass at the top. The mass of 500 kg is in the form of a concrete block measuring 0.8 m × 0.8 m × 0.33 m; the mass is centrically positioned in the axis of the relevant column.

The subject of this study is the effect of the joint type on the behaviour of a multi-drum column during an earthquake. In this context, three different joints were adopted: (i) a joint from dry stone powder – columns C-PJ and Cm-PJ, (ii) a dry joint – columns C-DJ and Cm-DJ and (iii) an epoxy joint – columns C-EJ and Cm-EJ.

Fig. 1 Tested columns

The dry joint (DJ) was adopted because the largest number of multi-drum columns is probably made with such a joint, i.e., by direct block-to-block placement. Depending on the precision of the treatment of the contact stone surfaces, the transmission of horizontal stresses in the joint can be completely uniform but also with expressed concentrations and discontinuities. Such a joint cannot transmit tensile stresses and allows transversal shear discontinuity of the adjacent blocks.

The joint filled with stone powder (PJ) is also used in the history of multi-drum columns. Such a joint is simple to produce and probably allows a much more uniform transmission of normal stresses over the joint as well as significantly more uniform transmission of shear stresses in the plane of the joint compared to the dry joint. The dry stone powder joint also cannot transmit tensile stresses perpendicular to its plane. The dry stone powder joint also allows transverse and shear discontinuity of adjacent blocks in the column.

To analyse the stiff joint that can transmit tensile stresses perpendicular to its plane, an epoxy joint was chosen (EJ). The epoxy joint is made of a mixture of a two component epoxy adhesive (*MAPEI* type “Epojet”) and fine stone powder, in a proportion of 1:3. Such a joint has not been used in historic stone buildings, but provides approximately the same behaviour of a multi-drum column as that of a monolithic block.

Investigations of the effect of other types of joints (from different mortars, etc.) were not considered because of the limited scope of the study. However, the results of this study can contribute to further understanding of the effect of selected joint types on the seismic behaviour of the multi-drum columns.

The columns are supported on a shake-table indirectly over a rigid concrete plate of 50 mm in thickness, that is fixed to the shake-table. Except between the blocks, equal joints were adopted between the lowest block and the concrete plate and between the block and the mass at the top of the column. To make the effect of the joints more pronounced in relation to the monolithic columns, columns formed of six blocks were adopted. In addition to other numerous parameters, the number of blocks in the column (number of joints) undoubtedly also affects both the seismic behaviour and the bearing capacity of the multi-drum column. The effects of this parameter were explored by Buzov et al. [29].

Although the focus of this study was stone multi-drum columns, concrete blocks were used instead of stone blocks for the following reasons [29, 30]: (i) avoiding the use of a specific type of stone since the historical multi-drum stone columns are made from different types of stones, with its different parameters (strength, deformations, modulus of elasticity, angle of friction between the blocks etc.); (ii) to achieve a significantly greater uniformity of block quality; (iii) to shorten the duration and cost of the study. Because concrete is very similar to stone (artificial stone), the failure of the columns in all tests was only by loss of the columns stability and the coefficient of friction between the stone blocks is close to that between the concrete blocks with aggregates of that stone, it is considered that such approach does not have a significant impact to the given conclusions.

Table 1 presents an experimentally determined fundamental period T_1 of the considered columns (mean value on three equal samples). A significant impact of joint type on T_1 is

found. It is apparent that the epoxy joint results in a more rigid column, and the stone powder joint results in the softest column. A great impact was found regarding the mass at the top of the column on the increase of T_1 . The relationship between T_1 for columns C-PJ, C-DJ, C-EJ is 2.81:3.22:1, and that for columns Cm-PJ, Cm-DJ, Cm-EJ is 2.83:3.20:1. It should be noted that the column stiffness are significantly reduces during its oscillations due to the appearance of nonlinearity in it (rocking, slipping etc.), i.e. that the given values for T_1 are valid only for elastic column behaviour.

Table 1 Fundamental period T_1 of the tested columns

	C-PJ	C-DJ	C-EJ	Cm-PJ	Cm-DJ	Cm-EJ
T_1 [s]	0.152	0.174	0.054	0.507	0.573	0.179

3 Applied base accelerograms

The considered columns in Fig. 1 are subjected to the horizontal base accelerograms presented in Fig. 2. The artificial accelerogram (AA) was performed based on the elastic response spectrum [31] for type 1 and soil type A (Fig. 2a). The horizontal component N-S of the Petrovac earthquake (AP) is presented in Fig. 2b [32], and the horizontal component N-S of Stone earthquake (AS) is presented in Fig. 2c [32]. Fig. 3 presents the spectral values of the adopted accelerograms from Fig. 2. If the basic periods T_1 from Table 1 are connected with the spectral value diagrams from Fig. 3, then it can be expected that artificial accelerogram (AA) and Petrovac accelerogram (AP) are more unfavourable compared to the Ston accelerogram (AS). Because the diagrams from Fig. 3 are constructed for an elastic single-degree console column, and the considered columns are likely to be significantly softened during the action of the excitations (the appearance of nonlinearity in the joints), it is difficult to specify in advance the most unfavourable excitation for such a complex non-linear

problem. The intention of this study was to choose different types of accelerograms in terms of their duration and the predominant period at maximum acceleration.

A small number of excitations was adopted due to the shortening of the study, as well as the fact that for each further different accelerogram the response of the tested multi-drum column will be different and the global conclusions of the study regarding to the influence of the earthquake type on the effects of the considered column parameters will probably be the same. Namely, it was not the intention and it is not crucial to find the most unfavorable accelerograms for the observed small-scale multi-drum column, but to investigate the relative effects of the analyzed column parameters for few characteristic types of earthquakes by duration and acceleration changes in time. In analyse of real column at some location by using scaled model, it is necessary to define several possible types of earthquakes for that location (at least 6 [31]) with a adequate scaling of accelerogram time domain for adopted scaling level.

The adopted accelerograms (Fig. 2) were applied on the considered columns (Fig. 1) with the successive increase of the maximum acceleration (a_{gmax}) until the collapse of the column. The increment of acceleration increase Δa_{gmax} was adopted as 0.2 ms^{-2} for AA and AP and 0.5 ms^{-2} for AS. Although it is logical to expect that each different applied accelerogram resulted in a different response for each of the considered columns, it is considered that only three selected accelerograms will be relevant for the credibility of the given conclusions.

Fig. 2 Applied accelerograms

Fig. 3 Spectral accelerograms, velocities and displacements of the applied accelerograms

4 Measurement and equipment

The measured quantities are presented in Fig. 4. Horizontal displacement u_i are measured 5 mm below the top and 5 mm above the bottom of each block, and acceleration a_i is measured in the middle of the height of each block. The position of the column during the dynamic excitation was captured by a precise camera.

Testing was performed using a shake-table at the University of Split. Data collection from the sensors was performed using a Quantum-x mx 840 A system (HBM). For displacement measurements, displacement sensors PB-25-S10-NOS-10C (Uni Measure) were used. The acceleration was measured by a piezo-electric high-frequency accelerometer type 4610 (MS).

Fig. 4 Measured displacements (u) and accelerations (a)

5 Experimental results

As the experimental results are obtained on a small-scale model, it should be borne in mind that they are different from the results on real-size model (prototype). Namely, it is well known that a small-scale model can not adequately describe some important parameters of the real-size model. If n is a scaling factor (reduction coefficient) of the prototype, the ratio of some important parameters of the scaled model and prototype of the same material are shown in Table 2. Related to the subject study, it should be emphasized that in the small-scale model there are n -times smaller vertical stresses from gravitational loads, as well as from vertical and shear stresses from inertial (earthquake) forces. This particularly affects the effects of “rocking” and slipping of the blocks at the oscillation of the multi-drum column. As the frequency of the scaled model is larger than of the prototype, the effects of the same excitation on these models are different. Therefore, the time domain of the excitation for the scaled model needs to be reduced by $n^{1/2}$ (Table 2). Since a small-scale model in conducted study was treated as a prototype, because the relative effects of several parameters were studied on it, the time domain of the adopted excitations was not reduced. It is also well known that the strength and deformation of the same material also depend on the sample dimensions (size effect). Therefore, the strengths and deformations of the same stone (concrete) in the small-scale model and prototype will not be the same.

Thus, the obtained test results on the considered small-scale model can not be applied directly (without reserve) to the n -times larger prototype. They are exactly valid only for the tested model. However, as the relative effects of the considered parameters was investigated on the small-scale model, it is considered that they can be reliably apply to real multi-drum columns in practice. Namely, it is considered that the difference in considered relative effects on the small-scale model and on the n -times larger real construction is practically negligible.

Table 2 Relation between some parameters among small-scale model and prototype (n = reduction coefficient)

Dimension	n^{-1}
Weight / mass	n^{-3}
Inertial / shear force	n^{-3}
Bending moment of inertial force	n^{-3}
Normal stress / strain of gravitational load	n^{-1}
Normal stress / strain of inertial force	n^{-1}
Shear stress of inertial force	n^{-1}
Oscillation frequency	$n^{1/2}$
Oscillation period	$n^{-1/2}$

5.1 Columns C-PJ, C-DJ, and C-EJ

To check the reliability of tests, the accuracy of their repeatability was verified first. Namely, for AA, three tests were performed for columns C-PJ and C-DJ with $a_{gmax} = 1.0 \text{ ms}^{-2}$, and two tests were performed for column C-EJ with $a_{gmax} = 1.5 \text{ ms}^{-2}$. The maximum difference between the measured values for columns C-PJ, C-DJ, and C-EJ were 4.4 %, 6.2 %, and 1.9 %, respectively. Considering the possible influences of various parameters on the precision of the tests (column verticality, inhomogeneity in the joints, precision of acceleration application by shake-table, imperfection of measuring equipment, etc.), it was concluded that these deviations were within acceptable limits and that they would not have an important influence on the conclusions. To simplify the study and presentation of experimental results, the final tests were conducted only with single testing. Below are presented and briefly discussed only some of the experimental results.

The maximum base acceleration a_{gmax} of columns C-PJ, C-DJ, and C-EJ in their limit state, with columns grouped according to the type of joint, are shown on Fig. 5, wherein the thin line indicates the accelerations at the collapse of the column, and the thick line indicates the acceleration from the previous acceleration increment with its stability preserved. The same results are shown in Fig. 6, where the grouping is conducted according to the applied

accelerogram. It is evident that the bearing capacity of the columns depends significantly on both the type of joints and the applied accelerogram.

Fig. 5 Ultimate bearing capacity of columns C-PJ, C-DJ, C-EJ – presented by joint type

Fig. 6 Ultimate bearing capacity of columns C-PJ, C-DJ, C-EJ – presented by accelerogram type

From analysis of Fig. 5, it can be concluded that, for all types of joints, the most unfavourable was the Petrovac accelerogram (AP), followed by the artificial accelerogram (AA); the most favourable was the Ston accelerogram (AS). For the considered accelerograms AP, AA, and AS, the ratio of maximum acceleration a_{gmax} at the column failure with powder joints C-PJ was approximately 1.0:1.1:3.0, that for the column with dry joint C-DJ was approximately 1.0:1.3:2.5 and that for the column with epoxy joint C-EJ was approximately 1.0:1.5:2.0.

The joint type effect was dependent on the type of applied accelerogram (Fig. 6). The lowest bearing capacity was found for a column with a dry joint (C-DJ) exposed to AP, and the highest bearing capacity was found for a column with powder joints (C-PJ) exposed to AS. The ratio of the bearing capacities of the columns C-PJ, C-DJ, C-EJ for AP was approximately 1.2:1.0:1.3, and that for AS was approximately 1.6:1.1:1.0. The column with epoxy joint C-EJ was found to behave similar to a column formed in one piece without a joints (monolith), as confirmed by tests on such column for AP and AA (the maximum difference in the results were 3.2 %).

1 For the purposes of validation of the abovementioned relationships, which at first sight are not
2 clearly evident, further explanation is given below.

3 Imperfections and nonlinearities in the joint can significantly affect the behaviour and bearing
4 capacity of multi-drum columns. A stone-powder joint allows a more uniform vertical stress
5 transmission than a dry joint. The amount of rotation in a stone-powder joint is significantly
6 smaller than that in a dry joint, where because of imperfections in the surface treatment, the
7 rocking may become more expressed. Transmission of shear in the powder joint occurs on the
8 entire surface of the joint, whereas in the dry joint, transmission of shear only occurs at the
9 contact points of the contact surfaces. Column C-DJ clearly had significantly higher rotations
10 (rocking) in the joints and greater relative horizontal displacements of the blocks at the
11 contact surfaces than column C-PJ as a result of the excitation.

12 The horizontal displacements of blocks in some points for some columns during the action of
13 some accelerograms are presented in Figures 7 to 9. The purposes of these tests are to gain
14 better insight into the column displacements at the successive increase of the maximum
15 amplitude of the accelerogram and to possibly validate the numerical models used for the
16 simulation of multi-drum columns under an earthquake.

Fig. 7 Horizontal displacements $u_1, u_4, u_5, u_8, u_9,$ and u_{12} of column C-PJ for the Petrovac accelerogram at different $a_{g,max}$

Fig. 8 Horizontal displacements $u_1, u_4, u_5, u_8, u_9,$ and u_{12} of column C-DJ for the Petrovac accelerogram at different $a_{g,max}$

Fig. 9 Horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 , and u_{12} of column C-EJ for the Petrovac accelerogram at different $a_{g,max}$

The appearance of deformed column C-DJ for the Petrovac accelerogram at certain times for $a_{g,max} = 1.6 \text{ ms}^{-2}$ is shown at Fig. 10. Significant relative horizontal displacements of adjacent stone blocks and vertical eccentricity of the column were noticed.

Fig. 10 Appearance of the column C-DJ for the Petrovac accelerogram at $a_{g,max} = 1.6 \text{ ms}^{-2}$

5.2 Columns Cm-PJ, Cm-DJ, and Cm-EJ

Before final testing of columns Cm-PJ, Cm-DJ, and Cm-EJ, the repeatability of tests was verified to confirm their reliability. Regarding the columns without the mass at the top, for AA, the column tests were conducted for columns Cm-PJ and Cm-DJ at $a_{gmax} = 0.6 \text{ ms}^{-2}$, and two tests were conducted for column Cm-EJ at $a_{gmax} = 1.0 \text{ ms}^{-2}$. The maximum difference of the measured results for column Cm-PJ, column Cm-DJ, and column Cm-EJ was 4.2 %, 6.4 %, and 1.8 %, respectively. According to the sensitivity of the problem, it is also concluded that the deviations are within the acceptable limits and that the final tests for these columns will also be conducted with a single test. Note that only certain experimental results are presented and discussed.

The results for columns Cm-PJ, Cm-DJ, and Cm-EJ are shown in Figs. 11 and 12, in the same manner as the results for the columns C-PJ, C-DJ, and C-EJ are shown in Figs. 5 and 6. The bearing capacity of each of the columns with the mass at its top is slightly smaller than the bearing capacity of the same column without the mass. It is also evident that the bearing capacity of each column with a mass on its top significantly depends on the type of joint and type of the applied accelerogram.

For all types of joints, the most unfavourable was the artificial accelerogram (AA), followed by the Petrovac accelerogram (AP). This result is the opposite compared to the results of the columns without the mass at their tops. This difference is explained by the fact that the mass at the top of the column additionally softened the column (see Table 1). Thus, AA, with the highest spectral displacements and velocities (Fig. 3), was more unfavourable than AP. Here, the most favourable was the Ston accelerogram (AS). For accelerograms AA, AP, and AS, the ratio of the maximum acceleration a_{gmax} at failure of the column with the powder joint Cm-PJ

was approximately 1.0:2.4:4.0, that for the column with the dry joint Cm-DJ was approximately 1.0:2.6:6.0, and that for the column with the epoxy joint Cm-EJ was approximately 1.0:1.1:2.5.

The effect of the joint type also depended on the type of applied accelerogram (Fig. 13). The lowest capacity was found for columns Cm-PJ and Cm-DJ for AS. The ratio of the bearing capacities of columns Cm-PJ, Cm-DJ, and Cm-EJ for AA was approximately 1.0:1.0:1.4, that for AP was approximately 1.5:1.6:1.0 and that for AS was approximately 1.1:1.7:1.0. The column with the epoxy joint Cm-EJ also behaved like a column formed of one block (i.e., without joints). This result is verified on the column from one block for AA, where the difference between the maximum result for the column from one block with the mass at the top and the column Cm-EJ was 3.1 %.

For the explanation of the abovementioned results, an explanation similar to that used for the columns without mass at the top is valid, with the additional factors stated below.

Compared to the columns without the mass at the top, the columns with a mass at the top had significantly additional vertical stress in the joints. This additional vertical stress resulted in a lower rotation and smaller relative displacements of the blocks at the joint, i.e., more efficient transfer of shear in the powder joint, and especially the efficient transfer of shear in the dry joint.

Fig. 11 Ultimate bearing capacity of the columns Cm-PJ, Cm-DJ, Cm-EJ – presented by joint type

Fig. 12 Ultimate bearing capacity of the columns Cm-PJ, Cm-DJ, Cm-EJ – presented by accelerogram type

The horizontal displacements of blocks in some points for some of the columns during the acting of some excitation are shown in Figs. 13 to 15. Data are also presented to possibly validate the numerical models used for seismic analysis of multi-drum columns.

Fig. 13 Horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 , and u_{12} of column Cm-PJ for the artificial accelerogram at different a_{gmax}

Fig. 14 Horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 , and u_{12} of column Cm-DJ for the artificial accelerogram at different a_{gmax}

Fig. 15 Horizontal displacements u_1 , u_4 , u_5 , u_8 , u_9 , and u_{12} of column Cm-EJ for the artificial accelerogram at different a_{gmax}

The appearance of deformed column Cm-DJ at certain times for AA at $a_{gmax} = 0.8 \text{ ms}^{-2}$ is shown in Fig. 16. The relative transverse displacements and the ultimate bearing capacity of the column for this column with mass at the top were significantly smaller than those for the same column without the mass at the top (C-DJ).

Fig. 16 Appearance of the column Cm-DJ for the artificial accelerogram at $a_{gmax}=0.8 \text{ ms}^{-2}$

By comparing the results shown in points 5.1 and 5.2, the significant impact of the mass on the top of the column on the behaviour and the ultimate bearing capacity of the column during an earthquake is observed. The research presented in this paper is a continuation of relevant research published in Refs. [29, 30, 33, 34].

6 Conclusions

This paper presented the results of a shake-table study of the effects of the joint type on the behaviour and bearing capacity of small-scale free-standing multi-drum columns loaded only by their own weight and by additional weight on their top. In the study, the columns are exposed to the base excitation of three different earthquake accelerograms, with successive increase of acceleration to the columns failure. Three types of joints are varied: a dry joint (DJ), a joint filled with stone powder (PJ) and an epoxy joint (EJ). Based on the results obtained and the conducted analyses, the conclusions below may be drawn.

Globally, the type of joint in multi-drum column has great effects on their behaviour and bearing capacity during an earthquake. As the joint becomes softer, the stiffness of the multi-drum column compared to the monolithic column decreases. In the conducted experiment, the ratios of the fundamental period of the considered columns with the joints PJ, DJ, EJ were approximately 2.8:3.2:1.0. Although the resistance of the column with softer joints is smaller in regard to the column from one block, a favourable circumstance is that it generates less earthquake (inertial) forces. Based on the aforementioned factor and taking into account the characteristics of the excitation (spectral accelerations, velocities and displacements, duration of the excitation and oscillation time), it is difficult to reliably determine the dependences of the multi-drum column bearing capacity on the joint type and the excitation parameters. The multi-drum column with an epoxy joint in the conducted tests practically behaved as a column from one block.

In the test conducted for columns with self weight only (C-PJ, C-DJ, C-EJ), their lowest bearing capacity was found for the Petrovac accelerogram (AP); greater bearing capacity was found for the artificial accelerogram (AA), and the largest bearing capacity was found for the Ston accelerogram (AS). The bearing capacity ratios of the abovementioned columns were

1 approximately 1.0:1.0:1.3 for AA, 1.3:1.0:1.1 for AP and 1.7:1.1:1.0 for AS. Thus, in the case
2 of AA, the most unfavourable joints were equally PJ and DJ; in the case of AP, the most
3 unfavourable joint was DJ; in the case of AS, the most unfavourable joint was EJ. The most
4 unfavourable effect of soft joints (PJ, DJ) for AA and AP is probably the consequence of the
5 highest inserted earthquake energy for these excitations and the least resistance of the column
6 with such joints. The most unfavourable effect of epoxy joint for AS is probably the
7 consequence of the highest inserted inertial force and the highest stiffness of column C-EJ in
8 relation to columns C-PJ and C-DJ. For all the considered accelerograms, the greatest
9 difference in the bearing capacity was approximately 300 % for PJ, 250 % for DJ and 233 %
10 for EJ. For all the considered joints, the greatest difference in bearing capacity was
11 approximately 27 % for AA, 25 % for AP and 71 % for AS.

12 In the test conducted for columns with the additional weight on the top (Cm-PJ, Cm-DJ, Cm-
13 EJ), the smallest bearing capacity was found for the artificial accelerogram (AA); a greater
14 bearing capacity was found for the Petrovac accelerogram (AP) and the largest bearing
15 capacity was found for the Stone accelerogram (AS). The bearing capacity ratio of the
16 abovementioned columns was approximately 1.0:1.0:1.5 for AA, 1.6:1.7:1.0 for AP and
17 1.2:1.8:1.0 for AS. Thus, in the case of AA, the equally most unfavourable joints were PJ and
18 DJ, and in the case of AP and AS, the most unfavourable joint was EJ. In relation to the
19 columns with self weight only, columns with a mass at the top had additional stiffening of the
20 soft joints via the additional vertical force. This is considered to be a reason why EJ was the
21 most unfavourable, even for AP. For all the considered accelerograms, the greatest difference
22 in bearing capacity was approximately 657 % for PJ, 687 % for DJ and 250 % for EJ. For all
23 considered joints, the greatest difference in bearing capacity was approximately 50 % for AA,
24 57 % for AP and 83 % for AS.

In summary, it can be concluded that the bearing capacity of the multi-drum column during an earthquake, inter alia, depends on the type of joint. Accordingly, depending on the type of earthquake and other influential parameters, the multi-drum column may have significantly smaller or even significantly higher bearing capacity than the single-block column. It can be expected that the soft joints (dry joint, joint with stone powder, lime mortar etc.) are the most unfavourable for the earthquakes that cause great displacements and bring much energy to the system. In other cases, it can be expected that stiff joints (epoxy joint, strong mortar joint, etc.) and the columns from the monolith will be the most unfavourable for impact earthquakes with short duration and less inserted energy.

It is considered that the experimental database can find application in the validation of numerical models in seismic analysis of multi-drum columns.

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Effects of several bolt parameters on the bearing capacity of a composite multi-drum stone column under an earthquake

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ABSTRACT

The paper presents some results of a shake-table study on the effects of several bolt parameters on the bearing capacity of a composite free-standing multi-drum stone column under an earthquake. In each test, a small-scale column with six blocks per height was exposed to different horizontal base accelerations until the column collapsed. Firstly, the effect of five different materials for the bolts were investigated on the column bearing capacity. Then, the effects of bolt length, bolt diameter and the ratio of the hole diameter to the bolt diameter on the column bearing capacity were investigated. Based on the test results, the main conclusions of the study are presented.

1. Introduction

The use of bolts for connecting stone blocks in numerous historic stone buildings least to a far past. The widest application of the bolts has been in the construction of multi-part stone columns and arches. The use of bolts is led by the traditional experience of the builders because they contribute to the greater resistance of stone structures, especially at earthquake. The experience that has been applied to date has been the result of monitoring the resistance and durability of real stone structures with and without bolts. Unfortunately, to the best of our knowledge, to date rare investigations have been conducted related to the efficacy of the application of bolts in stone buildings, which is also the motivation of the presented study. Below some such available investigations are briefly mentioned.

To reinstate the original timber pole and empolia at interfaces between stone members, Papadopoulos and Vintzileou [1] numerically analysed new titanium connectors for the column capitals of the classical temple of Apollo Epikourios (Greece). The investigation showed that installation of a single pole at the interface between the capital and the uppermost drum leads to limited reduction of maximum and residual deformations of the column under strong earthquakes. Furthermore, poles located at every column interface ensure significant limitation of the column residual deformation. The main aspects of the design of new titanium poles and empolias were verified through specially designed experiments.

Konstantinidis and Makris [2] numerically investigated the seismic response of multi-drum column with motivation to understand the

possible advantages or disadvantages of retrofitting columns with metallic shear links that replace the wooden poles that were installed in ancient times. Vintzileou and Toubakari [3] experimentally studied the dowel action of titanium bars connecting marble fragments at different angles. Psycharis et al. [4] numerically investigated a column-architrave classical structure under seismic excitations. A proposal of reinforcement with titanium connections was analysed, and it was found that architrave connections generally improve the response by decreasing the permanent displacements. However, it was found that titanium dowels between the column drums do not improve significantly the behaviour and, in some cases, they are unfavourable to the safety of the structure. Zambas et al. [5] experimentally tested the titanium reinforcement for the restoration of marble architectural members of the Acropolis monuments.

Below are some experimental and numerical studies that have been related to the analysis of the influence of some other important parameters on the behaviour and bearing capacity of multi-drum stone columns under static and seismic loads.

Buzov et al. investigated the effects of drum height [6] and joint type [7] on the bearing capacity of the multi-drum column under static load. Moreover, Buzov et al. [8] investigated the effect of drum height on the seismic behaviour of a free-standing multi-drum column. Ascione et al. [9] numerically analysed the load carrying capacity of 2D FRP/strengthened masonry structures. D'Ambrisi et al. [10] experimentally and analytically investigated the bond between Carbon-FRCM materials and masonry. Alecci et al. [11] performed experimental investigations of brickwork columns confined with CFRP composite.

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Foraboschi [12] experimentally investigated effectiveness of novel methods to increase the FRP-masonry bond capacity. Also Foraboschi and Vanin [13] experimentally investigated new methods for bonding FRP strips onto masonry structures. Foraboschi [14,15] analysed stone columns in the framework of buildings of famous Italian architecture. Drosos and Anastasopoulos analysed an ancient Greek temple [16] and a free-standing small-scale multi-drum column [17]. Papalou [18] investigated the dynamic response of classical columns. Manos and Demosthenous [19] investigated models of ancient columns and colonades subjected to horizontal base motions in a study of their dynamic and earthquake behaviour. Papaloizou and Komodromos [20] studied multi-drum ancient columns and colonnades using the discrete element method. Konstantinidis and Makris [21] analysed multi-drum classical columns, Komodromos et al. [22] studied ancient columns, and Kawadiaz et al. [23] performed a seismic response parametric study of ancient columns.

The mechanical behaviour of classical multi-drum stone columns under earthquake, with stone blocks closely fitting together or jointed by dowels, clamps and other connecting elements, is the result of the complex interaction of an articulated composite assemblage. Because of the dominantly brittle behaviour of the stone blocks, the capability of the articulated assemblage to resist the seismic actions is dominantly based on external energy absorption mechanisms, such as rotations, slidings, and oscillations (rocking) of individual stone blocks. Possible different connections between the blocks have the purpose of contributing to the bearing capacity of the assembled structure under earthquake.

The scope of presented paper was to experimentally studied the efficiency of the bolts in multi-drum stone columns under earthquake, because this problem was not adequate analysed jet. The effects of several bolt parameters on the behaviour and ultimate bearing capacity of the free-standing composite multi-drum stone column during an earthquake were investigated by shake table. The effects of the material of the bolt, the length of the bolt, the bolt diameter and the diameter of the hole for the bolt were investigated. Each column is exposed to two earthquakes, with a successive increase of the maximum acceleration up to its failure or loss of stability. Selected study results are presented, and important conclusions for practical application are given. The main conclusions of the study is that the multi-drum columns with bolts have significantly higher bearing capacity than columns without bolts, that the bearing capacity depends significantly on the type of the earthquake and that the research results of the bolts parameters effects can be reliably used in practice. A detailed presentation of test results can be useful at validating of nonlinear numerical models for dynamic analysis of multi-drum stone columns.

2. Tested column

The free-standing small-scale multi-drum column presented in Fig. 1 was tested. The column is formed from six identical blocks with a circular cross-section of 192 mm in diameter and 200 mm in height. The dry joint between blocks was adopted, as the majority of ancient multi-drum columns are made with such a joint (direct block-to-block placement). The dry joint cannot transmit vertical tensile stresses, and it results in horizontal shear discontinuity of the adjacent blocks. The total height of the column is approximately 1200 mm, with a slenderness of approximately 35. The transmission of horizontal shear stresses and vertical compressive stresses in the joint can be uniform and with concentrations and discontinuities, depending on the flatness of contact block surfaces. The tested column represents a very complex composite structure formed from different materials and features a complex interaction of articulated assemblage.

The column is placed on a rigid concrete plate of 100 mm in thickness that is fixed to the shake-table. To achieve a significantly greater uniformity of block quality and to shorten the duration and cost of the study, concrete blocks were used instead of stone blocks. It is

Fig. 1. Tested column.

considered that this approach does not have a great impact on the given conclusions because the concrete is very similar to the stone and the failure of all tested columns was only by the loss of column stability. This approach was also used in Refs. [6–8].

The positions of the bolts in the column are also presented in Fig. 1. The bolts are placed between the blocks and between the concrete plate and the bottom block. To test the contribution of bolts, the same multi-drum column without bolts was tested.

Apart from the impact of the bolts, the behaviour of the multi-drum column during the earthquake affected, among other things, the number of blocks in the column and the type of joint [8].

3. Bolts

To investigate the effect of bolt material at column bearing capacity, five types of bolts from different materials were tested (with other equal parameters). Bolts with diameter $\phi_b = 12\text{mm}$ and height $h_b = 120\text{mm}$ were adopted, with the diameter of the bolt hole of $\phi_h = 14\text{mm}$ (Fig. 2). The bolts are freely inserted into the hole, without filling the rest of the space between the bolt and the block. The goal was to analyse the contribution of the bearing capacity of the bolts to the shear bearing capacity of the column, without the contribution of the bolts to the carrying bending moments in the column.

The adopted bolt has the following ratio of its diameter (ϕ_b) to

Fig. 2. Adopted geometry of the bolt.

Table 1
Basic bolt materials parameters.

	Wood	Lead	Rubber	Aluminium	Stainless steel
Tensile strength [MPa]	55	13	14	310	500
Modulus of elasticity [GPa]	15.5	12	0.1	70	193
Shear modulus [GPa]	5.70	4.29	0.03	26.32	73.66
Poisson ratio	0.36	0.40	0.45	0.33	0.31

column diameter (d): $\phi_b/d = 1/16$. The ratio of the bolt height (h_b) to the bolt diameter is $h_b/\phi_b = 10$, and the ratio of the diameter of the hole for the bolt and the bolt diameter is $\phi_h/\phi_b = 1.17$. It is considered that the geometry of the adopted bolt is consistent with the geometry of the tested column and is within the framework of practical application.

The wood bolts are made from quality dry oak. Industrial products were used for the lead bolts, the rubber bolts, the aluminium bolts, and the stainless steel bolts. Some basic properties of the materials used for the bolts are shown in Table 1. According to their properties, aluminium and stainless steel bolts are categorized as rigid bolts, the wood bolts are categorized as medium-hard bolts and the bolts of lead and rubber are categorized as soft bolts.

The effect of the length of the bolt (h_b) to the bearing capacity of the column was tested on lead and stainless steel bolts. It is adopted bolt diameter $\phi_b = 12\text{mm}$ and a diameter of the hole $\phi_h = 14\text{mm}$. Three lengths of the bolts were considered: $h_b = 72\text{mm}$ (short bolt), $h_b = 120\text{mm}$ (medium long bolt) and $h_b = 168\text{mm}$ (long bolt). The ratio h_b/ϕ_b for the short, medium-long, and long bolts is 6, 10 and 14, respectively.

The effect of the bolt diameter (ϕ_b) on the bearing capacity of the column was also tested for bolts made of lead and stainless steel. Bolts with $\phi_b = 12\text{mm}$ and $\phi_b = 20\text{mm}$ (with $\phi_h = \phi_b + 2\text{mm}$ and $h_b = 120\text{mm}$) were adopted, with the ratio ϕ_h/ϕ_b of 1/16 and 1/9.6, respectively.

The effect of the hole diameter (ϕ_h) on the bearing capacity of the column was also tested on both lead and stainless steel bolts. Bolts with diameter $\phi_b = 12\text{mm}$ and length $h_b = 120\text{mm}$ were considered. The diameters of the bolt hole were $\phi_h = 14\text{mm}$ and $\phi_h = 22\text{mm}$. The ratio ϕ_h/ϕ_b for the observed bolts is 1.17 and 1.83, respectively. Images of the bolts used are shown in Fig. 3.

4. Base accelerograms

The tested column in Fig. 1 is exposed to the horizontal base accelerograms presented in Fig. 4. Only two accelerograms are adopted because of the shortened duration of the study. It is considered that this will not affect the given conclusions because the selected accelerograms had completely different properties and represent the boundary cases of the most common types of opposite earthquakes. Specifically, the accelerogram shown in Fig. 4.a is an artificial accelerogram (AA) performed on the elastic response spectrum [24] for type 1 and soil type A. AA represents earthquakes with a long duration and a relatively long predominant period. The accelerogram in Fig. 4.b is a horizontal component N-S of the Ston earthquake (AS) [25]. AS is a short duration earthquake with a relatively short predominant period (impact-type earthquake).

The spectral values of the adopted accelerograms are shown in Fig. 5. For comparison of the spectral values of the adopted excitations with the tested column (Fig. 1), the first period of free oscillation of the column (T_1) was experimentally determined. For the column without bolts, $T_1 = 0.174\text{ s}$. For the column with bolts of stainless steel (column with the highest stiffness), $T_1 = 0.151\text{ s}$. If these periods are compared with the spectral value diagrams from Fig. 5, it can be expected that AA will be more unfavourable compared to AS. As the diagrams in Fig. 5 are formed for an elastic single-degree column, and the considered columns probably will be softened during the applied excitations, it is difficult to define in advance the most unfavourable excitation for the considered complex composite non-linear problem. The intention was to choose two different types of accelerograms in terms of their durations and their predominant periods during maximum accelerations.

The accelerograms in Fig. 4 were applied to each of the considered columns with the successive increase in the maximum acceleration ($a_{\max}$) until collapse occurred. The acceleration increment $\Delta a_{\max}$ was adopted as 0.2 ms^{-2} and 0.5 ms^{-2} for AA and AS, respectively. Although each differently applied accelerogram resulted in a different response for each of the considered samples, it is considered that only two selected accelerograms are relevant for the credibility of the given conclusions.

5. Measurement and equipment

Horizontal displacements (u_i) are measured 5 mm below the top and 5 mm above the bottom of each block (Fig. 6). The acceleration (a_i) values are measured in the middle of the height of each block.

Fig. 3. Images of the bolts used.

Fig. 4. Applied accelerograms.

Fig. 5. Spectral accelerations, velocities and displacements of the applied accelerograms.

Fig. 6. Measured quantities.

Deformations of the column during base excitation were tracked by a precise camera.

Testing of the columns was performed using the shake-table at the University of Split. Displacement sensors PB-25-S10-NOS-10C (UM) were used for the displacement measurements. Piezo-electric high-frequency accelerometers type 4610 (MS) were used for acceleration measurements. The Quantum-x mx 840 A system (HBM) was used for data collection from the sensors.

6. Test results

6.1. Effect of bolt material

The bearing capacity of the tested column, i.e., the maximum acceleration at which its failure occurred, for different bolt materials is shown in Fig. 7. The results are presented separately for the artificial

accelerogram (AA) and the Ston accelerogram (AS). The results for the case without bolts are also shown. Note that the bearing capacity for AA is approximately two-times lower than that for AS. The reason for this result is that the AA caused larger column displacements and had stronger oscillatory motion, resulting in higher energy in the column. For both applied accelerograms, clearly, the bearing capacity of the column without bolts is significantly lower than for the column with bolts. For AA, the bearing capacity of the column without bolts is approximately 68% of the average bearing capacity of the column with bolts. For AS, the bearing capacity of the column without bolts is approximately 59% of the average bearing capacity of the column with bolts. Greater efficiency of bolts for AS is the result of its impact action, i.e., the greater transverse force generated in the column.

Fig. 7 clearly shows that, for both applied accelerograms, relatively little differences were observed in the efficiencies of the considered types of bolts, i.e., the bearing capacity of the column was found to be relatively little dependent of the material of the bolt. The maximum capacity of the column was ensured by hard bolts (from aluminium and stainless steel), a somewhat smaller capacity was provided by medium hard bolts (from wood) and the smallest capacity was provided by soft bolts (from lead and rubber). For AA, the bearing capacity ratios of the column without bolts and the column with wood, lead, rubber, aluminium and stainless steel bolts are 1: 1.5: 1.42: 1.25: 1.5: 1.67. For AS, the abovementioned ratios are 1: 1.67: 1.56: 1.56: 1.78: 1.89.

The column without bolts had the expressed relative horizontal displacements between adjacent stone blocks by the column height, especially at higher maximum acceleration. As a result, eccentricities appeared in the transmission of the vertical load per column height. For the columns with bolts, no visually noticeable relative horizontal displacements of adjacent blocks per column height were found. In particular, the bolts largely provided the continuity of lateral displacements of adjacent blocks and ensured the efficient transfer of horizontal transverse forces in the column. This phenomenon is considered to be the main reason that the column with the bolts had significantly higher bearing capacity than the column without bolts. The collapses of all columns with bolts appeared in the same manner: with the loss of stability of the column via the pulling of the bolt at the column base.

Fig. 7. The effect of the bolt material on the column bearing capacity ($h_b = 120 \text{ mm}$, $\phi_b = 12 \text{ mm}$, $\phi_h = 14 \text{ mm}$).

The relatively low effect of the bolt material on the bearing capacity of the tested column is a result of the relatively small induced transverse earthquake forces and did not reach the bolts shear capacity. As previously mentioned, the collapse of the column was not followed by the shear collapse of the bolt, but it was followed by the pulling of the bolt at the base of the column during its rotation. The results would probably be different in the case of a column with an additional mass at the top; this case is planned to be investigated. The results would also probably be different if an adhesive between the bolt and block is used, i.e., the additional bearing capacity of the bolt at the bending of the column is achieved; this case is also planned to be investigated.

Horizontal displacement of the column top for AA at $a_{max} = 2.2 \text{ ms}^{-2}$ is shown in Fig. 8. It is evident that all types of bolt provide similar oscillatory column motion. The largest permanent horizontal displacement had a column without bolts.

Horizontal displacement of the column top for AS at $a_{max} = 4.0 \text{ ms}^{-2}$ is shown on in Fig. 9. It is also evident that all types of bolts clearly exhibit uniform oscillatory column displacements, and the largest permanent displacement occurred for a column without bolts.

For AA at $a_{max} = 2.2 \text{ ms}^{-2}$, the deformations of the column without bolts and with stainless steel bolts in some discrete times are shown in Fig. 10. In relation to the columns with bolts, the columns without bolts had far more pronounced relative horizontal displacements between adjacent blocks.

Images of some bolts in the base of the column after its collapse under AA action are shown in Fig. 11. It is evident that the lead bolt has the most pronounced lateral deformations.

6.2. Effect of the bolt length

The effect of the length of the adopted bolt on the bearing capacity of the tested column is shown in Fig. 12. Note that the effect is small and almost negligible because the adopted bolt primarily carries the shear forces, with the negligible capacity for bending moments.

6.3. The effect of the bolt diameter

The effect of the diameter of the adopted bolt on the bearing capacity of the tested columns is shown in Fig. 13. Clearly, the effect is significant for AA, whereas it is negligible for AS. For AA, a thicker bolt ($\phi_b = 20 \text{ mm}$) gives approximately 31% lower bearing capacity for the lead bolt and approximately 43% lower bearing capacity for the stainless steel bolt in relation to the thinner bolt ($\phi_b = 12 \text{ mm}$).

Reducing the bearing capacity of the column with the increase of the bolt diameter for AA is explained by the fact that the increase in the bolt diameter increases the stiffness of the column, resulting in the generation of higher inertial forces during such earthquakes, which in turn causes larger displacements and higher energy in the column. Thus, the increased shear bearing capacity of the bolt has no effect because the collapse of the column occurs via its rotation and the pulling of the bolt from the hole.

The almost equal capacity of the thicker and thinner bolts for AS explains the dominant effect of shear with respect to bending of the column for such excitation.

In the case of a column with an additional mass at the top, i.e., with larger seismic forces, the results of the tests would probably be different and thus should be investigated.

Fig. 8. Horizontal displacement of the column top for the artificial accelerogram (AA) at $a_{max} = 2.2 \text{ ms}^{-2}$ ($h_b = 120 \text{ mm}$, $\phi_b = 12 \text{ mm}$, $\phi_h = 14 \text{ mm}$).

Fig. 9. Horizontal displacement of the column top for the Ston accelerogram (AS) at $a_{\max} = 4.0 \text{ ms}^{-2}$ ($h_b = 120 \text{ mm}$, $\phi_b = 12 \text{ mm}$, $\phi_h = 14 \text{ mm}$).

Fig. 10. Deformations of the column without bolts and column with stainless steel bolts for the artificial accelerogram (AA) at $a_{\max} = 2.2 \text{ ms}^{-2}$.

Fig. 11. Images of some bolts in the base of the column after collapse for the artificial accelerogram AA ($h_b = 120 \text{ mm}$, $\phi_b = 12 \text{ mm}$, $\phi_h = 14 \text{ mm}$).

Fig. 12. The effect of the bolt length (h_b) on the bearing capacity of the column ($\phi_b = 12 \text{ mm}$, $\phi_h = 14 \text{ mm}$).

6.4. The effect of the hole diameter

The effect of the hole diameter for the bolt in relation to the bolt diameter is shown in Fig. 14. Note that the effect is significant. For AA,

with the increase of the hole diameter in relation to the bolt diameter, the bearing capacity of the column with lead and stainless steel bolts is reduced by approximately 21% and by approximately 33%, respectively. For AS, the aforementioned reduction in column bearing

Fig. 13. The effect of the bolt diameter (ϕ_b) to the bearing capacity of the column ($h_b = 120 \text{ mm}$).

Fig. 14. The effect of the hole diameter on the bearing capacity of the column ($h_b = 120$ mm, $\phi_b = 12$ mm).

capacity is approximately 17% and 21%, respectively.

Reduction of the column bearing capacity by increasing the hole diameter in relation to the bolt diameter is explained by the fact that the larger hole diameter enables an increase of the relative displacement between adjacent blocks across the column height, i.e., the inertial/earthquake forces and eccentricities of vertical load are increased. The conclusions would probably be different in the case of the application of adhesive in the space between the bolt and the block; this case should be investigated.

7. Conclusions

Based on the experimental research studies of free-standing composite multi-drum columns exposed to the horizontal base accelerations of the two different earthquake accelerograms, with the successive increase of their maximum acceleration up to the collapse of the column under test, the main conclusions can be made below. It must be emphasized that the earthquake accelerograms had different dynamic characteristics, i.e., the different predominant period and duration. One of the accelerograms exhibited a long duration and long predominant period (AA), and the other exhibited a short duration and short predominant period (AS).

In all the conducted tests, a column without bolts had a lower ultimate bearing capacity compared to the same column with bolts. The ratio of these bearing capacities was found to depend on the type of earthquake and the bolt material and was in the range of approximately 1: 1.25 to approximately 1: 1.89. It is obvious that the bolts significantly contributed to the bearing capacity of the tested multi-drum columns during the earthquake.

The bearing capacity of a column with and without bolts was found to be strongly dependent on the type of earthquake. The bearing capacity was significantly higher for the Ston accelerogram (AS) than for the artificial accelerogram (AA). This phenomenon is explained by the fact that the AS was generated the less earthquake energy in the column than AA, resulting in larger displacements of the column. The bearing capacity ratio of the column with the bolts to that without the bolts was approximately 1: 1.47 for AA and approximately 1: 1.69 for AS. The greater efficiency of bolts for AS is explained by the fact that it has an impact character, i.e., it has a greater influence of shear compared to bending.

With successive base accelerations, the relative horizontal displacements between the adjacent blocks at the column height were found to be very pronounced in the column without bolts (especially for AA). Thus, a large eccentricity of the column relative to its base and an accelerated loss of the column stability were observed. At the column with bolts, the relative horizontal displacements of adjacent blocks by

column height were not visually noticeable. Because of the lower eccentricity of the column weight by its height, the bearing capacity of the column with bolts was significantly higher than the bearing capacity of the column without bolts.

The failure of all the columns with the bolts originated from the loss of their stability because of rotation of the column in the base and pulling of the lowest bolt. The diameter of all used bolts was smaller than the hole diameter, i.e., the adopted bolts carry only the shear (without carrying the bending).

With the same diameter and length of the bolts, the ratio of bearing capacity for the column with bolts of wood, lead, aluminium and stainless steel for AA and AS was approximately 1: 0.94: 0.83: 1: 1.11 and approximately 1: 0.93: 0.93: 1.07: 1.13, respectively. Obviously, the effect of bolt material did not have a major influence in the conducted test. This is explained by the fact that the bolt shear capacity was not reached; however, pulling of bolts occurred at the loss of column stability. In the case of larger earthquake shear forces, the impact of bolt material may be significant.

With the same material for the bolt (lead, stainless steel) and bolt diameter (12 mm), the bolt length (72 mm, 120 mm, 168 mm) had very little effect on the ultimate bearing capacity of the column at the earthquake. In particular, the bearing capacity of the column with the longest bolt in relation to the bearing capacity of the column with the shortest bolt reached the maximum of approximately 5.9% and approximately 7.7% for AA and AS, respectively. The reason for this result is that the adopted bolts practically had no capacity for bending, but only for shear. By using the bolts that have additional bearing capacity for bending (enabling transmission of vertical shear between the bolt surface and the stone block), the effect of the bolt length could be significant.

With the same material for the bolt (lead, stainless steel) and the same length of the bolt (120 mm), the bolt diameter (12 mm, 20 mm) had an unexpected effect on the bearing capacity of the column. For AA, the relation of the ultimate bearing capacity of the column with the bolt diameters of 12 mm and 20 mm was 1: 0.76 for the lead bolt and 1: 0.7 for the stainless steel bolt. The lower bearing capacity of the column with a larger bolt diameter can be explained by the greater stiffness of the column provided, resulting in the generation of larger earthquake forces. As the potentially higher shear bearing capacity of the bolt has not been realized, the column with a larger bolt diameter collapsed at a lower maximum acceleration than the column with a smaller bolt diameter. For AS, practically no difference in the bearing capacity of the columns was observed for different bolt diameters. This is explained by the fact that AS did not cause larger column displacements, i.e., it did not generate larger earthquake forces in the column with a larger bolt diameter in relation to the column with a smaller bolt diameter.

Regarding the abovementioned results, note that the smaller bolt with the 12 mm diameter is still relatively large in relation to the column diameter (approximately 1/16).

With the same material for the bolt (lead, stainless steel), the same bolt diameter (ϕ 12 mm) and the same length (120 mm), the hole diameter (14 mm, 22 mm) had a significant effect on the seismic bearing capacity of the column. For AA, the relation of the column bearing capacity with the hole diameter of 14 mm and 22 mm for the lead bolt was approximately 1: 0.82 and for the stainless steel bolt was approximately 1: 0.75. For AS, the relation of the column bearing capacity with the hole diameters of 14 mm and 22 mm for the lead bolt and the stainless steel bolt was approximately 1: 0.86 and approximately 1: 0.82, respectively. The reduction of the column bearing capacity with the increase of the bolt hole diameter relative to the bolt diameter is explained by the fact that the larger hole allows larger relative displacements between the adjacent blocks, larger seismic forces and additional vertical eccentricity, resulting in the lower bearing capacity of the column with a larger hole. This behaviour does not apply to the case where the space between the hole and bolt is filled with a bonding material; this approach can contribute to a significant increase in the bolt bearing capacity at the bending of the column.

Globally, it can be concluded that the study is innovative and contributes to new scientific findings in the research area, and that the results of the study can be reliably used in practice.

It is considered that the experimental database of this study can be useful for the verification of numerical models for analysis of the stone columns strengthened by bolts.

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