Paper No. 6. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

11 WCEE

DETERMINATION OF EARTHQUAKE DURATION DEPENDENT BEHAVIOUR FACTORS FOR UNREINFORCED BRICK MASONRY PANELS BY NONLINEAR TIME HISTORY CALCULATIONS

J. EIBL[1], E. KEINTZEL[2] and V. VRATSANOU[3]

[1, 2] Universität Karlsruhe, Institut für Massivbau und Baustofftechnologie, Am Fasanengarten, Postfach 6980, D-76128 Karlsruhe, Germany
[3] Willibaldstr. 53, D-80689 München, Germany

ABSTRACT

The nonlinear behaviour of structures under seismic loading depending in an important measure on the duration of the seismic events, effective, realistic values of the behaviour factors, used in the seismic design, will depend also on this parameter. In the paper a method is presented, and subsequently demonstrated on the example of unreinforced brick masonry panels, to investigate this dependence by nonlinear time history calculations. By using this method a calibration of behaviour factors for masonry buildings, adapted to different regional seismicities, could be performed, leading in regions of low seismicity to higher values of the behaviour factor than in regions of high seismicity, and viceversa.

KEYWORDS

Behaviour factors, constitutive model for masonry, duration of earthquakes, failure criterion for masonry, regional seismicity, response spectrum analysis, unreinforced masonry.

INTRODUCTION

In the seismic design of structures the influence of their nonlinear behaviour under seismic loading is introduced by behaviour factors, by which the seismic forces, obtained from a linear analysis, are divided. However, the nonlinear behaviour of a structure, and as a consequence also its effective, realistic behaviour factor, due to the higher energy input and damage potential of long duration earthquakes (see e.g. Bertero and Uang, 1992; Meskouris and Krätzig, 1991), depends in an important measure on the earthquake duration. In the following a method is presented, and subsequently demonstrated on the example of unreinforced brick masonry panels, to investigate this dependance. To this end nonlinear time history calculations are performed for masonry panels, up to their failure under seismic loading, considering different earthquake intensities and durations. The analysis of the seismic performance of unreinforced masonry is realized by using a constitutive model, elaborated by Vratsanou (1991, 1992) for the numerical simulation of the behaviour of masonry under cyclic loading and implemented in a FE computer program.

Uniaxial Stress-Strain Relation

In the constitutive model developed by Vratsanou (1992) the masonry is considered as an ideal nonlinear homogenous material with the uniaxial stress-strain relation represented in Fig. 1. With the notations

E_o	-	initial tangent modulns,
f_c, f_t	-	compressive and tensile strength,
ε_c	-	strain at the attainment of f_c in monotonic loading,
$\sigma_f = -f_c C_\sigma$	-	stress at compression failure (point U in Fig. 1),
$\varepsilon_f = \varepsilon_c C_\varepsilon$	-	strain at compression failure (point U in Fig. 1),
$\bar{\sigma} = \sigma/(-f_c)$	-	stress normalized to f_c,
$\bar{\varepsilon} = \varepsilon/\varepsilon_c$	-	strain normalized to ε_c

the stress-strain relation for masonry under uniaxial cyclic loading (Fig. 1) may be expressed as follows:

- in the tension range (between T and O) by considering the linear relation

$$\sigma = E_o \, \varepsilon \tag{1}$$

- for the compression range, until to the attainment of the compressive strength (between O und C) by modifying slightly a relation proposed by Naraine and Sinha (1989a), which becomes

$$\sigma = E_o \, \varepsilon \, \exp \frac{\varepsilon}{\varepsilon_c} \ln \frac{(-f_c)}{E_o \, \varepsilon_c} \, , \tag{2}$$

- for the descending branch of the stress-strain curve in the compression range, by considering the linear variation

$$\bar{\sigma} = 1 + \frac{1 - C_\sigma}{1 - C_\varepsilon} (\bar{\varepsilon} - 1) \, , \tag{3}$$

- for unloading and reloading in the compression range, until to the attainment of the compressive strength, by considering straight lines with the slope E_o,
- for unloading and reloading in the compression range, from a point A between C and U, using the test results given by Naraine and Sinha (1989b), first the coordinates of the points P and B are defined as

$$\bar{\varepsilon}_P = 0.45 + 0.95 \, (\bar{\varepsilon}_A - 1) \qquad \bar{\sigma}_P = 0$$

$$\bar{\varepsilon}_B = \bar{\varepsilon}_A + 0.06 \qquad \bar{\sigma}_B = 1 + \frac{1 - C_\sigma}{1 - C_\varepsilon} (\bar{\varepsilon}_B - 1) \, ,$$

having then for the unloading

$$\bar{\sigma} = (\bar{\varepsilon} - \bar{\varepsilon}_P) \, \exp. \, 2.90 \, (\bar{\varepsilon} - \bar{\varepsilon}_P - b_1) \tag{4}$$

with

$$b_1 = - 1.44 \, \bar{\varepsilon}^2{}_P + 2.075 \, \bar{\varepsilon}_P - 0.28 \, ,$$

and for the reloading

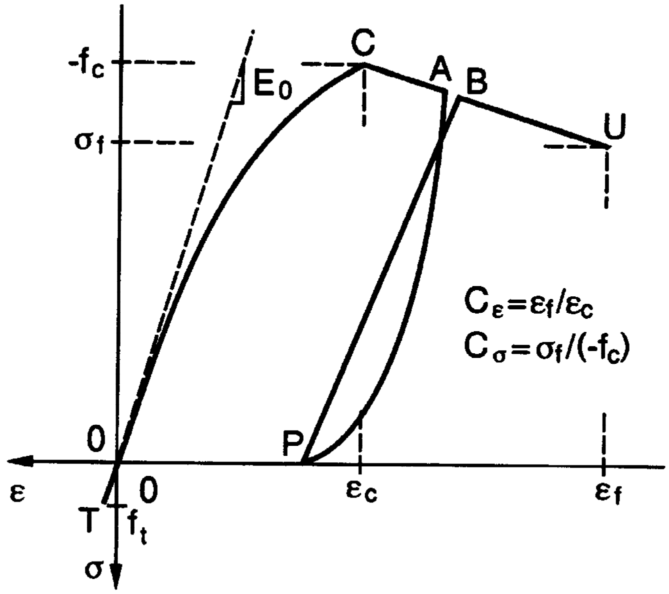


Fig. 1 Stress-strain diagram for masonry under uniaxial cyclic loading (Vratsanou, 1992)

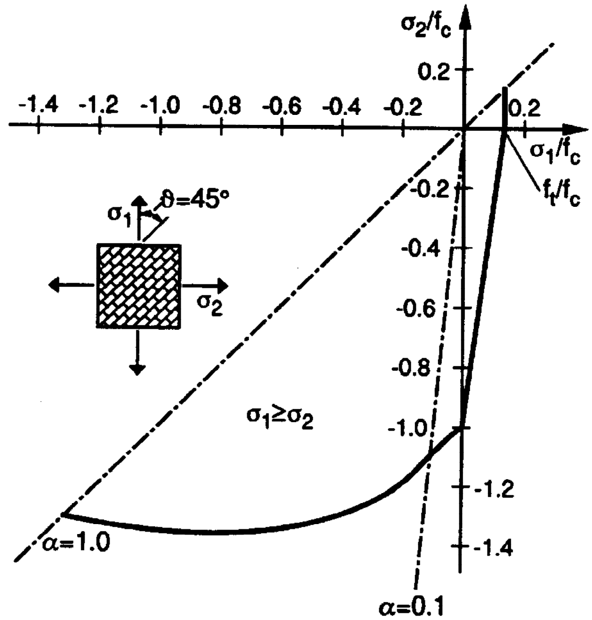


Fig. 2 Failure criterion for masonry (Vratsanou, 1992)

$$\bar{\sigma} = (\bar{\varepsilon} - \bar{\varepsilon}_P)\,\frac{\bar{\sigma}_B}{\bar{\varepsilon}_B - \bar{\varepsilon}_P} \tag{5}$$

Biaxial Loading

For the constitutive model developed by Vratsanou (1992) the behaviour of masonry under biaxial loading is reduced to the stress-strain relation under uniaxial loading by means of the principle of "equivalent uniaxial strain", proposed by Darwin and Pecknold (1974). The equivalent uniaxial strain ε_{iu} is always associated with the current principal stress axis i and depends, as a consequence of the failure criterion, discussed later, on the current stress ratio $\alpha = \sigma_1/\sigma_2$. It is defined in an incremental representation by the relation

$$\varepsilon_{iu} = \int d\,\varepsilon_{iu} = \int \frac{d\,\sigma_i}{E_i}\,, \tag{6}$$

where E_i represents the tangent modulus in the principal direction i, changing generally during the loading. Because the equivalent uniaxial strain does not transform in the same manner as stresses, it is only a fictitious measure in order to consider the variation of the material parameters during biaxial loading.

The biaxial stress-strain relation is idealized as incrementally linear orthotropic by the expression

$$\begin{bmatrix} d\,\sigma_{11} \\ d\,\sigma_{22} \\ d\,\tau_{12} \end{bmatrix} = \frac{1}{1-\nu^2} \begin{bmatrix} E_1 & \nu\sqrt{|E_1\,E_2|} & 0 \\ \nu\sqrt{|E_1\,E_2|} & E_2 & 0 \\ 0 & 0 & (1-\nu^2)\;G \end{bmatrix} \begin{bmatrix} d\,\varepsilon_{11} \\ d\,\varepsilon_{22} \\ d\,\tau_{12} \end{bmatrix}, \tag{7}$$

where ν represents the Poisson ratio and

$$G = \frac{1}{1 - \nu^2} \quad \frac{E_1 + E_2 - 2 \nu \sqrt{|E_1 E_2|}}{4} . \tag{8}$$

If further the incremental equivalent uniaxial strain in the direction i (respectively j) is expressed by the relation

$$d \varepsilon_{iu} = \frac{1}{1 - \nu^2} \left[d \varepsilon_i + \nu \sqrt{\left| \frac{E_i}{E_j} \right|} \; d \varepsilon_j \right] , \tag{9}$$

the relation (7) may be transformed into

$$\begin{bmatrix} d \sigma_{11} \\ d \sigma_{22} \\ d \tau_{12} \end{bmatrix} = \begin{bmatrix} E_1 & 0 & 0 \\ 0 & E_2 & 0 \\ 0 & 0 & G \end{bmatrix} \begin{bmatrix} d \varepsilon_{1u} \\ d \varepsilon_{2u} \\ d \gamma_{12} \end{bmatrix} , \tag{10}$$

It is shown that Equ. (10) corresponds to an uncoupling of the stress-strain relations in the two principal directions. In this way the stress-strain relation in biaxial loading may be reduced by means of equivalent uniaxial strains to uniaxial stress-strain relations.

Failure Criterion

The failure criterion represented in Fig. 2 and used for the numerical simulation of the masonry behaviour is based on the experimental study by Page *et al.* (1980), who tested masonry panels with different bed joint orientations for a range of principal stress ratios $\alpha = \sigma_1/\sigma_2$, with $\sigma_1 \geq \sigma_2$. As earthquake damages in masonry structures appear mostly by diagonal cracking, the failure criterion in calibrated by the test results obtained for an angle of $\vartheta = 45°$ between the directions of principal stresses and the bed joints. In these conditions the biaxial strengths σ_{2c} and σ_{1c} are determinded in the compression-compression range and $0.1 \leq \alpha \leq 1$ by the expressions

$$\sigma_{2c} = \frac{0.876 + 4.30 \alpha + 0.024 \alpha^2}{(1 + \alpha)^2} (- f_c) \quad and \quad \sigma_{1c} = \alpha \, \sigma_{2c} . \tag{11}$$

For $0 \leq \alpha \leq 0.1$ the biaxial strength σ_{2c} $(\alpha = 0.1) = - 1.08 f_c$ is decreased linearly to σ_{2c} $(\alpha=0) = -f_c$. For the tension-compression range a linear decrase from $\sigma_{2c} = -f_c$ for $\sigma_1 = 0$ to $\sigma_{2c} = 0$ for $\sigma_1 = f_t$ is considered, whereas for the tensile strength it is always assumed $\sigma_{1t} = f_t$.

For the strain corresponding to the attainment of the biaxial strength σ_{ic} it is introduced

$$\varepsilon_{ic} = \varepsilon_c \, \sigma_{ic} \, / \, (- f_c). \tag{12}$$

A numerical simulation of the postcracking behaviour is also given (Vratsanou, 1992).

Numerical Applications

The proposed constitutive model for masonry was implemented to the FE program ADINA and used for the numerical simulation of experimental results, obtained in tests under monotonic and cyclic loading up to failure. The good agreement between test results and numerical solution has shown the applicability of the proposed model to the calculation of the behaviour of unreinforced masonry under seismic loading.

PARAMETRIC INVESTIGATION: INFLUENCE OF THE EARTHQUAKE DURATION ON BEHAVIOUR FACTORS FOR UNREINFORCED BRICK MASONRY PANELS

In the following the results of the calculation of the seismic behaviour of the brick masonry panel shown in Fig. 3a, with the FE idealization of Fig. 3b, by using the above explained constitutive model, is presented. The material properties, considered in the calculations, are: $f_c = 10$ MN/m^2, $f_t = 0.8$ MN/m^2, $E_0 = 10^4$ MN/m^2, $\nu = 0.20$, $\varepsilon_c = -2 \cdot 10^{-3}$, $C_\sigma = 0.85$, $C_\varepsilon = 2.50$. No separate viscous damping is taken into account in addition to the hysteretic damping due to the nonlinear deformations of the panel. With a concentrated mass $m = 7 \cdot 20.0$ kNs^2/m $= 140$ kNs^2/m at the top of the panel the fundamental period of the elastic system is $T = 0.1$ s. An uniform vertical load with the values $\sigma_p = 1.6$ MN/m^2 and $\sigma_p = 3.2$ MN/m^2 is also considered. Beyond the above presented criterion for local failure, the appearance of a large, irreversible displacement at the top of the panel is used as criterion for its general failure.

The seismic excitation is introduced as a horizontal acceleration λ a(t), acting at the base of the panel up to the arbitrarily chosen moment $t = t_0$, where a(t) corresponds to the E-W component of the Friuli earthquake, 15.09.1976, recorded at Buia, represented in Fig. 4, and λ is a parameter. Nonlinear time history calculations of the seismic behaviour of the panel are performed by means of the mentioned FE program for each of different chosen values t_0 for a series of succesive values of the parameter λ, increased between $\lambda = 2.5$ and $\lambda = 5.0$ every time by $\Delta\lambda = 0.5$. In this way values $\lambda_u(t_0)$, for wich the failure of the panel occurs, are determined for different durations t_0 and represented in Fig. 5a as a function of t_0. The figure shows the decrease of the ultimate seismic loading with the increase of the considered earthquake duration.

For the calculation of the panel by response spectrum analysis, considering the acceleration spectrum $S_a(T,D)$ of the seismic motion of Fig. 4 with the damping ratio $D = 0.05$, the design resistance of the panel may be expressed as

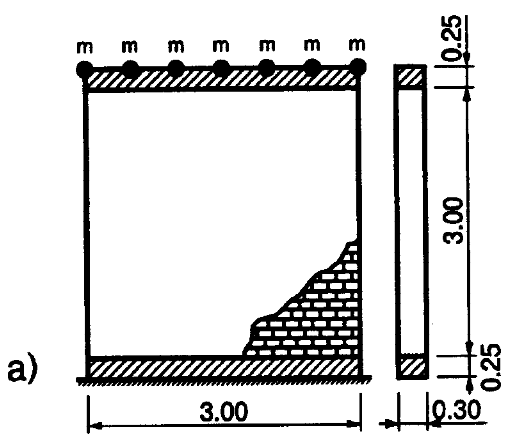
$$F_{ud} = m \, \lambda_{ud} \, S_a \, (T = 0.1 \; s, \; D = 0.05). \tag{13}$$

If in the calculation of the response spectrum the acceleration time history is considered likewise only up to the moment $t = t_0$, the value $\lambda_{ud}(t_0)$ of the parameter λ, for which the design resistance of the panel is attained in a linear spectral analysis, is determined as

$$\lambda_{ud} \, (t_o) = \frac{F_{ud}}{m \; S_a \; (T, \; D = 0.005, \; t_o)} \; . \tag{14}$$

Finally, the behaviour factor q representing the ratio between the ultimate seismic loading of the real, nonlinear system and that of the linear system considered in design, its earthquake duration depending value $q(t_0)$ is calculated as the ratio between $\lambda_u(t_0)$ and $\lambda_{ud}(t_0)$, that is as

$$q \, (t_o) = \frac{\lambda_u \, (t_o)}{\lambda_{ud} \, (t_o)} = \frac{\lambda_u \, (t_o) \; m \; S_a \; (T, \; D = 0.05, \; t_o)}{F_{ud}} \; . \tag{15}$$



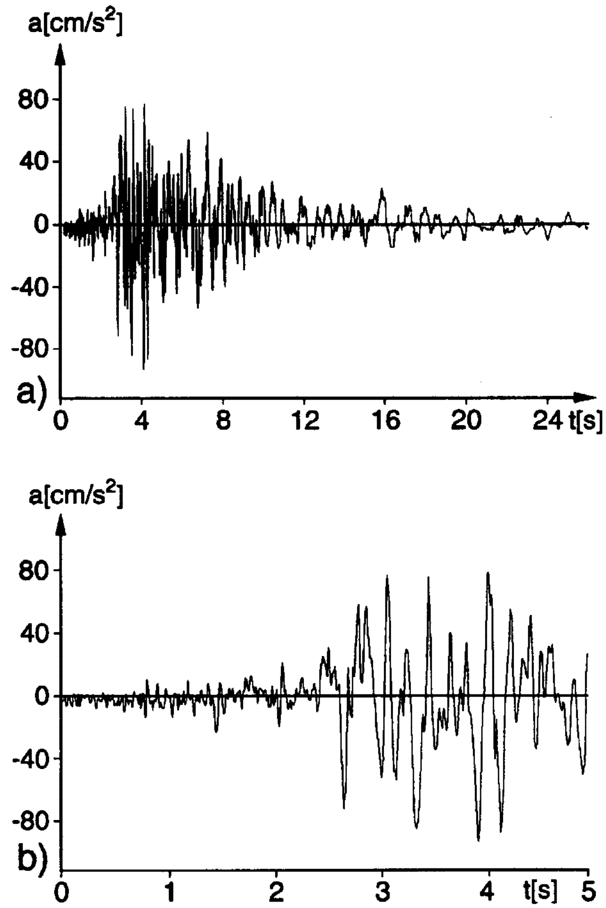


Fig. 3 Masonry wall for parametric investigations.
a) Dimensions,
b) FE-representation

Fig. 4 Ground acceleration a(t) recorded at Buia (Friuli), 15.09.1976, E-W.
a) Complete registration,
b) detail for the first 5 seconds

Evaluating the design resistance of the panel according to the German Masonry Code DIN 1053 (1984), it is found for $\sigma_p = 1.6$ MN/m^2 the value $F_{ud} = 365.5$ kN and for $\sigma_p = 3.2$ MN/m^2 the value $F_{ud} = 565.3$ kN.

The acceleration response spectrum $S_a(T,D = 0.05, t_o)$ of the considered earthquake is represented in Fig. 6 for different values of the parameter t_o. Due to the evolution in time of the frequency content of the accelerogram of Fig. 4, studied by Scherer (1984) and by Scherer and Schueller (1988), the spectral value for $T = 0.1$ s is the same for all the values t_o, considered in Fig. 6. This spectral value corresponds to the P-waves, appearing at the beginning of the seismic motion (see Fig. 4b), whereas the peak at $T = 0.2$ s corresponds to the S-waves, appearing later, and the peak at $T = 0.8$ s to the surface-waves, constituting the Coda of the accelerogram. As shown by the spectral curves of Fig. 6, the decrease of λ_u with increasing earthquake duration is not caused by higher spectral values, appearing later, but by the material degradation. The softening of the system due to nonlinear deformations may have also a certain influence.

Introducing in Equ. (15) the value $S_a(T = 0.1\text{ s}, D = 0.05, t_o) = 1.89$ m/s^2, which, as shown above, does not depend for the considered case on t_o, the earthquake duration dependent values of the behaviour factor $q\ (t_o)$, represented in Fig. 5b, are obtained. The different influence of a vertical load σ_p on the curves of Fig. 5a and on those of Fig. 5b may be explained by the different consideration of the tensile strength of masonry in the constitutive model (where it is always assumed with the value $\sigma_{1t} = f_t$, see Fig. 2) and in DIN 1053, 1984, (where, perpendiculor to the bed joints, it is neglected). Other differences between the behaviour of the considered panel under a particular earthquake and that of a real masonry structure under

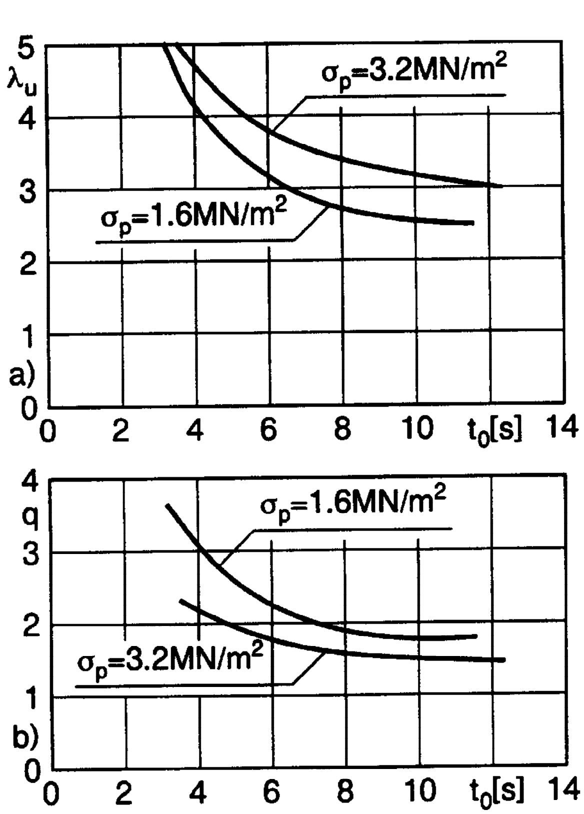


Fig. 5 Influence of the earthqauke-action duration t_0 on the seismic load bearing capacity of the wall in Fig. 3a.
a) Magnification factor λ_u, leading to the failure of the wall at time t_0 under the ground acceleration $\lambda_u a(t)$,
b) behaviour factor q as a function of t_0

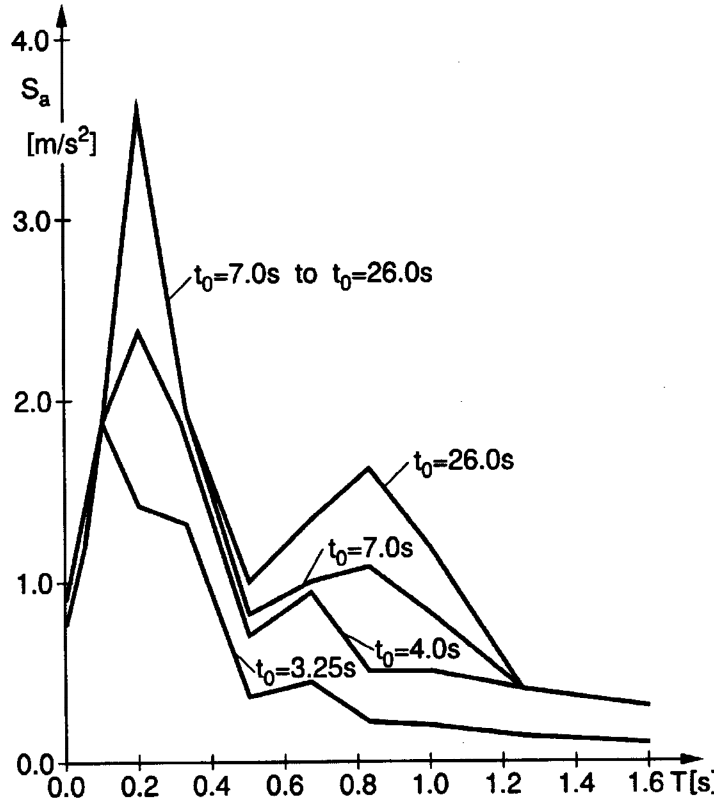


Fig. 6 Acceleration response spectrum for the ground acceleration of Fig. 4, from $t = 0$ to $t = t_0$, damping ratio $D = 0{,}05$

a more general seismic excitation appear as well in the constitutive model of the material (real existing anisotropy of masonry neglected in the failure criterion, particular choice of the strength and strain parameters of masonry), as in the model of the element, nonpossessing the redistribution possibilities of a structure, and in the position of the fundamental frequency of the element relative to the response spectrum and to the distribution in time of the prevailing frequencies of the seismic event. All these differences make that the numerical results, derived in this paper, could be generalized only with precaution.

CONCLUSIONS

The examination of the effects of past earthquakes has shown the decisive influence of the earthquake duration on the damages in masonry buildings, caused by ground motions with the same elastic response spectrum value. So the idea appears to take into account this influence in the determination of behaviour factors for masonry buildings: in regions of low seismicity, where earthquakes of lower magnitudes and consequently of shorter durations are expected, higher values of the behaviour factor could be admitted than in regions of high seismicity, where severe earthquakes of longer duration have to be considered. The method explained and exemplified in the paper demonstrates the possibility of quantitative investigations in this field. It may be used for the calibration of behaviour factors for masonry buildings, adapted to different regional seismicities.

REFERENCES

Bertero, V.V. and C.M. Uang (1992). Issues and Future Directions in the Use of an Energy Approach for Seismic-Resistant Design of Structures. In: *Nonlinear Seismic Analysis and Design of Reinforced Concrete Buildings*. (P. Fajfar and H. Krawinkler, ed). Elsevier, London.

Darwin, D. and D.A. Pecknold (1974). Inelastic Model for Cyclic Biaxial Loading of Reinforced Concrete. *Civil Engineering Studies, SRS No. 409*. University of Illinois at Urbana-Champaign, Urbana, Illinois.

DIN 1053, Teil 2 (1984). Mauerwerk. Mauerwerk nach Eignungsprüfung, Berechnung und Ausführung. Beuth, Berlin.

Mescouris, K. and W.B. Krätzig (1991). Seismic damage assessment of buildings. In: *Earthquake Resistant Construction and Design* (S.A. Savidis, ed). Balkema, Rotterdam.

Naraine, S. and S. Sinha (1989a). Behaviour of Brick Masonry under Cyclic Compressive Loading. *ASCE, Journal of the Structural Engineering 115*, 1432-1445.

Naraine, S. and S. Sinha (1989b). Loading and Unloading Stress-Strain Curves for Brick Masonry. *ASCE, Journal of the Structural Engineering, 115*, 2631-2644.

Page, A.W. *et al.* (1980). The In-Plane Failure of Masonry - A Review. *Procedings of the 7th International Symposium on Load-Bearing Brickwork.*London, 1980.

Scherer, R.J. (1984). The Importance of Surface Wawes in Structural Damage. *Proceedings of the Conference on Structural Analysis and Design of Nuclear Power Plants*. Porto Alegre, Brasil.

Scherer, R.J. and G.I. Schueller (1988). Quantification of Local Site and Regional Effects on the Nonstationarity of Earthquake Accelerations. *Proceedings of the 9th World Conference on Earthquake Engineering*. Tokyo-Kyoto.

Vratsanou, V. (1991). Determination of the Behaviour Factors for Brick Masonry Panels Subjected to Earthquake Actions. In: *Soil Dynamics and Earthquake Engineering V.* (IBF, Institut für Bodenmechanik und Felsmechanik, Universität Karlsruhe, ed). Computational Mechanics Publications, Southampton, Boston and Elsevier Applied Science, London. New York.

Vratsanou, V. (1992). Das nichtlineare Verhalten unbewehrter Mauerwerksscheiben unter Erdbebenbeanspruchung. Hilfsmittel zur Bestimmung der q-Faktoren. Dissertation, Universität Karlsruhe, Karlsruhe.