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Seismic Analysis of a Stone Masonry Aqueduct using Discrete Elements

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ABSTRACT

The seismic behavior of a stone masonry aqueduct is examined using discrete element models, based on a deformable block formulation. The work reported herein is the third phase of a study carried out on Águas Livres aqueduct in Lisbon. In the first phase of the study, two-dimensional models were used to analyze independently the in-plane and out-of-plane responses of the structure. In the continuation of this research, three-dimensional discrete element models provided a more realistic representation of the behavior of the aqueduct under seismic loading. The present paper focuses on the analysis of the out-of-plane response of one of the tallest pillars in the structure with a 2D model. Some important problems are addressed such as the influence of different joint patterns on the overall seismic behavior and the handling of high compressive stresses, causing either block splitting or crushing of block corners and mortar joints.

INTRODUCTION

The use of discrete element methods has been generalized to various fields that require tools for the numerical simulation of the mechanics of systems of particles or blocks. Masonry provides a natural application for these techniques, as the deformation and failure modes of these structures are strongly dependent on the role of the joints. This approach is well suited for collapse analysis, and may thus provide support for studies of safety assessment, namely of historical stone masonry structures.

The Águas Livres aqueduct in Lisbon (Fig. 1), built in 18th century, survived the 1755 earthquake (one of the strongest in the world) with only minor damage, in spite of the great losses produced in the surrounding region (Oliveira et al. 1995). It is a historical structure that employs pillars with cross-sections composed by an outer stone shell and some in-fill material. Given the importance of this historical construction, it has been the object of several experimental and analytical studies (Oliveira et al. 1991). Different numerical techniques were used to assess the dynamic behavior of the aqueduct. Early studies employed finite element models, assuming continuous idealizations (Azevedo & Drei 1995). Later, studies based on the discrete element method showed the ability of this approach to provide insight into the behavior of this masonry structure, particularly having in view the assessment of ultimate loading scenarios (Sincraian et al. 1998, Lemos et al. 1998).

The present work is the third phase of a numerical study carried out on Águas Livres aqueduct in Lisbon. In the first phase of the study, two-dimensional models were used to analyze independently the in-plane and out-of-plane responses of the structure. The blocks were assumed elastic. Time domain analyses were performed with different seismic records, scaled by various factors in order to induce structural failure. The results obtained with 2D models gave support to previous studies, which pointed to the good performance of the aqueduct under seismic events. In the continuation of this research, 3D discrete element models of a single pillar provided a more realistic representation of the heterogeneous structure. The understanding of the mechanical behavior of this composite structure is an essential step to improve the safety assessment capabilities.

This paper focuses on the analysis of the out-of-plane response of one of the tallest pillars in the structure. The collapse of the aqueduct would certainly involve an out-of-plane mode. Some important problems are addressed such as the influence of different joint patterns on the overall seismic behavior and the handling of high compressive stresses observed in the previous analyses, causing either block splitting or crushing of block corners and mortar joints. The possible fracturing and detachment of the limestone shell were accounted for by considering some cracks in the blocks.

DISCRETE ELEMENT MODELING

Discrete element methods represent a structure as an assembly of component blocks in mechanical interaction across joint surfaces. In the code UDEC (Itasca 1996), used in the present study, blocks may be either rigid or deformable, the latter being discretized into a finite element mesh. The representation of contact between blocks is not based on joint elements,

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but relies on sets of point contacts, of either vertex-to-vertex or vertex-to-edge type (in 2D). The assignment of contact areas allows the interface constitutive relations to be formulated in terms of stresses and relative displacements across the joint. An advantage of this approach is the natural transition it allows into the large displacement regime, as the contact locations and orientations are continuously updated in the course of analysis. UDEC includes efficient routines for contact detection and update. The solution procedure is based on the explicit time integration of the equations of motion of the rigid blocks, or the nodal points of deformable blocks. This technique is also used for quasi-static problems, using artificial viscous damping controlled by an adaptive algorithm.

DESCRIPTION OF THE MODELS

Geometry, block structure and mechanical properties

In this study, only the central part of the "Águas Livres" aqueduct was analyzed. The length of the modeled region is 280 m out of 370 m of the arched section that extends over the Alcântara Valley. The lowest part of the valley is crossed by the largest and tallest arch – about 40 m wide and 65 m high (58 m of free height). The other arches are about 26 m wide. The numerical model includes ten pillars with gothic arches (Fig. 1). The pillars have different heights ranging from 30 to 65 m, and, in their slender part, an average section of 7 x 9 m. Some have a visible enlargement at the base, with a section of 12 x 9 m; for others, the enlargement is buried. At the top of the structure there is the water channel that runs along the axis of the aqueduct, protected by an accessible gallery. There is also an aeration tower every two pillars.

The aqueduct was built, according to the 18th century usual technology, with two different materials (Oliveira et al. 1991). The external part is a regular wall made with good limestone. The stones are well cut having approximately similar dimensions. The joints between stones are very thin. The mortar seems to be compact and of good quality.

The thickness of this external wall was assumed to be one meter in both directions, while higher values were assumed for the pillars' enlargements (about 1.8 m in the transverse direction and 2.8 m in the longitudinal one). The inner part of the structure is filled with a heterogeneous masonry made up by mortared rubble and irregular stones.

Two-dimensional models were created for independent analyses in the longitudinal and transverse directions and are extensively described elsewhere (Sincraian et al. 1998). The different material regions used in the longitudinal model are shown in Figure 1, in different colors. Also, the block structure can be noticed.

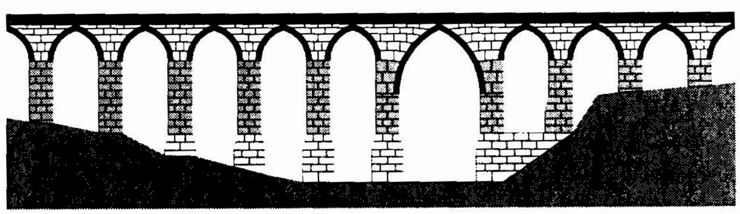


Figure 1. Longitudinal model

The numerical representation of such a large and complex structure involves many simplifications. The aim of a discrete element model is not to include every block, but only enough joints to allow the essential deformation and failure modes to develop. Therefore, only some joints are selected, and the block size in the numerical model is much larger than in reality. The basic transverse section model (Fig. 2a) corresponds to the tallest pillar. In a 2D model, it is impossible to represent the real structure of the pillars, with the outer limestone shell and fill, so a blocky pattern was created, with equivalent material properties, to account for the global behavior of the pillar section. Equivalent pillar properties were used in the slender part, which were scaled as a function of the out-of-plane thickness, to simulate the thicker base and upper sections. In the upper region, thickness accounts for the half-bay arches on either side of the pillar. Several different joint patterns were analyzed for the transverse models shown in Figs. 2b and 2c.

Deformable blocks with a coarse internal mesh were used. Blocks were assumed to be elastic. Failure mechanisms involving joint opening or sliding were investigated. Meanwhile, some cracks in the blocks were considered to accommodate possible fracturing and detachment of the limestone shell (Figs. 2d and 2e). Block properties were the following:

	Young's modulus	Poisson's ratio	Density
Limestone :	50 GPa	0.24	2700 kg/m^3
Pillars :	20 GPa	0.24	2500 kg/m^3

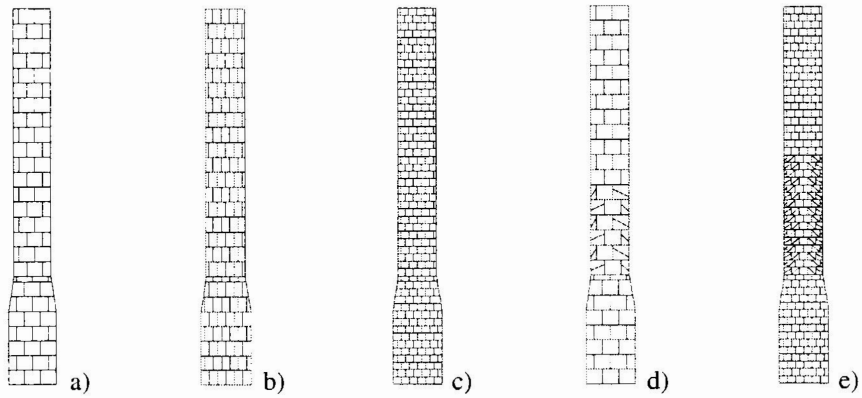


Figure 2. The models developed in the study

The pillar's equivalent Young's modulus is similar to the one used in previous studies in order to match the natural frequencies measured *in situ* (Oliveira et al. 1991). In the transverse direction the UDEC model lowest frequency is 0.9 Hz, as obtained in the tests.

For the mortar joints, the model employed assumed constant elastic stiffnesses, zero tensile strength and a Mohr-Coulomb failure criterion in shear, with the properties shown in table I.

For the limestone joints, the Coulomb slip model with residual strength was used in order to approximate a displacement-weakening response. This is accomplished by setting the joint friction, cohesion and tensile strength to reduced values (usually zero) whenever either the tensile or shear strength is exceeded (Itasca 1996). The assumed properties for the joints are presented in table I.

Table I. Adopted mechanical properties for the joints

	Normal stiffness (GPa/m)	Shear stiffness (GPa/m)	Friction angle	Cohesion (MPa)	Tensile strength (MPa)	Residual cohesion (MPa)	Residual friction	Residual tensile strength (MPa)
Mortar joints	100	100	30°	1.0	0	–	–	–
Limestone joints	100	100	40°	7.0	2.0	0	40°	0

In the dynamic analyses, only the mass-proportional component of Rayleigh damping was used, corresponding to 2.5% of critical damping at 0.9 Hz for the transversal model. The stiffness-proportional component was not used to avoid the reduction of the time step.

Loading

All the analyses were performed in two stages: first, each model was brought to equilibrium under its own weight; then, the seismic input was applied at all the nodes along the model base. For this phase of the study two records were considered. The first one was an artificially generated accelerogram, according to the Portuguese code for a long-distance earthquake in a stiff soil (type 2, soil type 1), with a peak ground acceleration of 0.11g (Guerreiro, 1998), assumed to represent an event of the type of the 1755 Lisbon earthquake. The second seismic input was the NS component of the 1977 Romanian earthquake, corresponding to a deep earthquake with a low frequency content, having one big shock and a peak ground acceleration of 0.21g. The two records are displayed in Figure 3.

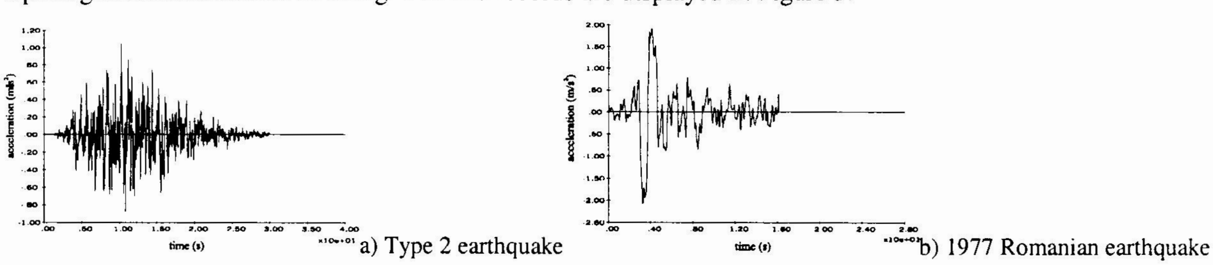


Figure 3. The seismic records used in the study

For each type of model a series of runs was performed, with the input record scaled by factors ranging from 1.0 to 4.0.

The first phase of the study carried out on the aqueduct (Sincraian et al. 1998) showed that the most severe action for this type of structure is the 1977 Romanian earthquake, due to the fact that it is especially rich in the low frequency domain. Therefore, this record was extensively used in the work presented herein.

Since the dynamic loading discussed above led to considerable damage in the structure, a second earthquake of a reduced size was applied on two of the models to study its effect. Again, the 1977 Romanian earthquake was considered for this stage in the study.

ANALYSIS OF RESULTS

As stated in previous studies (Oliveira et al. 1991), the aqueduct survived the 1755 earthquake without significant damage. This may have been due to the fact that the earthquake acted mainly in the longitudinal direction with respect to the aqueduct. In this direction the structure is very stiff and thus it is not affected by the low frequency content of this seismic motion. The collapse of the aqueduct would certainly involve an out-of-plane mode. Therefore, for safety assessment capabilities, an extensive out-of-plane analysis of the seismic response of this structure is necessary. The work reported herein addresses some of the problems encountered in a previous study on the transverse model of the aqueduct (Sincraian et al. 1998). The normal stress peaks obtained in some sections (40 MPa for the Romanian record with a pga of 0.2g) were above the limestone strength. In order to solve this problem, several different joint patterns based on the initial transverse model, used in the study mentioned above, were considered and the possible fracturing and detachment of the limestone shell were accounted for. The chosen joint patterns were obtained by halving the block sides, both vertically and horizontally. The 1977 Romanian earthquake record was used for the input. Figure 4 displays the damaged structure at the end of the input (magnified 1.5 times) for two different joint patterns.

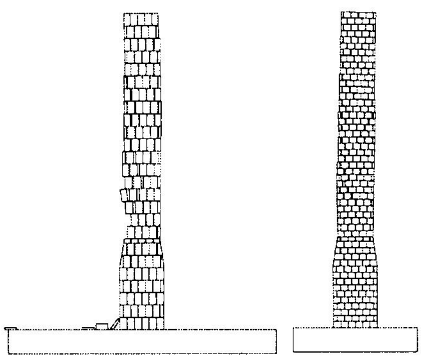


Figure 4. The damaged structure for pga = 0.3g

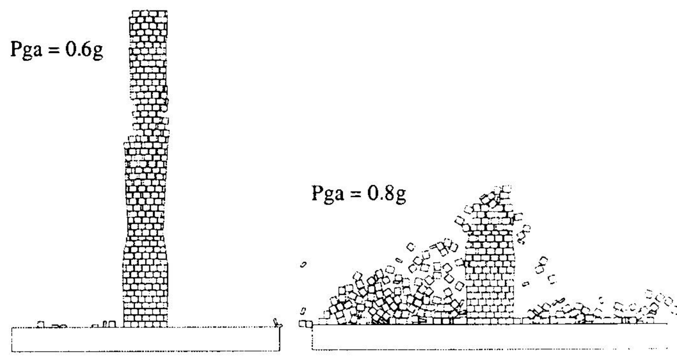


Figure 5. The damaged pillar for two different pga levels

It can be noticed that the structure is already severely damaged, some blocks are falling and the joint shear and separation are out of an acceptable range. The maximum relative top displacement is more than 1 m. Also, the initiation of the failure mechanism is clearly observed. Obviously, it starts with the weakest section, above the enlargement of the pillar. It can be seen that the model with larger blocks is more vulnerable to the seismic motion, probably due to the slenderness of the blocks. The most slender blocks start falling first, followed by the others. The second model behaves more similar to the initial one (Fig. 2a). They have the same ratio between block height and width and are more stable, although the permanent relative top displacement (at the end of the run) is in the order of 80 cm. It should be mentioned that the normal stress peaks in the section above the pillar enlargement dropped to a value of about 14 MPa, which is below the limestone strength. For this model, two runs were performed with the initial record multiplied by factors of 3 and 4, in order to induce structural failure. The damaged structure at the end of the run is shown in Figure 5, respectively.

The dynamic loading with a pga of 0.3g led to considerable damage in the structure. A second earthquake of a reduced size could have more serious consequences. In order to study this effect, the same seismic input, unmagnified (pga = 0.2g), was applied to the system on top of the first one. Only the model with larger blocks was used. This second earthquake motion caused the collapse of the already damaged pillar. The evolution of failure process is presented in Figure 6. The initiation of the failure mechanism is clearly visible in the slender mid-height of the structure.

Another important aspect of the behavior of block masonry structures is the block splitting or crushing of block corners due to very high compressive stresses. Usually, the mortar joints are destroyed first. The joints of the Águas Livres aqueduct are very thin (about 4 cm), so it is possible that some cracks exist in the limestone blocks, especially in the slender part of the pillars.

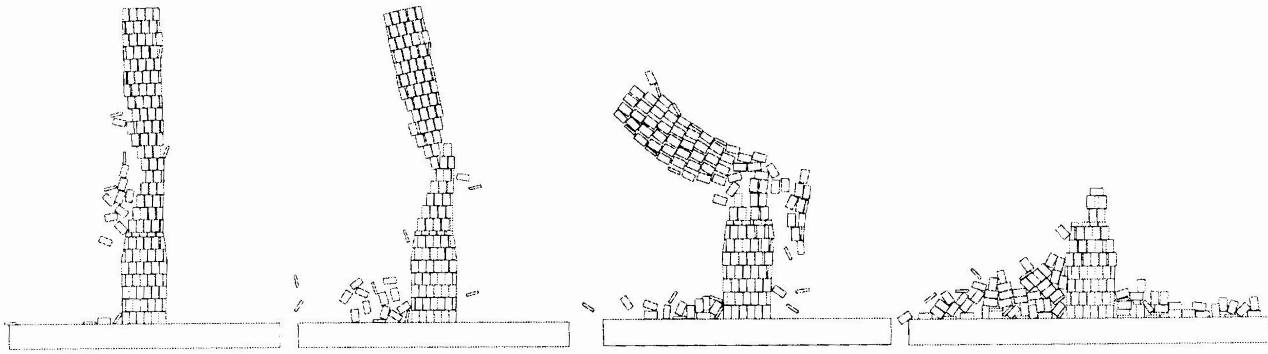
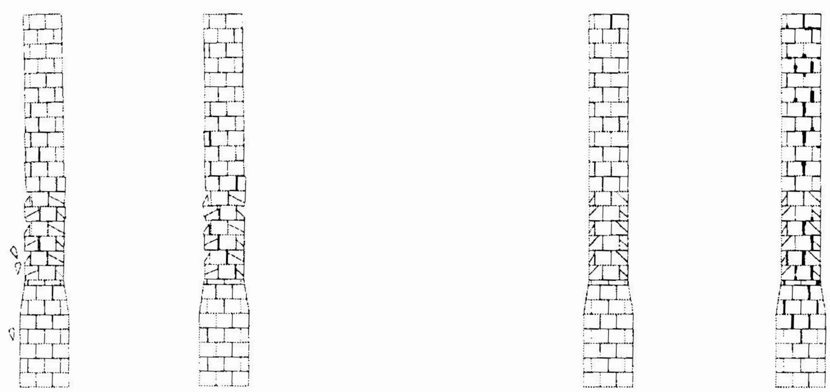


Figure 6. Different stages of the failure process

This phenomenon could lead to significant damage or even the collapse of the structure in the case of a severe seismic event. Two different models were developed in order to take into account this unfavorable condition (displayed in Figs. 2d and 2e). Some initial cracks were considered in the weakest part of the analyzed pillar, with mechanical properties simulating limestone joints.

A study of sensitivity to the input motion was first undertaken. The model shown in Figure 2d was subject to the Romanian record scaled to a pga of 0.3g and to the artificially generated accelerogram simulating an event similar to the 1755 Lisbon earthquake, also scaled to a pga of 0.3g, respectively. The results are presented in Figure 7. As it can be seen, the damage is much larger in the case of the first record. For this earthquake two different stages of the damage process are illustrated in Figure 7a. The cracks in the stones yield to large shear displacements, opening of the joints and detachment of some blocks. Although the pillar did not fall, it can be considered as structural failure. In what regards the second record, it was thought that its longer strong motion could cause the collapse of the structure. On the contrary, the structure seems to be more stable in this case and has less damage than in the case of the Romanian record. The damaged structure (first frame in Fig. 7b) and the separation and sliding in the joints (darker lines in figure 7b, second frame) at the end of the run are presented in the figure.



a) Romanian earthquake (pga = 0.3g) Portuguese earthquake (type II) (pga = 0.3g)

Figure 7. Structural sensitivity to the input motion

The influence of a second earthquake simulating an after shock was also investigated. For this, the model shown in figure 2e was employed. The 1977 Romanian record scaled to a pga of 0.3g was applied as a main shock and then the same earthquake reduced to a pga of 0.2g was used as a following shock. It was found that the structure with initial cracks in the blocks is more vulnerable than the one with intact blocks. Some images of the failure process during the main seismic shock are displayed in Figure 8. This motion rendered the structure with unrecoverable structural damage.

The second earthquake caused the complete failure of the structure. The evolution of collapse is depicted in Figure 9. It is interesting to notice the rigid body like movement of the upper thicker part of the pillar. The blocks move together with little separation or sliding. This phenomenon was already observed in real cases of very tall and slender structures acted by strong seismic motions.

It should be added that for the last two models analyzed here, the very high normal stress peaks obtained in the previous study dropped to values in the range of 12 MPa.

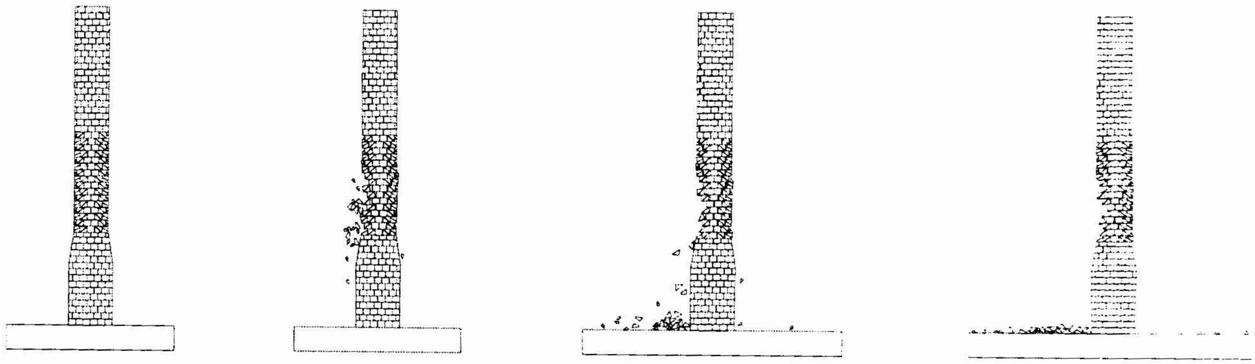


Figure 8. Stages of the damage process for the main seismic shock

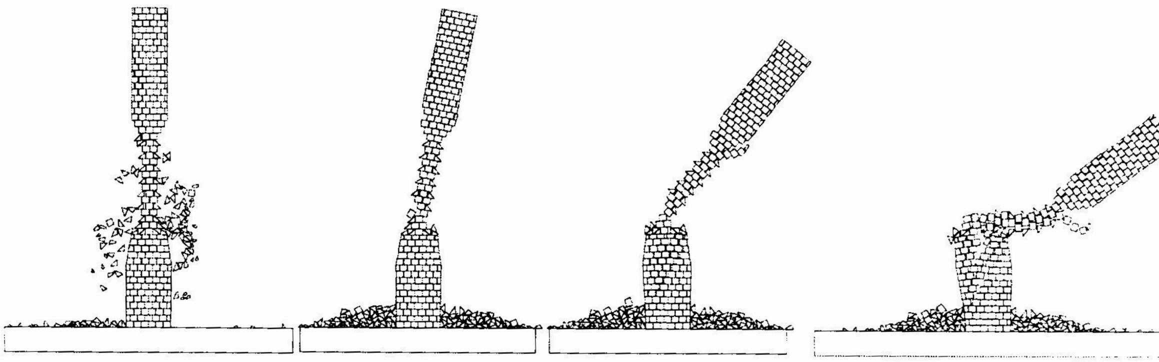


Figure 9. Stages of the collapse process for the second seismic shock

CONCLUSIONS

The suitability of the discrete element method for the seismic analysis of block masonry strutures is confirmed by this study. The numerical analyses performed indicate that type of block masonry pillar under study is able to withstand a significant seismic action, if the blocks remain intact. A possible mechanism of block failure was investigated, considering the existence of incipient fractures within some blocks, which led to structural collapse under a lower seismic input. The vulnerability of a damaged structure to a second earthquake was also examined, stressing the importance of defining damage indicators to characterize the progressive effects of seismic loading, and to allow a comparative evaluation of the results of numerical simulations.

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