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USE OF BASE ISOLATION IN HISTORICAL AND TRADITIONAL STRUCTURES

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ABSTRACT

It is the objective of this study, the analysis of the special benefits of base isolation for low ductile systems such as stone and brick masonry structures of historical or traditional constructions. These types of structures are usually highly damaged during strong earthquakes and thus the applicability of innovative solutions like base isolation systems should be analysed to examine their efficiency.

The comparison between the seismic load levels that can be resisted by this type of structures with and without the base isolation systems may give very important information regarding the decrease of their seismic vulnerability.

The reaching of structural collapse is examined for increasing levels of the seismic action, both for the isolated and non-isolated structures. A comparison is made between the seismic levels inducing collapse in the two types of structures.

Based on the available results, it can be concluded that this type of structural systems, specially those having large masses, have large benefits from base isolation. Specially important for this type of structures is also the fact that not only the internal forces significantly decrease with the isolation but also that the deformations are substantially reduced.

KEYWORDS

Base isolation; historical structure; stone masonry; fiber model; non-resistance in tension.

INTRODUCTION

It is intended with this study to show the special benefits of base isolation for low ductile systems such as stone and brick masonry structures of historical or traditional constructions, which, due to their stiff and brittle structural components, are usually highly damaged during strong earthquakes.

Past cases of collapse, like the Carmo church in Lisbon, destroyed during the 1755 earthquake, or more recently the Friuli earthquake induced collapses of several churches such as the S. Rocco - Forgaria, S. Lucia and S. Caterina - Venzone, S. Lorenzo - Monte di Buia, have shown that the main reasons for collapse are:

the lack of ductility and resistance in tension of the stone masonry and the high displacements. In these and other similar constructions, typical failure mechanisms include failure of the stone masonry walls, collapse at the base of columns and at the base and keystone of arches.

To diminish the potential for earthquake failure, seismic isolation can be used, to keep structural deformations within reasonable low values. This level of earthquake resistance cannot be provided by earlier aseismic techniques, which in any case are also difficult to apply due to architectural and aesthetical reasons. The valuable interiors and facades often do not allow application of traditional strengthening procedures.

Practical isolation systems must trade off between the extent of force reduction and acceptable relative displacements across the isolation system during the earthquake motion. As the isolator flexibility increases, movements of the structure relative to the ground may become a problem under other vibration loads applied above the level of the isolation system, particularly wind loads. Acceptable displacements in conjunction with a large degree of force reduction can be obtained by providing damping, as well as flexibility in the isolator.

Although there are few isolated structures all over the world, it seems that the application of this technique to increase the seismic safety of old structures with high cultural-historic value has good future prospects. This is so because these structures, due to their rigidity and geometry, often have frequencies corresponding to high values of the earthquake spectra, and because other possible strengthening techniques do not seem to be applicable. The increasing acceptance of seismic isolation as a technique is shown by the number of retrofitted seismic isolation systems which have been installed (Skinner *et al.*, 1993). Examples are the completed isolation retrofit of the Salt Lake City and County Building in Utah and the Mackay School of Mines (United States), the Parliament House in Wellington (New Zealand), the Melfi bell tower, the San Pietro church in Frigento and the Fontana Magiore in Perugia (Italy).

The fact that earthquake motion dominated by short-period content favours the adoption of seismic isolation does not rule out its use where long period type motions are expected. The large displacement demands imposed by low frequency components may be more readily accommodated by isolated structures with provision for large isolator displacements than by conventional structures.

STUDY METHODOLOGY

A methodology to study the structures under consideration was developed, based on a modified version of the non-linear structural analysis program DRAIN-2D (Kanaan and Powel, 1975), which can model the base isolation systems, infill panels of brick masonry which exhibit non-linear behaviour and also structural elements characterising structural cross sections which do not resist in tension.

The behaviour of structures composed of prismatic elements made out of stone, such as columns and arches, is modelled by means of an element representing the cross section of the stones and exhibiting limited resistance in tension. This new element was developed and implemented in DRAIN-2D (Bento and Azevedo, 1995) and analyses the non-linear dynamic behaviour of stone masonry structures taking into account different important characteristics observed in this type of constructions. The element consists of a linear elastic beam element with non-linear rotational springs or hinges at each end, which may account for degrading flexural stiffness and strength, softening and the effect of the axial force variation.

The hinge behaviour under cyclic loading is analysed by means of a fiber model, where the section is considered to be composed of a number of element fibers describing the material which has a limited resistance in tension. The fiber model allows for the use of complex material models under cyclic loading and reflects the section behaviour rather accurately. It uses a stress-strain relationship to represent the non-linear material behaviour of the stone masonry mortar as shown in figure 1. In figure 2 is shown a typical moment-curvature relation for a given section of a column made out of stone masonry when subjected to a constant axial load and an horizontal cyclic loading.

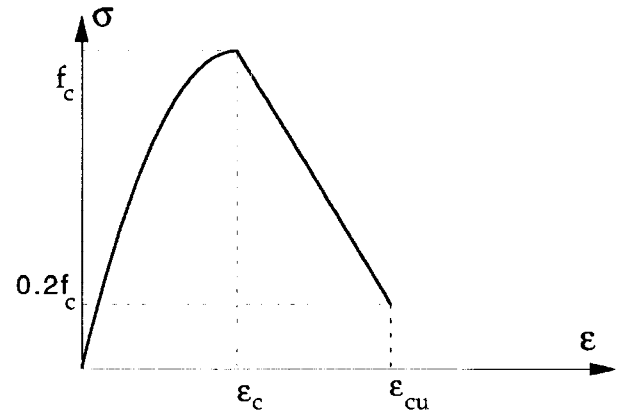

Fig. 1 - Stress-strain relationship for the mortar between stones

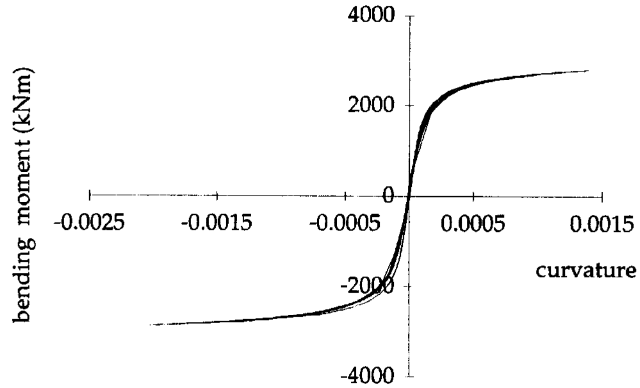

Fig. 2 - Typical moment-curvature relationship for an element subjected to a cyclic loading

It is interesting to notice that the loops in the moment-curvature diagram are quite realistic, taking into account the behaviour of this kind of materials which do not resist in tension. When the joint between stones is not open, which means that all the cross-section of the element is in compression, one can notice the linear elastic behaviour of the structure with a very high stiffness. Once the joint opens, the stiffness of the structure is drastically reduced due to the fact that just a small part of the cross-section is still in compression and the other part is not resisting in tension. When the reversed loading is applied, the joint gradually closes and the structure will regain step by step the initial stiffness. If the axial load remains unchanged, the unloading stiffness is the same as the loading one.

To simulate the behaviour of the masonry walls, the original DRAIN-2D non-linear infill panel element is used. The element is assumed to have shear stiffness only. It is modelled by a rectangular finite element with two degrees of freedom for each node. The model can take into account the brittle failure of the masonry wall (which is unavoidable in case of historical structures). Two different failure codes can be used, governing the type of behaviour after failure according to whether the strength and stiffness are reduced to zero after failure or a post yielding stiffness is assumed, simulating the friction resistance along the two sides of a fractured surface. In the present studies the last possibility was assumed. If sudden failure takes place, the force being resisted by the element immediately prior to failure will suddenly be transferred to the remaining structure, essentially as a shock loading.

A bilinear model was used to represent the behaviour of the base isolation system. Non-linear isolators provide hysteretic energy dissipation, either through sliding friction systems or through the plastic deformation of metals such as steel or lead. It is usually possible to achieve greater and more reliable energy dissipation with non-linear hysteretic isolators than with linear isolators and viscous damping. The non-linearity also allows the structure to be stiff in small-amplitude motions, while in larger-amplitude motions, such as those resulting from strong earthquake ground motions, the isolator softens to give the large base flexibility required for effective isolation.

The bilinear isolator model has an initial stiffness k_1 and a post-yielding stiffness k_2. The most important parameters characterising the base isolation system are: 1) the isolator frequency (α) that represents the equivalent frequency of a rigid body system with the weight of the structural system (W) and the stiffness of the isolator after yielding (k_2), and 2) the yield ratio $\eta=Q_y/W$, relating the yield force Q_y of the isolator to the weight W of the structure.

A parametric study was carried out and the most adequate base isolation solutions for each type of construction are proposed and their applicability analysed.

The seismic action is modelled by means of artificial accelerograms and thus allowing for a semi-probabilistic representation of the seismic input. Four different accelerograms were generated based on a power spectral density function compatible with the response spectrum presented in the Romanian code for earthquake resistant design of structures corresponding to a high magnitude earthquake on a hard soil.

Four different types of typical structures were analysed. The first (1), a column or pedestal of a statue, the second (2) an arch, the third (3) a church arch with masonry infills and the fourth (4) the global resisting structure of a church (Branco *et al.*, 1990) including both the arch with infill masonry and the exterior walls linked at the roof level for displacement compatibility. Figure 3 shows the geometry of the four systems.

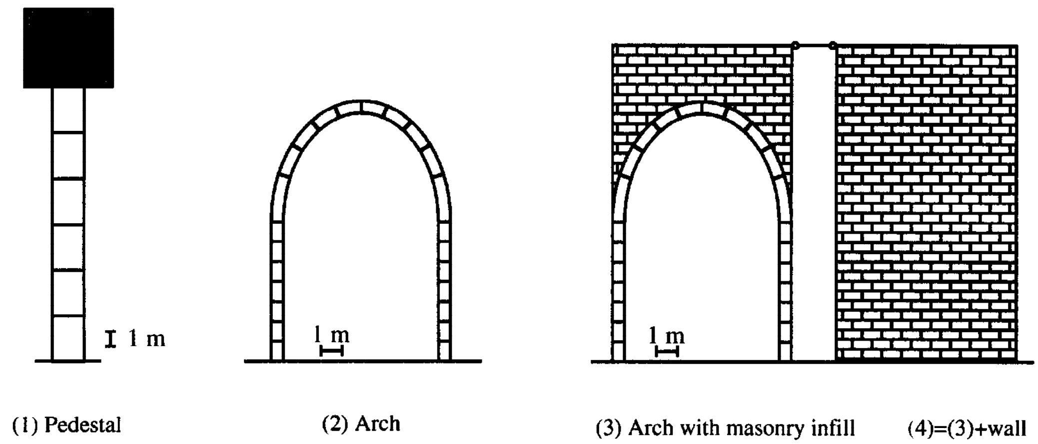

Fig. 3 - Structural systems under analysis

The natural frequencies of the analysed structures are respectively 1.5 Hz, 0.64 Hz, 1.7 Hz and 2.0 Hz, for the pedestal, arch alone, arch with infill masonry and arch together with masonry wall.

RESULTS

The analysis of the structural response was based on the use of the already mentioned program and the results were examined in terms of various important parameters such as the maximum curvature, the global and relative displacements for the structure, the maximum displacement in the isolator and the shear force at the base of the structure.

The reaching of structural collapse was examined for increasing levels of the seismic action, both for the isolated and non-isolated structures. Collapse can be considered to occur for different reasons: non equilibrium, excessive local curvature or excessive displacements. Non equilibrium is reached when it is no longer possible to equilibrate the external forces with the available resisting stresses in the fibers.

The influence of the peak ground acceleration (PGA) level on the curvature and on the maximum displacement in the structure, both for non-isolated and isolated structures was studied. Optimal solutions for base isolation were chosen based on the analysis which is shown later on.

It can be seen, both in figures 4 (for the pedestal) and 5 (for the arch with infill masonry) that, for the same peak ground acceleration value, there is a strong decrease in curvature and maximum displacement in the structure when base isolation is used. This behaviour is more evident for higher peak ground acceleration levels, showing the significant increase of the isolator efficiency for strong earthquake motions. One can notice that for any given level of curvature or maximum displacement, the base isolated structure can withstand an earthquake two to three times stronger than the one withstood by the non-isolated structure. This is specially relevant for the arch structures (fig. 5) where the introduction of base isolation seems to guarantee safety for significantly higher acceleration levels.

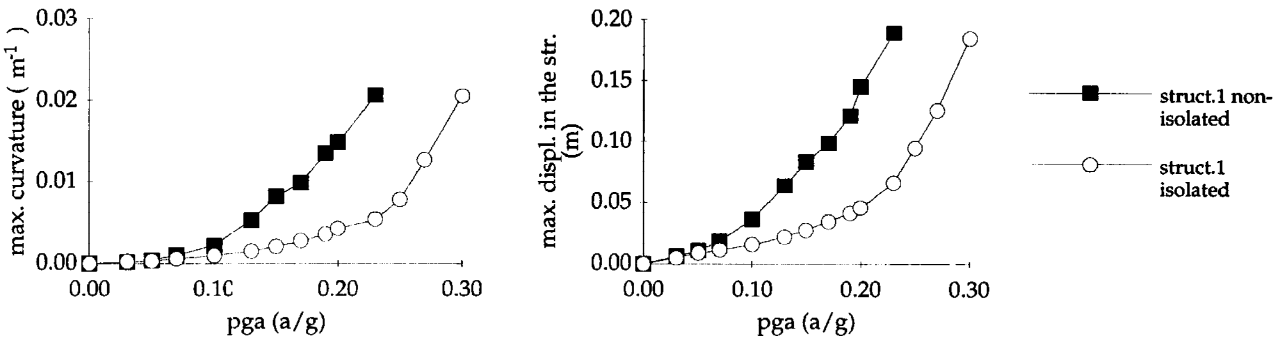

Fig. 4 - Pedestal. Increase in maximum curvature and displacement for increasing PGA

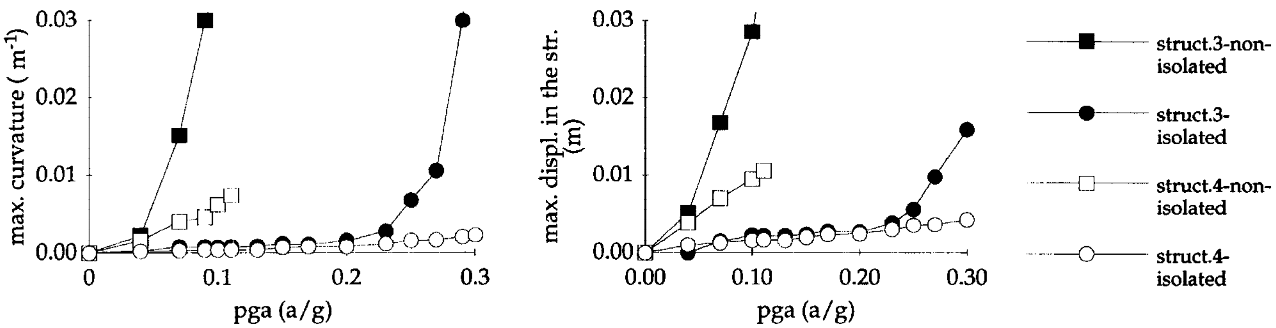

Fig. 5 - Arch with masonry wall. Increase in maximum curvature and displacement for increasing PGA

As the program used for this analysis models the yielding that takes place in the plastic hinges at each element ends, the influence of the plastic hinge length on the maximum curvature and displacement in the structure was studied. Fig. 6 illustrates this influence for the non-isolated structure (pedestal for statue). The assumed length of the element (at the base of the structure) was around 100 cm. One can observe the fact that for a given PGA level, the curvature does not vary significantly with the variation of the plastic hinge length. On what concerns the maximum displacement in the structure, it can be noticed that the plastic hinge length has already some influence, specially for the higher PGA levels. It is thus suggested that the plastic hinge length to be used is related to the equivalent thickness of the mortar layers.

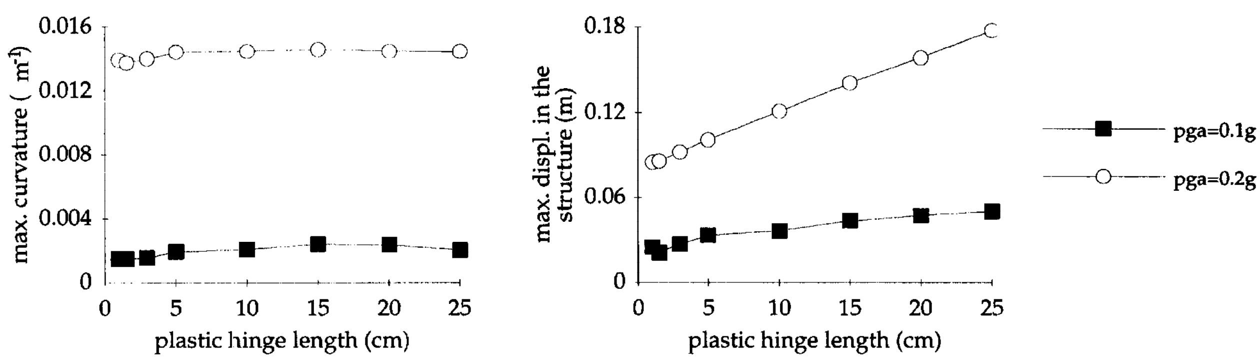

Fig. 6 - Plastic hinge length influence on the maximum curvatures and displacements.

The successful seismic isolation of a particular structure is strongly dependent on the appropriate choice of the isolator, in order to get an adequate force reduction and horizontal flexibility. The seismic response of the base isolated structure is mainly governed by two parameters: the yield level of the isolator (η) and the isolator frequency (α).

In order to get an optimal solution for the isolation system, structure 2 (arch) was base isolated using different types of isolators, having equivalent frequencies α of 0.2 and 0.4 Hz and yield levels η in a range between 5% and 30%. In all cases the initial stiffness of the base isolator is considered six times higher than the post yielding stiffness ($k_1=6k_2$).

Four different accelerograms corresponding to the same power spectrum were used (a1, a2, a3, a4) with a PGA value of 0.23g. The structural response was analysed in terms of the average response values for the four time histories. The results are displayed in figures 7 to 10.

As can be seen in figure 7, the dispersion of the results is quite important, showing that the structural response is very sensitive to the ground motion.

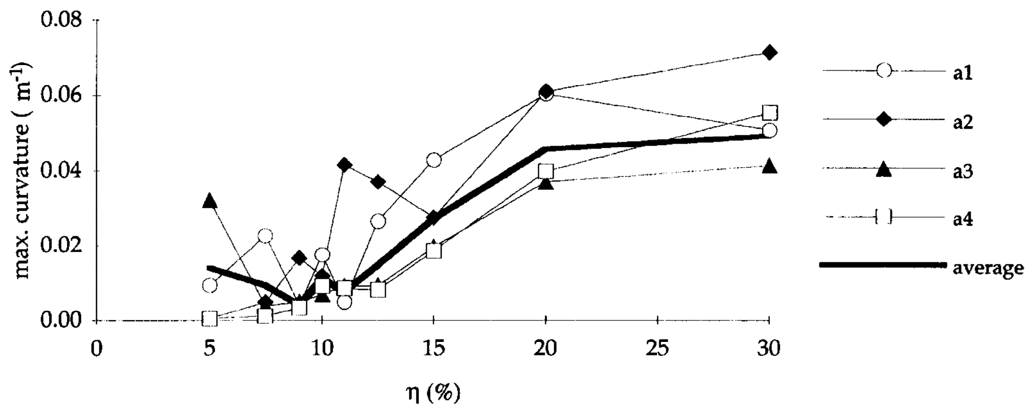

Fig. 7 - Influence of ground motion and η on the maximum curvature (α= 0.2)

Although the response values are scattered, in general terms it can be said that for each α value, there is an optimal η value which minimises the response for a given motion as can be observed in fig. 8 for η values $\approx$10%.

In figure 8, the relative displacement is the displacement between the top of the structure and the top of the isolator, while, in figure 9, the isolator displacement is the displacement between the base of the structure and the ground, and the top displacement in the structure is the displacement between the top of the structure and the ground.

For the base isolated structure, the maximum curvature, relative displacement in the structure (fig. 8) and base shear force (fig.10) are very much reduced, specially for the lower isolator frequency (0.2 Hz) and low yield levels of the isolator, relatively to the values obtained for the non-isolated structure. On the other hand, an isolator which is effective in reducing seismic demands on a structure induces relatively large displacements (fig.9). The total structural displacements are then a little larger than the displacements of the supporting isolator, since they are moderately increased by structural deformation. The values obtained for the vicinity of collapse for the non-isolated structure are represented, in figures 8 to 10, by an horizontal dashed line.

From figures 8 and 9 it can be concluded that the maximum curvature in the base isolated structure is considerably reduced with a quite little cost in terms of the maximum top displacement in the structure (fig.9). This is more remarkable in the case of isolators with η values in the range of 5 to 15 %. It can also be seen that for an α value of 0.2 Hz and an η value in the range of 7.5 to 11 %, the maximum relative displacement is very low which means that the structure above the isolator has an almost rigid body behaviour.

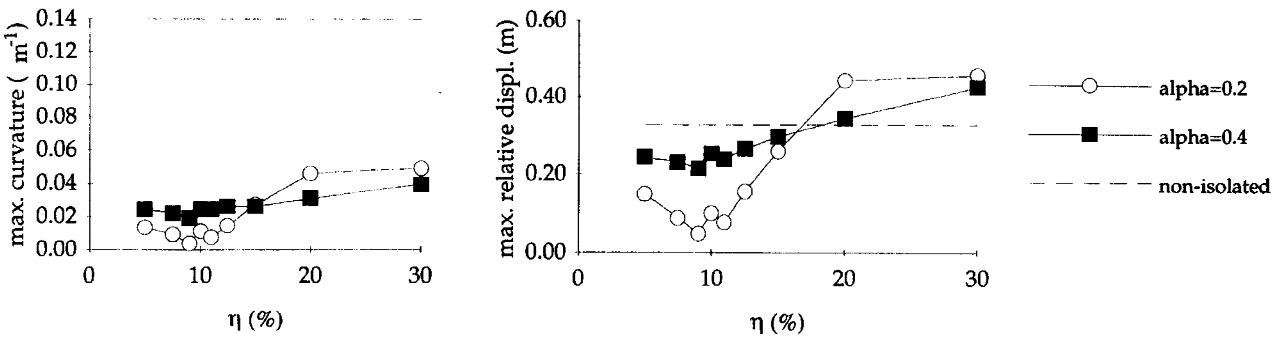

Fig. 8 - Influence of α and η on the average maximum curvature and relative displacement

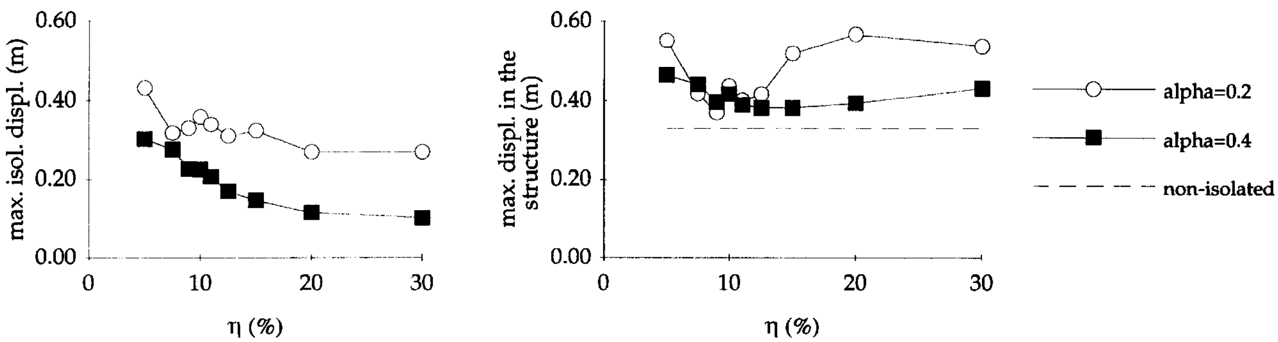

Fig. 9 - Influence of α and η on the average maximum displacement in the isolator and in the structure

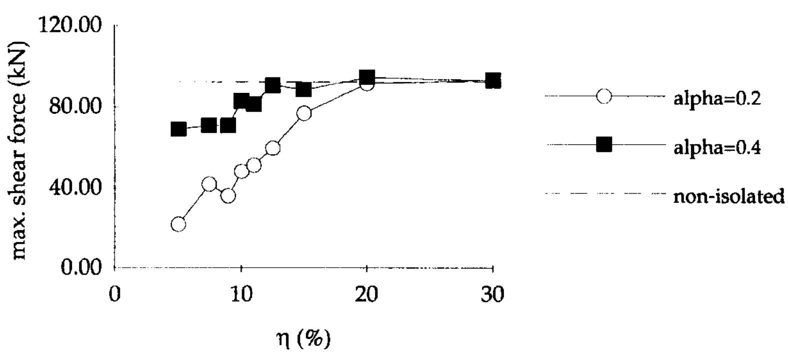

Fig. 10 - Influence of α and η on the average maximum shear force

The failure mechanisms obtained for the analysed cases are very similar to observations made in real cases when an earthquake has damaged a historical building. The typical failure patterns in both isolated and non-isolated situations, correspond to failure at the base of the structure for pedestal or arch alone and at the base of the arch, or top of the columns for the arch with infill masonry.

CONCLUSIONS

The obtained results show that the models used to study the seismic behaviour of the analysed structures can represent rather adequately some of their most important behaviour features such as the fact that the joints between stones do not resist in tension and also that there can be non-linear behaviour of the materials constituting those joints, being able to simulate the behaviour of the structures up to collapse due to non equilibrium. There are anyway some limitations in the utilised model. One of the limitations is the behaviour in shear, and the other, the fact that the program does not take into consideration the geometrically non-linear behaviour of the structures. This last limitation can be not so relevant if some criteria for maximum displacement are set. In what regards the shear behaviour, the limitation can be partly overcome if it is verified if the available shear and friction resistance is, at each time, greater than the acting shear force.

From what was presented, it can also be concluded that base isolation may be a good means of improving the safety of structures of old monuments. Specially important for this type of structures is the fact that not only the internal forces significantly decrease with the isolation but also that, if the appropriate solution is chosen, the deformations are substantially reduced. Thus, special care should be taken choosing the most appropriate solution depending on factors such as the type of seismic loading, the local soil conditions and the structural dynamic characteristics.

The structural response of this type of structures is highly influenced by the seismic loading, even when the different considered seismic input corresponds to ground motions with similar frequency content and peak values. This should be taken into consideration by considering an appropriate number of seismic inputs that can represent different possible future ground motions acting the structures under study.

The greatest uncertainty in the major response of most isolated structures is the maximum seismic displacement which will be demanded of the isolators. As a result, isolators should usually be given considerable reserve capacity for displacements beyond even extreme design values. Different kinds of isolators such as sliding and elastomeric bearings can be combined in order to accommodate high displacements in the isolation system. Such combinations confer the maximum benefit of each component to the system as a whole. Bumpers can also be used to limit base-level displacements if needed.

It is also necessary to make provisions for clearance around the structure. The seismic gap must remain secure. This fact can considerably diminish the possibility of wider application of this type of seismic protection.

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