On the Structural Analysis and Seismic Protection of Historical Masonry Structures

Panagiotis G. Asteris*

Department of Civil & Construction Engineering, School of Pedagogical & Technological Education, Athens, Greece

Abstract: In this paper, a general methodology for the seismic protection of historical structures and monuments is presented. The proposed methodology is applied to one historical and monumental structure, in Cyprus, within the frame of the European Project for the Conservation of Historical Mediterranean Sites by Innovative Seismic-Protection Techniques. According to the proposed method, the structure under consideration was analysed with and without the implementation of vibration control devices. The entire vulnerability analysis leads to the development of fragility curves, which determine the possibility of a building to be damaged beyond a specified damage level for strong ground motions. These results are quite important during the analysis and redesign procedure for a historical structure since it gives the opportunity to investigate several different seismic scenarios with different repair/strengthening decisions.

INTRODUCTION

The majority of the main structural systems for historical structures or monuments are masonry elements, composed of stone, bricks and mortar. For all types of old historical masonry structures (including monuments) erected in seismic zones of high seismicity, earthquake is always their number one "enemy" due to their very bad response to earthquakes.

The responsibility of protecting a historical structure falls mainly on the shoulders of the engineer. A successful intervention on a monument requires a good comprehension of its structural behaviour under static and dynamic (earthquake) loading. For an engineer, taking part to the restoration process of a historical structure, through the analysis of its structural system, means mainly to face the demanding task of equipping the historical structure with the capability to withstand future actions with the minimum possible amount of damage, while bearing in mind the characteristics and values which make this structure unique and worthy of special attention. This has to be carried out within the conditions imposed by current regulations and scientific Charters (e.g. the Athens Charter 1931 [1], the Venice Charter 1964 [2], etc.), which make the process of analysis more complicated.

The analysis of ancient monuments poses important challenges because of the complexity of their geometry, the variability of the properties of traditional materials, the different building techniques, the lack of knowledge on the existing damage from the actions, which affect the monuments throughout their lifetime and the lack of codes. Nevertheless, rational methods of structural analysis, based on modern engineering principles have been developed in the last two decades [3-18].

*Address correspondence to this author at the Department of Civil & Construction Engineering, School of Pedagogical & Technological Education, N. Heraklion, GR 141 21, Athens, Greece; Tel (+30) 2102853554;
E-mail: pasteris@otenet.gr

The estimation of the seismic vulnerability of a historical monument is a multi-phased (and multifaceted) process that ranges from the description of earthquake sources, to the characterization of structural response, and to the description of measures for seismic protection.

The basic tool for a reliable vulnerability analysis is the quantitative estimation of the damage level of the monument's structural system. To estimate and describe the damage of this system (usually masonry elements) an analytical cubic polynomial method (failure criterion) has been proposed by the author. In addition, for the implementation of the proposed failure criterion, a specific computer program has been developed. According to this program, which uses as Input Data the Finite Element Analysis results as well as the mechanical characteristics of masonry material, coloured graphic images of the failure for each individual element within the structure are produced.

Proper probabilistic analysis of the above results leads to the development of fragility curves. Based on these curves the probability of a building to be damaged beyond a specified damage stage for various level of ground shocking can be determined. This information is quite important during the analysis and redesign procedure since it gives the opportunity to investigate several different repair/strengthening scenarios.

STRUCTURAL ANALYSIS AND REPAIR METHODOLOGY

Modeling of a historical masonry structure is a difficult task, since masonry does not easily conform to the hypotheses usually assumed for other materials (isotropy, elastic behaviour, homogeneity); furthermore, appropriate constitutive laws for the materials are still not well developed. The continual modifications, which have taken place during the building's history, produce several uncertainties in the model definition (geometry, materials, connection).

Based on the Finite Elements Method (FEM), a basic methodology for the earthquake resistant design and reha-

1874-8368/08 2008 Bentham Open

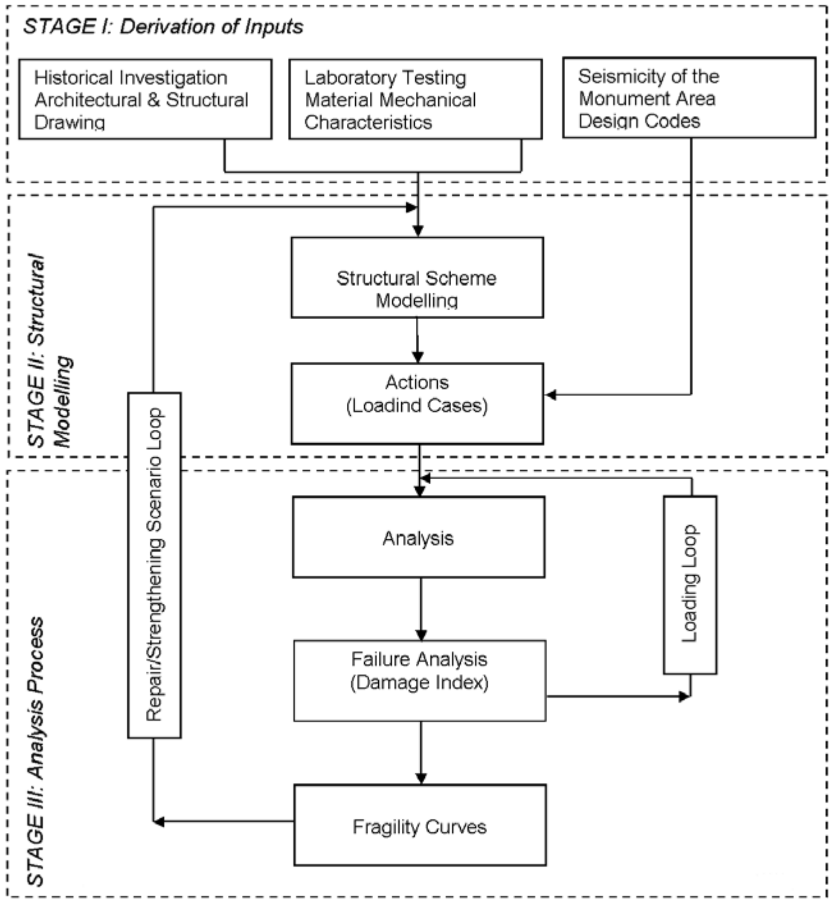


Fig. (1). Flow chart of the general methodology followed in this study.

bilitation of damaged masonry historical structures has been developed and presented. The main steps of the method are schematically presented in Fig. (**1**), and are briefly described as follows:

Step 1: Preparation of detailed architectural and structural drawings, describing the current condition of the structure.

Step 2: Determination of Material Characteristics. Mechanical characteristics of the materials composing the structure are the basic input data needed for the analysis. In particular, the compressive and tensile strength of the materials, their modulus of elasticity and Poisson's ratio, are of primary importance.

Step 3: Structural Simulation. A 3-D finite element model is the most suitable one for the analysis. For higher model reliability, specific simulation parameters, such as the rotation capacity of the wooden floor connection with the masonry wall, the rigidity degree of connections between intersected walls, the influence of spandrel beams, etc., have always to be taken into account.

Step 4: Perform Actions. Loadings foreseen by the codes for the relevant use of the structure have to be considered. An appropriate seismic loading has also to be taken into account, especially for structures built in seismic areas.

Step 5: Analysis. In the past decades several attempts have been done to assume models used for other materials, but the results were very poor. Elastic models can give an indication on the mechanical behaviour of the structure but they cannot follow the behaviour beyond the elastic range [13]. Nonlinear models can be very heavy to handle and costly.Using the data of Steps 1, 2, 3 & 4, FEM elastic (or elastoplastic) analysis is performed by SAP2000 [19] and stresses (normal & shear), and displacements at the nodes of the mesh elements are calculated. This process is iterated for the loadings considered in step 4.

Step 6: Determination of Seismic Vulnerability. Taking into consideration conclusions made in Step 2, concerning material characteristics, a failure criterion is established and used for the definition of the failed regions of the structure. These failure results are used as input data for the development of fragility curves. Based on these curves the possibility of a structure to be damaged beyond a specified level (heavy, moderate, insignificant damage) for various levels of ground shocking is determined. This information is quite important during the analysis and redesign procedure for a historical structure since it gives the opportunity to investigate several different scenarios with different repair/strengthening decisions.

Step 7: Making Repairing and/or Strengthening Decisions. Decisions have to be taken concerning repair and/or strengthening of the existing structure. The methods to be used, the extent of interventions, the type of the materials, etc., are mainly related to the results of Step 6. It has to be noted, however, that structural analysis is not always sufficient to give reliable judgements since, sometimes, there are too many uncertainties on material characteristics, inner cracks and discontinuities, permanent deformations and accumulation of stresses in plastic zones, which may impair the results of calculations [20]. For this reason, qualitative (and subjective) criteria based on the observation of the structure and the historical knowledge of the technologies, phenomena, events, etc. must also be considered before taking any repairing and/or strengthening decisions. This process is iterated (steps 3 to 7) for each repairing scenario considered.

The main steps of the process will be presented in more detailed in the following sections.

A BRIEF REVIEW ON THE STRUCTURAL CONTROL TECHNIQUES

In recent years, considerable attention has been paid to research and development of structural control devices, with particular emphasis on seismic response of structures. Full-scale implementation of active control systems have been accomplished in several structures, mainly in Japan; however, cost effectiveness and reliability considerations have limited their wide spread acceptance. Because of their mechanical simplicity, low power requirements, and large, controllable force capacity, semi-active systems provide an attractive alternative to active and hybrid control systems for structural vibration reduction.

Supplemental passive, active, hybrid, and semi-active damping strategies offer attractive means to protect structures against natural hazards. Following is a brief discussion of structural control techniques for the seismic protection of structures.

Active Control

An active control system is one in which an external source powers control actuator(s) that apply forces to the structure in a prescribed manner. These forces can be used to either add or dissipate energy in the structure. In an active feedback control system, the signals sent to the control actuators are a function of the response of the system measured with physical sensors. Active control makes use of a wide variety of actuators, including active mass dampers, hybrid mass dampers, tendon controls, which may employ hydraulic, pneumatic, electromagnetic, or motor driven ball-screw actuation. An essential feature of active control systems is that external power is used to effect the control action. This makes such systems vulnerable to power failure, which is always a possibility during a strong earthquake or a strong wind.

Active control acts as a means of extra protection for structures that are at high risk for seismic activity. Passive control devices are also used to protect some existing structures, or buildings in areas of low seismic activity. The overall idea of active control is a revolutionary one. It has the capability to elevate structural concepts from a static and passive level to a dynamic and adaptable level. However, active control may not be the most cost affective measure in protecting buildings that are in areas of low risk for earthquakes

Passive Control

Passive control systems, including base isolation systems, viscoelastic dampers, and tuned mass dampers, are well understood and are widely accepted by the engineering community as means for mitigating the effects of dynamic loading on structures [21-23]. A passive control system does not require an external power source. Passive control devices impart forces that are developed in response to the motion of the structure. Initial design may use a tapered distribution of mass and stiffness, or use techniques of base isolation, where the lowest floor is deliberately made very flexible, thereby reducing the transmission of forces into the upper stories. The passive controlled devices have four advantages (i) it is usually relatively inexpensive; (ii) it consumes no external energy; (iii) it is inherently stable; and (iv) it works even during a major earthquake or a strong wind. However, these passive-device methods are unable to adapt to structural changes and to varying usage patterns and loading conditions [24].

It is worth noting here that for the special case of existing buildings such as historical and monumental (tower) fluid viscous damper is a type of passive device that can be used in the seismic retrofit of these structures. Retrofitting an existing building with viscous dampers results in reduction of seismic demand upon the structural elements of the existing building. In a building retrofitted with velocity dependent viscous dampers, the damper force is out of phase with the hysteretic or drift demands in the structural system. In other words, the damper delivers zero force to the lateral system at maximum drifts and delivers maximum force to the lateral system when the structure passes through the gravity state during dynamic response to earthquake input. In summary, the viscous dampers provide the maximum damping effect when the lateral movement of the structural system is at its highest velocity, i.e. in passing through the gravity or initial state.

We present in more detail the case of passive control devices due to the fact that these have been used for the seismic protection of the historical structure studied in this paper. In particular, we present details of the model as well as the simulated behaviour of fluid viscous dampers.

The vibration control devices should be selected taking into account that masonry structures' relatively stiff structures. Thus, large energy dissipation should be activated with small displacements. For this reason, fluid viscous dampers, which are velocity-dependent systems, seem to be the most appropriate compared to other types of dampers.

Fluid viscoelastic behavior can be modeled with advanced models of viscoelasticity [25]. However, fluid viscoelastic devices can be modeled using the Maxwell model of Fig. (**2**) in most instances.

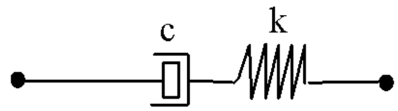


Fig. (2). Maxwell model for fluid viscoelastic energy dissipation devices.

The non-linear force-deformation relationship is given by:

$$f = k \cdot d_k = c \cdot \dot{d}_c^{\,\xi} \tag{1}$$

where k is the spring constant, c is the damping coefficient, ξ is the damping exponent (the damping exponent ξ must be positive; the practical range is between 0.20 and 2.00), $\dot{d}_c$ is the deformation rate across the damper, and d_k is the deformation across the spring. The spring and damping deformations sum to the total internal deformation:

$$d = d_k + d_c \tag{2}$$

If pure damping behaviour is desired, the effect of the spring can be negligible by making it sufficiently stiff.

Hybrid Control

Hybrid-control strategies have been investigated by many researchers to exploit their potential to increase the overall reliability and efficiency of the controlled structure [26-28]. The common usage of the term "hybrid control" implies the combined use of active and passive control systems. A hybrid control may use active control to supplement and improve the performance of a passive control scheme. Alternatively, passive control may be added to an active control scheme to decrease its energy requirements. For example, a structure equipped with distributed viscoelastic damping supplemented with an active mass damper on the top of the structure, or a base isolated structure with actuators actively controlled to enhance performance. It should be noted that the only essential difference between an active and a hybrid control scheme, in many cases, is the amount of external energy used to implement control. Hybrid control schemes alleviate some of the limitations that exist when each system is acting alone, thus leading to an improved solution. A side benefit of hybrid control is that, in the case of a power failure, the passive component of the control still offers some degree of protection, unlike an active control system.

Semi-Active Control Systems

Control strategies based on semi-active devices appear to combine the best features of both passive and active control systems and to offer the greatest likelihood for near-term acceptance of control technology as a viable means of protecting civil engineering structural systems against earthquake and wind loading. The attention received in recent years can be attributed to the fact the semi-active control devices offer the adaptability of active control devices without requiring the associated large power sources. In fact, many can operate on battery power, which is critical during seismic events when the main power source to the structure may fail.

Semi-active control devices do not add mechanical energy to the structural system, but have properties that can be controlled to optimally reduce the responses of the system [29]. Preliminary studies indicate that appropriately implemented semi-active systems perform significantly better than passive devices and have the potential to achieve, or even surpass, the performance of fully active systems, thus allowing for the possibility of effective response reduction during a wide array of dynamic loading conditions [29-31]. Examples of such devices include variable-orifice fluid dampers, controllable friction devices, variable stiffness devices, semi-active impact dampers, adjustable tuned liquid dampers, and controllable fluid dampers (electrorheological fluids and magnetorheological fluids).

Extensive and in-depth state-of-the-art reports on the structural control can be found in Symans and Constantinou [31], Spencer and Nagarajaiah [24].

VULNERABILITY ANALYSIS

The estimation of the seismic vulnerability of a monumental structure is a multi-phased process that ranges from the description of earthquake sources, to the characterization of structural response, and to the description of measures of seismic protection. In this section the basic tools for a reliable vulnerability analysis are presented.

Seismic Hazard Modelling

Early earthquake codes categorized the seismic hazard on the basis of the past experience of the region's inhabitants through hazard maps. These hazard maps were based entirely on historical records from which the maximum-recorded intensity of any point was deduced. The basic disadvantage of these maps was the lack of information concerning recurrence period and thus they fail to identify over where large earthquakes have a long return period. Nowadays, we have the benefit of a) A growing catalogue of recorded earthquakes worldwide, and b) The development of theories on the potential of mapped fault and c) Their accompanying strain rates to generate earthquakes with some regularity.

For the case of historical structures more refined parameters are needed for hazard analysis. Such parameters are the duration, frequency content and predominant periods of motion. As it will be presented below, elastic analysis might possibly be used. For this reason response spectrum may be used for the analysis taking into account the different soil conditions existing in the area under investigation. It is also possible to use a probabilistic response spectrum. For the case of inelastic analysis, acceleration time-histories (recorded or artificial derived motion) may be used for vulnerability analysis.

Structural Model

Historical and existing masonry structures have usually an inadequate resistance to horizontal actions. Furthermore, historical city centres present high vulnerability under horizontal loads and this is mostly due to the absence of adequate connections between the various parts [32-34].

The complex geometry most of the historical masonry structures usually have, generate sizeable variations in stiffness due both to heterogeneities in masonry work and to abrupt changes in cross-section among the component elements. This fact makes complicated to carry out accurate computations through the application of conventional material strength techniques. In such cases, only the use of finite element method (FEM) enables the derivation of credible computational results. Implementation of the FEM allows not only to ascertain the overall functioning of the structure, but also to determine the values of stresses existing in the most "sensitive" parts of the structure [35].

Failure Criterion

The basic step of the proposed methodology is the quantitative damage evaluation of masonry, which is the basic material of historical and monumental structures. The damage is estimated by a cubic polynomial function that is used for composite materials. In this method, the failure surface in the stress space can be described by the equation [14,36]:

$$f(\sigma_\iota) = F_i\sigma_i + F_{ij}\sigma_i\sigma_j + F_{ijk}\sigma_i\sigma_j\sigma_k + \cdots - 1 = 0 \qquad (3)$$

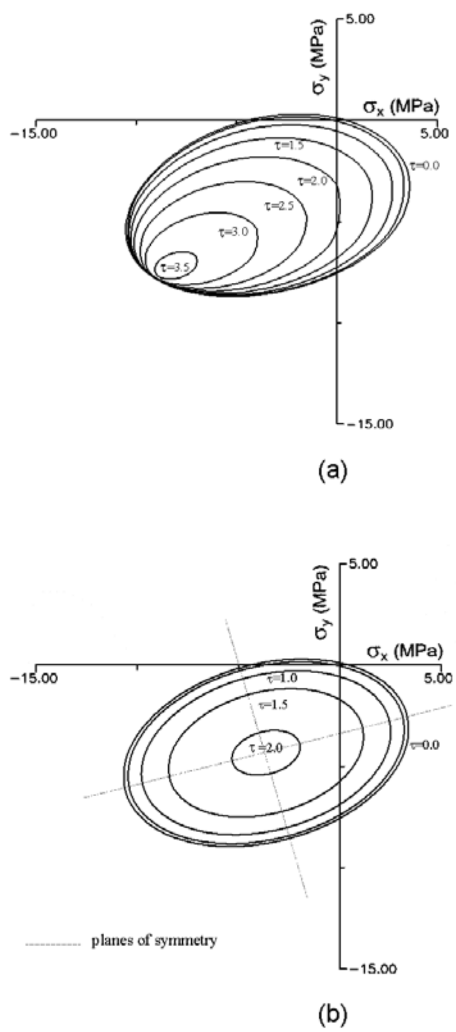


Fig. (3). Failure surface of masonry in normal stress terms: (**a**) General failure criterion; (**b**) Simplified failure criterion.

In this equation σ_ι (ι = 1, 2,..., 6) are the components of stresses and F_i, F_{ij}, F_{ijk} (i, j, k = 1, 2,..., 6) are coefficients to be properly determined.

If one restricts the analysis to a plane stress state, keeping terms up to third order, then Equation (3) reduces to:

$$\left.\begin{aligned} f\left(\sigma_x,\sigma_y,\tau\right) = {} & F_1\sigma_x + F_2\sigma_y + F_{11}\sigma_x^2 + F_{22}\sigma_y^2 + \\ & F_{66}\tau^2 + 2F_{12}\sigma_x\sigma_y + 3F_{112}\sigma_x^2\sigma_y + \\ & 3F_{122}\sigma_x\sigma_y^2 + 3F_{166}\sigma_x\tau^2 + \\ & 3F_{266}\sigma_y\tau^2 - 1 = 0 \end{aligned}\right| \qquad (4)$$

Eliminating all third order terms in Eq. 4, a simplified yield criterion can be derived:

$$\left.\begin{aligned} f\left(\sigma_x,\sigma_y,\tau\right) = {} & F_1\sigma_x + F_2\sigma_y + F_{11}\sigma_x^2 + \\ & F_{22}\sigma_y^2 + F_{66}\tau^2 + 2F_{12}\sigma_x\sigma_y - 1 = 0 \end{aligned}\right| \qquad (5)$$

This simple form of the yield criterion has already been used by other investigators [37,38] to define the failure of brick masonry under biaxial stress. According to Syrmakezis and Asteris [36], the general yield criterion (Fig. **3a**) fit the non-symmetrically dispersed experimental data better than the simplified model (Fig. **3b**).

For the implementation of the proposed failure criterion, a special-purpose computer program, named FAILURE, has been developed [9]. The program uses as Input Data the Finite Element Analysis results (stresses), and the mechanical characteristics of masonry material (strengths), and produces coloured graphic images of the failure for each individual element within the structure.

Damage Index

Damage control in a building is a complex task. There are several response parameters that can be instrumental in determining the level of damage that a particular structure suffers during a ground motion; the most important ones are: deformation, relative velocity, absolute acceleration, plastic energy dissipation and viscous (or hysteretic) damping energy dissipation. Controlling the level of damage in a structure consists primarily in controlling its maximum response.

Damage indices establish analytical relationships between the maximum and/or cumulative response of structural components and the level of damage they exhibit [39]. A performance-based numerical methodology is possible if, through the use of damage indices, limits can be established to the maximum and cumulative response of the structure, as a function of the desired behavior(s) of the building for the different levels of design ground motion. Once the response limits have been established, it is then possible to estimate the mechanical characteristics that need to be supplied to the building so that its response is likely to remain within these limits.

For the case of masonry structures we propose a new damage which employs as response parameter the percentage of the failed area of the structure to the total area of the structure. The proposed damage index, [DI], for a masonry structure can be estimated by:

$$DI = \frac{A_{fail}}{A_{tot}} \times 100 \quad . \qquad (6)$$

where A_{fail} is the failed surface area of the structure and A_{tot} the total surface area of the structure.

Structural Performance Levels

As practiced today, performance-based seismic design is initiated with a discussion between the client and engineer about appropriate performance objectives. The engineer then prepares a design capable of meeting these objectives. Performance objectives are expressed as an acceptable level of damage, typically categorized as one of several performance levels, such as immediate occupancy, life safe or collapse prevention, given that ground shaking of specified severity is experienced.

In the past, the practice of meeting performance-based objectives was rather informal, non-standard, and quite qualitative. Some engineers would characterize performance as life-safe or not; others would assign ratings ranging from poor to good. This qualitative approach to performance prediction was appropriate given the limited capability of seismic-resistant design technology to deliver building designs capable of quantifiable performance.

We consider three structural performance levels: a) heavy damage, b) moderate damage and c) insignificant damage, in a similar way to the Federal Emergency Management Agency (FEMA 273) [40]. The performance levels are defined by the values of DI (as shown in Table **1**). Especially a value of [DI] less than 10% can be interpreted as insignificant damage; from 10% to less than 20%, as moderate damage; and larger or equal than 20% as heavy damage.

Table 1. Proposed Structural Performance Levels for Un-Reinforced Masonry

Overall Damage	Heavy Damage	Moderate Damage	Insignificant Damage
	Extensive cracking: face course and veneer may peel off. Noticeable in-plane and out-of-plane offsets.	Extensive cracking. Noticeable in-plane offsets of masonry and minor out-of-plane offsets.	Minor cracking of veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.
[DI]	≥ 20%	10% ≤ ~ < 20%	< 10%
	Collapse prevention	Life safety	Immediate occupancy

FRAGILITY CURVES

Evaluating seismic fragility information curves for structural systems involves a) information on structural capacity, and b) information on the seismic hazard. Due to the fact that both the aforementioned contributing factors are uncertain to a large extent, the fragility evaluation cannot be carried in a deterministic manner. A probabilistic approach, instead, needs to be utilized in the cases in which the structural response is evaluated and compared against "limit states", that is, limiting values of response quantities correlated to structural damage.

Fragility curves can be obtained from a set of data representing the probability that a specific response variable R (e.g. displacement, drift, acceleration, damage) exceeds predefined limit states $r_{\min}$ for various earthquake hazards on a specific structure or on a family of structures.

Numerical calculation of fragility requires information on the expected response and its variability. This involves the creation of a detailed model of the structure and the application of numerical techniques for probabilistic evaluation of the structural response.

Simplified methodologies for fragility evaluation have been proposed by Kircher *et al.* [41] and incorporated in HAZUS99 [42]. These methodologies assume that the spectral ordinates are log-normally distributed, assuming the variability is represented by the logarithmic standard deviation.

Fragility is evaluated as the total probability of a response R exceeding the allowable response value $r_{\lim}$ (limit-state), for various earthquake intensities I. Mathematically, the fragility is given by the following equation,

$$Fragility = P\left[R \geq r_{\lim} \middle| I\right] = \sum_{j}^{3} P\left[R \geq r_{\lim} \middle| I, C\right] P\left(C = c_j\right) \qquad (7)$$

where $P\left(C = c_j\right)$. is the probability that capacity c_j occurs. In the following example basic steps for the development of the fragility curves, are thoroughly presented.

ILLUSTRATIVE EXAMPLE

The methodological approach presented in this paper is illustrated here in a comprehensive form, by means of a case study of rehabilitation of a historical structure in Cyprus, namely the restoration of the church of Agios Ioannis Prodromos in the village of Askas, Cyprus (Fig. **4**); this structure contains a vast cycle of important and rare Byzantine wall paintings dating from the 15th and 16th centuries.

Description of the Monument

Robert Gowing [43] of the Courtauld Institute Conservation of Wall Painting Department performed an exploration to clarify the various construction phases of the church. According to that work, the earliest building phase appears to consist primarily of a large semi-domed apse and the surrounding east wall. Specific architectural features do not assist dating of this section of the church. The painted decoration provides the only clue with a proposed date, based on stylistic examination, of around the middle of the 16th century.

Gowing reports that extensive rebuilding appears to have occurred around the beginning of the 17th century, involving the complete enlargement of the body of the church. Constructed as a three-aisled basilica plan church, the design accommodated the original apse and east wall, retaining their painted decoration.

The third phase that was noted by Gowing is dated to 1952. This involved the raising of the outer walls to increase the height of the aisles. The exterior changes are visible on the south and east walls with a noticeable change in the construction type. The new roof is noticeably lower in pitch as a result of maintaining the old peak height and the increased outer walls.

Seismicity of Cyprus

Cyprus lies within the second largest earthquake-stricken zone of the earth, but in a relatively less active sector. The level of the seismic activity in the Cyprus region is significantly lower than that in Greece and Turkey. This zone stretches from the Atlantic Ocean across the Mediterranean Basin, through Greece, Turkey, Iran, and India as far as the Pacific Ocean. The energy released by the earthquakes in this zone represents 15% of the universal seismic energy [44,45].

Analysis and Rehabilitation Process

A short description of the actions undertaken, for each of the aforementioned steps of the previous section, is given below:

Preparing the Structural Model: The structure is 5.45m high with 20.25m length and 9m width. The roof of the church is formed by both a new and an older wooden. A bell tower exists but is structurally independent (and, thus, is not considered here). The Finite Element Analysis model of the structure has been developed in three dimensions and consists of 7269 shell, and 475 frame elements (Fig. **4**) using the SAP2000 (2003) libraries. The finite element mesh has been developed in such a way that the ideal massing at the joints of the structure would result to a more realistic modelling of the inertia forces. At critical areas of the structure a more dense mesh of shell elements has been developed, for a more accurate structural analysis. Six degrees of freedom (three translations and three rotations) have been considered for a complete determination of the system deformation in three dimensions.

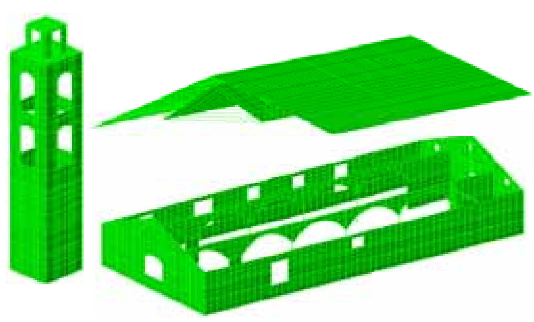


Fig. (4). Structural model of the Agios Ioannis Prodromos church (roof and bearing walls) and the bell-tower.

Actions considered: Earthquake action, along the two main axes of the building, has been considered in both directions (left-right and right-left). Consequently, 5 action combinations have been used according to the Eurocodes aseismic regulations [46], shown in Table **2**. Both vertical and horizontal loads have been applied in the model as nodal ones.

Table 2. Loading Case Combinations

Loading Case	Combinations
1	1.35 G + 1.50 Q
2	1.00 G + 0.30 Q + 1.00 Ex + 0.30 Ey
3	1.00 G + 0.30 Q + 0.30 Ex + 1.00 Ey
4	1.00 G + 0.30 Q - 1.00 Ex - 0.30 Ey
5	1.00 G + 0.30 Q - 0.30 Ex - 1.00 Ey
where: G = dead loads, Q = live loads, and E = earthquake loads.	

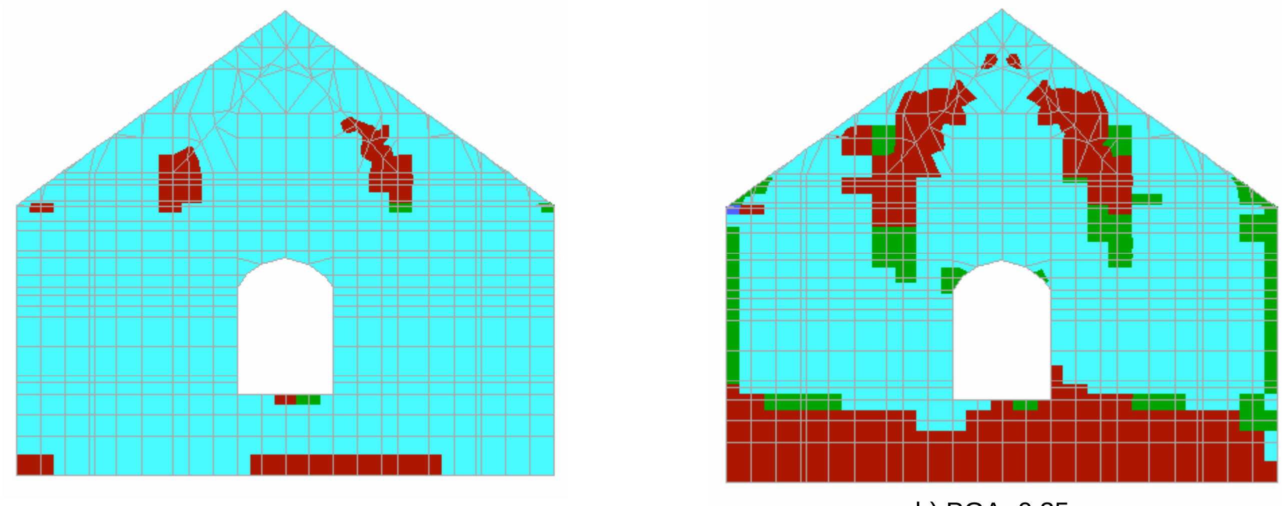
Embedding vibration control devices-viscous damper element: The vibration control devices should be selected taking into account that masonry structures' relatively stiff structures. Thus, large energy dissipation should be activated with small displacements. For this reason, fluid viscous dampers, which are velocity-dependent systems, seem to be the most appropriate compared to other types of dampers.

For the seismic protection of the monument the use of fluid viscous dampers has been selected as the most suitable means. Especially, dampers have been inserted across the two series of arches as well as a set of diagonal dampers just below the roof in horizontal level.

Failure analysis of the structure: The failure analysis of the structure (without and with dampers) was based on the failure criteria explained at a previous paragraph. The analysis concerns a range of Peak Ground Accelerations between 0.10g to 0.40g and masonry tensile strength ranging from 0.05MPa to 0.55MPa. Failure results refer to percentage of the overall failure, as well as to picture, as such of the Figs. (**5**), and (**6**), distinctly the type, extent and position of damage.

Probabilistic analysis - Fragility curves: The results concerning the failure areas of the structure were analysed with probabilistic methods. Especially the Probability Distribution Function and the associated Probability Density Function were estimated for each level of Peak Ground Acceleration applied at the structure by the StatSoft, Inc. [47] Statistical Package. Using these Probability Distribution Functions, the probabilities of damage of the structure for the three structural performance levels (insignificant, moderate and heavy damage) have been determined. The fragility curves for the structure, without and with dampers, are obtained when log-normal curves are fitted to the data obtained in the previous step. Fig. (**7**) shows the fragility curves of the structure without damper. Fig. (**8**) shows the fragility curves when the structure is fitted with dampers.

The effect of the fluid viscous dampers on the response of historical and monumental structure can be depicted from Figs. (**7**) and (**8**). The probability of insignificant damage from a seismic motion with demand represented by PGA=0.15g

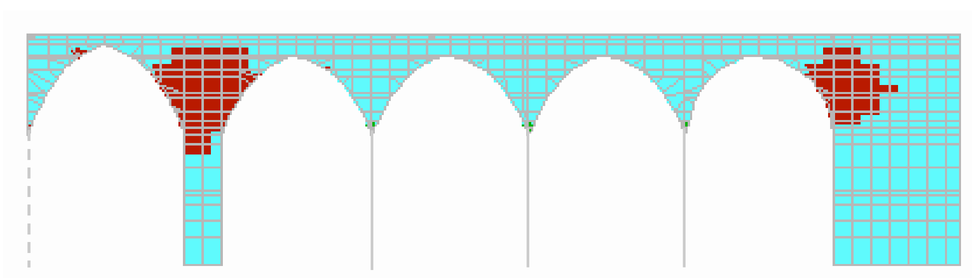


a) PGA=0.10 g b) PGA=0.35 g

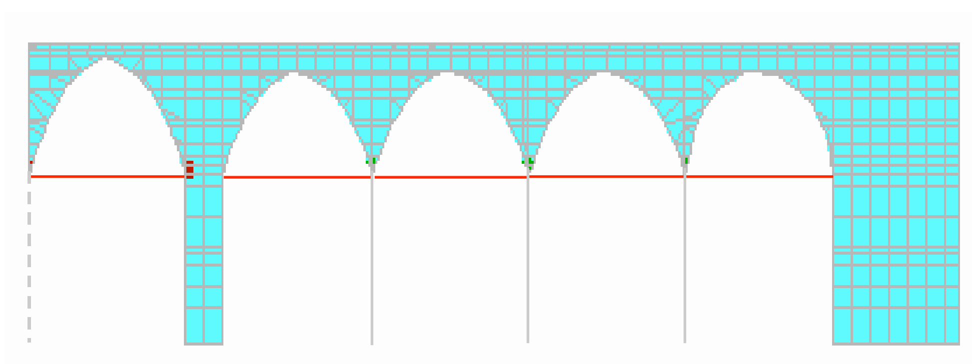
FAILURE UNDER BIAXIAL TENSION FAILURE UNDER BIAXIAL COMPRESSION

FAILURE UNDER HETEROSEMOUS STRESSES NON FAILURE

Fig. (5). Failure results of external wall (existing structure) for PGA values 0.10 and 0.35 g.



a) Structure without dampers



b) Structure with dampers

FAILURE UNDER BIAXIAL TENSION FAILURE UNDER BIAXIAL COMPRESSION

FAILURE UNDER HETEROSEMOUS STRESSES NON FAILURE

Fig. (6). Typical failure areas for internal wall with and without dampers (PGA=0.10 g).

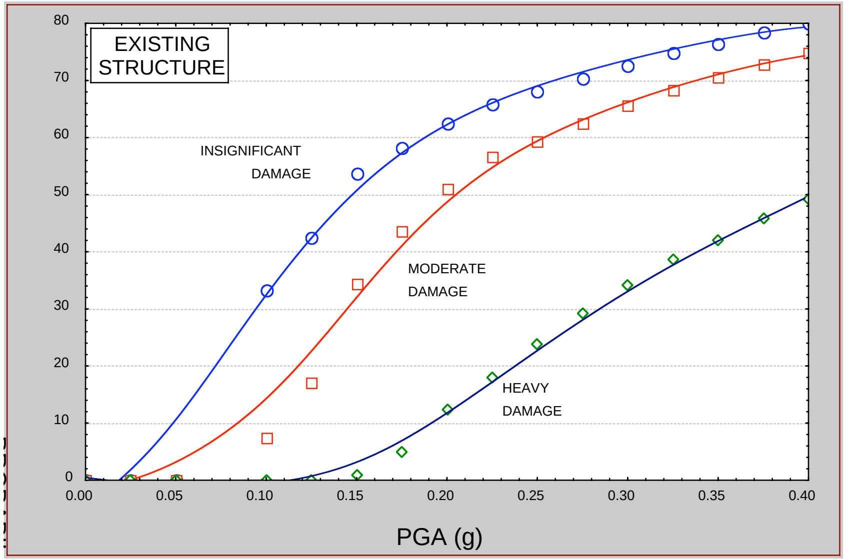


Fig. (7). Fragility curves for Agios Ioannis Prodromos Church in Cyprus (existing structure).

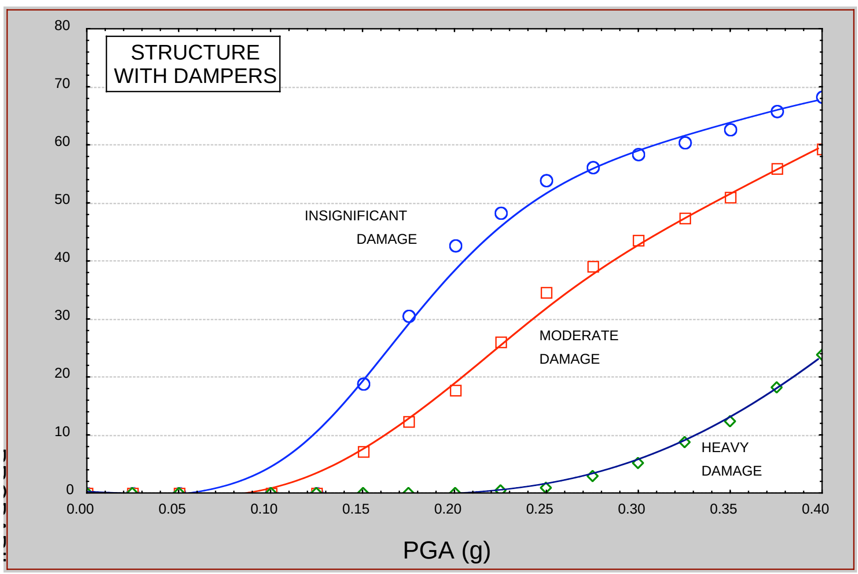


Fig. (8). Fragility curves for Agios Ioannis Prodromos Church in Cyprus (structure with dampers).

is reduced by 65% (that is, from 51% probability of damage to 19% probability of damage, as can be seen in Figs. (**7**) and (**8**) when the structure is fitted with fluid viscous dampers. This is a considerable reduction, which indicates that fluid viscous dampers can be effective in seismic protection of monumental structures in regions that are at high risk from earthquakes.

CONCLUSIONS

A general methodology for the quantitative estimation of the seismic vulnerability of historical structures has been proposed. According to the results of the analysis of the rehabilitated structure under study, the use of vibration control devices can significantly reduce seismic vulnerability, leading to an alternative method of strengthening historical structures against dynamic (earthquake) loads. The use of vibration control devices is particularly important, in encountering the complicated effects of unknown and uncertain interventions during the monument's lifetime, and thus effectively protecting the historical structure.

REFERENCES

[1] ICOMOS 1931, "The Athens Charter for the Restoration of Historic Monuments", Adopted at the First International Congress of Architects and Technicians of Historic Monuments, Athens, Greece, 1931, Available; http://www.icomos.org./athens_charter.html.

[2] ICOMOS 1964, "The Venice Charter for the Restoration of Historic Monuments, Adopted at the Second International Congress of Architects and Technicians of Historic Monuments", Venice, Italy, Available; http://www.icomos.org./e_venice.html.

[3] P.B. Lourenço, "Computational Strategies for Masonry Structures", Ph.D. thesis, Delft University of Technology, 1996.

[4] P.B. Lourenço, "Computations on historic masonry structures", *Progress in Structural Engineering and Materials*, vol. 4(3), pp. 301-19, 2002.

[5] P.B. Lourenço, "Recommendations for restoration of ancient buildings and the survival of a masonry chimney" *Construction and Building Materials*, vol. 20(4), pp. 239-51, 2006.

[6] P.B. Lourenço, K.J. Krakowiak, F.M. Fernandes and L.F. Ramos, "Failure analysis of Monastery of Jerónimos, Lisbon: How to learn from sophisticated numerical models", *Engineering Failure Analysis*, vol. 14(2), pp. 280-300, 2007.

[7] C.A. Syrmakezis, A. Giuffre, F. Wenzel and M. Yorulmaz, "Seismic aspects of preservation of historical monuments. General Report of Working Group 7 of the European Association on Earthquake Engineering", in Proceedings of the 9th European Conference on Earthquake Engineering, Moscow, Sept., 1990, pp. 303-46.

[8] C.A. Syrmakezis, A. Giuffre and M. Yorulmaz, "Seismic aspects of preservation of historical monuments. General Report of Working Group 7 of the European Association on Earthquake Engineering", in Proceedings of the 10th European Conference on Earthquake Engineering, Vienna, Sept. 1994, pp. 3099-128.

[9] C. Syrmakezis, M. Chronopoulos, A. Sophocleous and P. Asteris, "Structural analysis methodology for historical buildings", in Proceedings of the Fourth International Conference on Structural Studies of Historical Buildings, STREMA 95, Crete, Greece, 30 May 1995, vol. 1, pp. 373-82.

[10] C.A. Syrmakezis, "Seismic protection of historical structures and monuments" *Structural Control and Health Monitoring*, vol. 13(6), pp. 958-79, 2006.

[11] L. Binda, C. Modena, G. Baronio and S. Abbaneo, "Repair and investigation techniques for stone masonry walls", *Construction and Building Materials*, vol. 11(3), pp. 133-42, 1997.

[12] L. Binda, A. Saisi and C. Tiraboschi, "Investigation procedures for the diagnosis of historic masonries", *Construction and Building Materials*, vol. 14(4), pp. 199-23, 2000.

[13] L. Binda and A. Saisi, "Research on historic structures in seismic areas in Italy", *Progress in Structural Engineering and Materials*, vol. 7(2), pp.71-85, 2005.

[14] P.G. Asteris, "Analysis of anisotropic nonlinear masonry", Ph.D. thesis, National Technical University of Athens, Greece, 2000.

[15] P.G. Asteris, A.D. Tzamtzis, P.P. Vouthouni and D.S. Sophianopoulos, "Earthquake Resistant Design and Rehabilitation of Masonry Historical Structures", *Practice Periodical on Structural Design and Construction, American Society of Civil Engineers (ASCE)*, vol. 10(1), pp. 49-55, 2005.

[16] M. Devaux and P. Lestuzzi, "Seismic vulnerability of monumental buildings in Switzerland", in Proceedings of 9th International Conference on Structural Studies, Repairs, and Maintenance of Heritage Architecture IX, vol. 83, pp. 215-225, 2005.

[17] Y. Belmouden and P. Lestuzzi, “Seismic vulnerability assessment of existing buildings in Switzerland, Research Report”, Ecole Polytechnique Federale de Lausanne, 2006.

[18] F.l. Moon, T. Yi, R.T. Leon and L.F. Kahn, "Recommendations for the seismic evaluation and retrofit of low-rise URM structures", *Journal of Structural Engineering (ASCE)*, vol. 132(5), pp. 663-672, 2006.

[19] SAP2000, *Static and Dynamic Finite Element Analysis of Structures*. Computrs & Structures, Inc., Berkeley, 2003.

[20] G. Croci, "Restoration of historical structures: An introduction", *Structural Engineering International*, vol. 11(4), pp. 222, 2001.

[21] N. Yan, C.M. Wang and T. Balendra, "Optimal damper characteristics of ATMD for buildings under wind loads", *Journal of Structural Engineering (ASCE)*, vol. 125(12), pp. 1376-1383, 1999.

[22] C.M. Wang, N. Yan and T. Balendra, "Control on Dynamic Structural Response Using Active-Passive Composite-Tuned Mass Dampers", *Journal of Vibration and Control*, vol. 5(3), pp. 475-489, 1999.

[23] T. Balendra, C.M. Wang and N. Yan N, "Control of wind-excited towers by active tuned liquid column dampers", *Engineering Structures*, vol. 23(9), pp. 1054-1067, 2001.

[24] B.F. Spencer and S. Nagarajaiah, "State of the Art of Structural Control", *Journal of Structural Engineering*, vol. 129(7), pp. 845-855, 2003.

[25] N. Makris, M.C. Constantinou and G.F. Dargush, "Analytical Model of Viscoelastic Fluid Dampers", *Journal of the Structural Engineering Division*, vol. 119(11), pp. 3310-3325, 1993.

[26] A. Kareem, T. Kijewski and Y.Tamura, "Mitigation of motions of tall buildings with specific examples of recent applications", *Wind Structures*, vol. 2(3), pp. 201-251, 1999.

[27] Nishitani and Y. Inoue, "Overview of the application of active/semiactive control to buildings structures in Japan", *Earthquake Engineering and Structural Dynamic*, vol. 30, pp. 1565-1574, 2001.

[28] J.N. Yang and S.J. Dyke, "Kobori Panel Discussion: Future perspectives on structural control", in Proceedings of the 3rd World Conference on Structural Control, Wiley, New York, 2003, pp. 279-286.

[29] B.F. Spencer and M.K. Sain MK, "Controlling buildings: A new frontier in feedback", *IEEE Control Systems Magazine*, vol. 17(6), pp. 19-35, 1997.

[30] B.F. Spencer, "Civil engineering applications of smart damping technology", in Proceedings of the 5th International Conference on Vibration Engineering 2002, China, pp. 771-782.

[31] M.D. Symans and M.C. Constantinou, "Semi-active control systems for seismic protection of structures: A state-of-the-art review", *Engineering Structures*, vol. 21(6), pp. 469-487, 1999.

[32] L. Ramons and P.B. Lourenço, "Modeling and vulnerability of historical city centers in seismic areas: a case study in Lisbon", *Engineering Structures*, vol. 26, pp. 1295-1310, 2004.

[33] G. Milani, P.B. Lourenço and A. Tralli, "3D Homogenized limit analysis of masonry buildings under horizontal loads", *Engineering Structures*, vol. 29 (11), pp. 3134-3148, 2007.

[34] P.B. Lourenco, G. Milani, A. Zucchini and A. Tralli, "The analysis of masonry structures: review and possibilities of homogenisation techniques", *Canadian Journal of Civil Engineering*, vol. 34(11), pp. 1443-1457, 2007.

[35] G. Bruno, "Influence of foundation settlement on the behaviour of Strasbourg cathedral, France", *Structural Engineering International (IABSE)*, vol. 11(4), pp. 223-227, 2001.

[36] C.A. Syrmakezis and P.G. Asteris, "Masonry failure criterion under biaxial stress state", *Journal of Materials in Civil Engineering (ASCE)*, vol. 13(1), pp. 58-64, 2001.

[37] U. Andreaus, "Failure criteria for masonry panels under in-plane loading", *Journal of Structural Engineering (ASCE)*, vol. 122(1), pp. 37-46, 1996.

[38] M. Dhanasekar, A.W. Page and P.W. Kleeman, "The failure of brick masonry under biaxial stresses", *Proceedings of The Institution of Civil Engineers*, Part 2, vol. 79, pp. 295-313, 1985.

[39] Y.J. Park, A.H. Ang and Y.K. Wen, "Damage-limiting aseismic design of buildings", *Earthquake Spectra*, vol. 3(1), pp. 1-26, 1987.

[40] FEMA-273, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, 1997.

[41] C.A. Kircher, A.A. Nasser, O. Kutsu and W.T. Holmes, "Developing of Building Damage Functions for Earthquake Loss estimation" *Earthquake Spectra*, vol. 13(4), pp. 664-681, 1997.

[42] National Institute of Building Science (NIBS), HAZUS99 Technical Manual, Developed by the Federal Emergency Management Agency through agreements with the National Institute of Building Sciences (Washington, D.C.: NIBS), 1999.

[43] R.Gowing, The church of Ayios Ioannis Prodromos, Askas, Cyprus, Report, Conservation of Wall Painting Department, Courtauld Institute of Art, University of London, 1997.

[44] N.N. Ambraseys, "The seismic history of Cyprus", Revue Union Intern. Secours, Geneva, vol. 3, pp. 25-48, 1965.

[45] N.N. Ambraseys, "Reappraisal of the seismic activity in Cyprus: 1894-1991", *Bolletino di Geofisica Teorica ed Applicata*, vol. 34(133), pp. 41-80, 1992.

[46] Eurocode No 6 (EC6), Common unified rules for masonry structures, EEC, Report EUR 9888 EN, 1988.

[47] StatSoft, Inc., STATISTICA for Windows, Computer program manual, Tulsa, 1999.

Received: April 16, 2008 Revised: June 08, 2008 Accepted: June 09, 2008