Effectiveness of seismic strengthening measures

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Effectiveness of Seismic Strengthening Measures

Efficacité des interventions de renforcement parasismique

Wirksamkeit von Verstärkungsmassnahmen gegen seismische Einwirkungen

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SUMMARY

The effectiveness of alternative techniques for seismic strengthening of stone-masonry buildings is studied by linear elastic Finite Element Analyses, using thick-plate/plane-stress combination elements to model the in-plane and out-of-plane behaviour of the walls. A biaxial failure criterion is developed for stone-masonry and used for safety checking. Alternative structural measures are compared on the basis of the resulting average reduction of the 'equivalent' biaxial stress in the masonry.

RÉSUMÉ

L'efficacité de techniques alternatives de renforcement parasismique de bâtiments en pierre est étudiée par des analyses linéaires, élastiques suivant la méthode des éléments finis. Le comportement des murs dans et hors de leur plan est simulé usant des éléments plats, épais sous un état bi-dimensionnel de contraintes. Un critère biaxial de rupture développé pour la maçonnerie en pierre a été utilisé pour la vérification. Les méthodes alternatives d'intervention ont été comparées entre elles à la base de la réduction moyenne des contraintes biaxiales équivalentes de la maçonnerie.

ZUSAMMENFASSUNG

Die Wirksamkeit verschiedener Methoden zur Verstärkung von Mauerwerksgebäuden gegen seismische Belastungen wird mit Hilfe linear-elastischer Finite-Elemente-Berechnungen untersucht. Zur Simulation des Verhaltens von Wänden in Scheiben- und Plattenwirkung, werden kombinierte Elemente von dicken Platten und Scheiben verwendet. Zur Kontrolle der Sicherheit wird ein biaxiales Versagenskriterium für Mauerwerk entwickelt. Die alternativen Verstärkungsmassnahmen werden aufgrund der resultierenden durchschnittlichen Reduktion der äquivalenten biaxialen Spannung im Mauerwerk miteinander verglichen.

1. INTRODUCTION

Stone or brick masonry or combinations thereof is the traditional construction material in Europe. Old masonry buildings, most of them constructed prior to this century, are common in our modern cities and towns, providing them with their traditional character and sense of historical continuity. For this reason old buildings with little individual historical or architectural value are often restored and put into new use. To a certain extent this usually requires structural interventions, to reverse the effects of post structural deterioration and damage and/or to bring the old building up to the safety level required from new structures by modern structural design codes. In seismic regions, e.g. in most of Southern Europe, old buildings often have suffered significant structural damage during past earthquakes and are subject to higher future risk. Therefore in such regions the importance of strengthening interventions is larger. Because of the low architectural and historical importance of most individual old buildings, interventions are not subject to strict requirements of reversibility and absolute respect to the original type and material of construction. Accordingly, the type and extent of structural interventions is usually decided on the basis of cost considerations. In this respect it is useful to have a general idea of the structural effectiveness of the various possible alternative strengthening measures, especially as the latter are often of a non-engineered nature, i.e. they are empirically applied, without design calculations or safety checks.

Field observations from past earthquakes, as well as detailed analytical studies by the first two authors [1], have shown that seismic damage to low-to-medium rise masonry buildings with flexible (e.g. timber) floors is mainly due to out-of-plane horizontal forces on the walls. These forces induce large magnitude nearly horizontal tensile stresses due to out-of-plane bending of the walls, as well as horizontal transfer forces at the intersections of orthogonal load-bearing walls. The former induce nearly vertical cracking, especially over upper storey openings, and out-of-plane overturning of the walls, whereas the latter cause separation of the walls from the transverse ones. This type of action and damage calls for the introduction of horizontal elements, such as reinforced concrete tie-beams or slabs and horizontal prestressing, to resist the horizontal tensile stresses in the walls, and to tie them together. In [2,3] the first two authors have compared analytically the effectiveness of such strengthening devices to that of vertical ones, such as tie-columns and vertical prestressing, and of universal interventions, such as one- or two-sided shotcrete jacketing of the walls. The tool used was the Finite Element linear elastic Analysis in three dimensions, applied to three two-storey (plus basement) stone-masonry buildings in Kalamata, Greece, statically subjected to the estimated horizontal response acceleration of 0.42g of the Kalamata 1986 earthquake, separately in the two horizontal directions but simultaneously with the gravity loads. The effectiveness of the various strengthening measures was quantified by computing the average reduction in the magnitude of the principal tensile stress in the masonry over each individual wall, storey or building, effected by each intervention.

In the present paper the same three buildings, considered typical of Greek and other Southern European stone-masonry construction of the 19th and early 20th century, are studied under a larger variety of strengthening interventions or combinations thereof under a horizontal response acceleration of 0.4g, equal to the design acceleration of stone-masonry buildings (behaviour factor equal to 1.5) in the main seismic-prone area of Greece (Zone 3, with a design ground acceleration of 0.24g). The main difference, though, with the earlier study [2,3] is the failure criterion used: Instead of the principal tensile stress criterion, which is certainly inadequate for biaxial stress conditions involving a significant compressive principal stress as well, an isotropic multiaxial failure criterion is developed herein and fitted to biaxial test results, and applied further over a denser grid of points over the surface of the masonry walls.

2. FAILURE CRITERION OF STONE MASONRY UNDER MULTIAXIAL STRESSES

Uncoursed rubble stone masonry is the typical material of old masonry structures, especially historic ones, in Greece and other Southern European countries, including the infill of brick-faced walls of

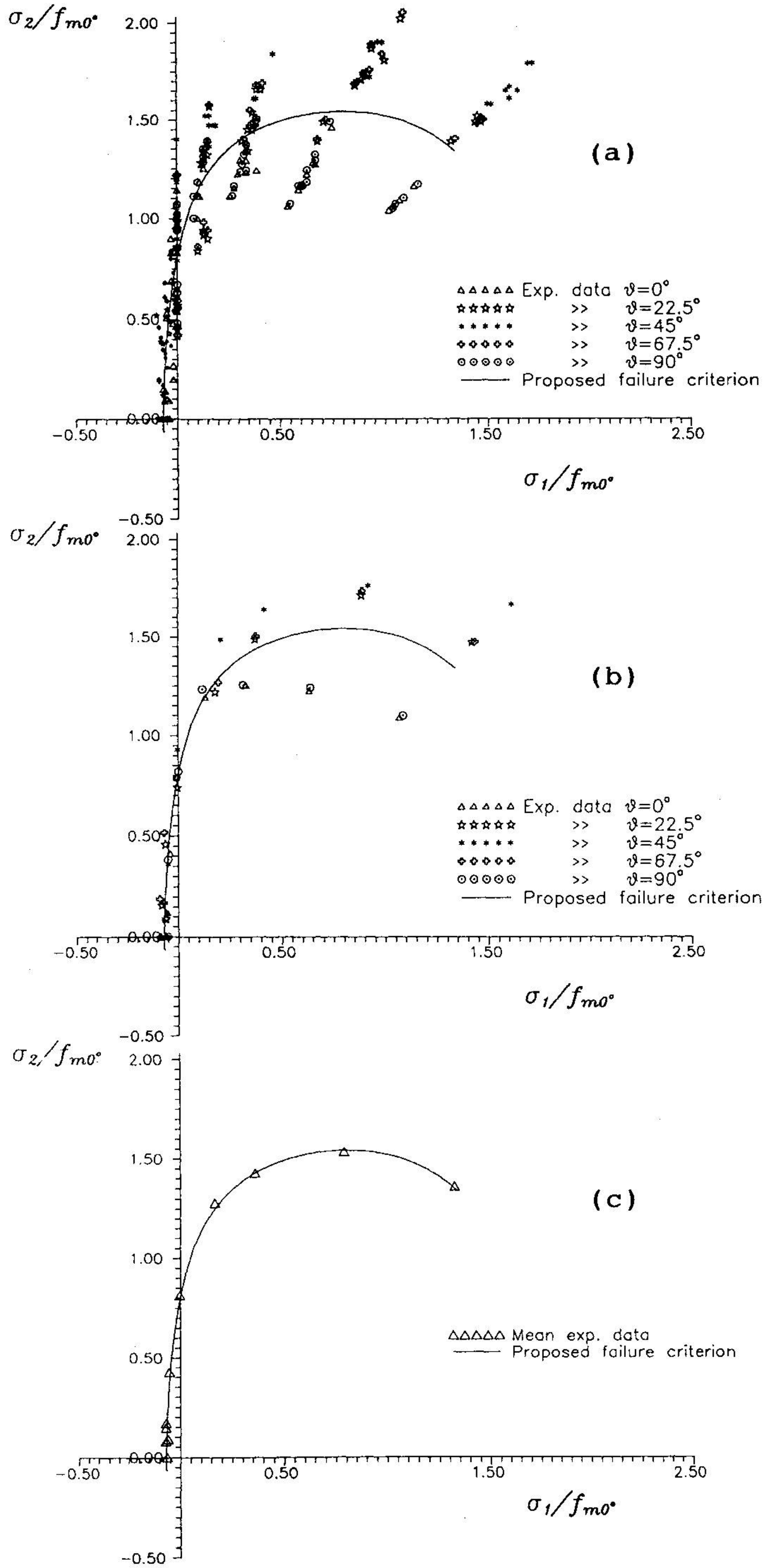

Fig. 1 Test data from [4, 5] and proposed failure criterion. (a) Individual data; (b) average data for each θ value; (c) average data for all θ.

medieval and Renaissance monuments. Despite of its importance, though, it has never been the subject of a systematic experimental investigation, such as those reported for brick masonry under biaxial stresses [4, 5]. In view of this lack of test data the authors had to resort to these latter results on solid brick masonry tested to failure under a variety of biaxial tension-compression and compression-tension principal stress combinations, oriented at various angles θ equal to 0^o, 22.5^o, 45^o, 67.5^o and 90^o with respect to the bed joints, and to fit a biaxial failure criterion to them after removing the dependence on the angle of θ.

The individual test data in [4, 5] are presented in Fig. 1(a) in biaxial principal stress space, using a different symbol for each of the five values of θ above. Fig. 1(b) shows the average of the test data separately for each value of θ. It is clear from Fig. 1(b) that in the compression-compression range strength is systematically and significantly lower for principal stresses parallel and normal to the bed joints ($\theta=0^o$ and $\theta=90^o$), whereas in the range from 22.5^o to 67.5^o the exact value of θ is not very important. It is assumed herein that the behaviour of isotropic masonry, such as stone masonry of the type considered herein, will be close to the average of all data, regardless of the value of θ. Accordingly a failure criterion was fitted to these average data as shown in Fig. 1(c). This criterion follows the four-parameter model proposed by Ottosen [6] for the failure of concrete under triaxial stresses:

$$\alpha\frac{J_2}{f_w}+\lambda\frac{\sqrt{J_2}}{f_w}+\beta\frac{I_1}{f_w} = 1 \tag{1}$$

in which $I_1=\sigma_1+\sigma_2+\sigma_3$ is the first stress invariant, $J_2=[(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2]/6$ is the second deviatoric stress invariant, f_w is the uniaxial compressive strength of masonry and parameter λ equals:

$$\lambda=c_1\cos\frac{\cos^{-1}(c_2 cos3\theta)}{3} \qquad if \quad cos3\theta\geq 0 \tag{2a}$$

$$\lambda=c_1\cos(\frac{\pi-\cos^{-1}(-c_2 cos3\theta)}{3}) \qquad if \quad cos3\theta<0 \tag{2b}$$

in which $\cos3\theta=3\sqrt{3}J_3/2J_2$, with $J_3=(\sigma_1-I_1/3)(\sigma_2-I_1/3)(\sigma_3-I_1/3)$ the third deviatoric stress invariant.

For given value of the "shape" parameter c_2 parameters α, β and c_1 can be computed from:

$$\alpha=\frac{9-(1+\frac{\lambda_1}{\lambda_2})\frac{3}{b}+(1-2\frac{\lambda_1}{\lambda_2})\frac{3}{f}}{3-(1+\frac{\lambda_1}{\lambda_2})b+(1-2\frac{\lambda_1}{\lambda_2})f} \tag{3a}$$

$$\beta=\frac{1}{3}(\frac{1}{f}-\frac{1}{b}+\frac{b-f}{3}\alpha) \tag{3b}$$

$$c_1=\frac{1}{\lambda_2\sqrt{3}}(\frac{2}{f}+\frac{1}{b}-\frac{2f+b}{3}\alpha) \tag{3c}$$

in which $\lambda_1=\cos((\pi-\cos^{-1}c_2)/3)$, $\lambda_2=\cos((\cos^{-1}c_2)/3)$, $f=f_{wt}/f_w$ is the ratio of uniaxial tensile and compressive strengths (equal to 0.085 on the average in Figs. 1) and b is the strength ratio in equal biaxial to uniaxial compression (1.65 on the average in Figs. 1). The best fit to the average data in Figs. 1 is obtained for $c_2=0.959$, in which case Eqs. (3) give $\alpha=0.665$, $\beta=3.84$ and $c_1=13.8$

The four-parameter model by Ottosen provides a very good fit to the failure data for concrete under triaxial stress conditions, $\sigma_1, \sigma_2, \sigma_3\neq 0$, albeit with parameter values very different from the ones fitted herein to the average biaxial strength data for masonry. Due to the similarities between concrete and uncoursed rubble stone masonry, it is expected that Eqs. (1) and (2) along with the above values of the four parameters will provide a good fit even to the triaxial strength of masonry ($\sigma_3\neq 0$).

The proximity of a biaxial stress state (σ_1,σ_2) to the failure criterion of Eq. (1) is quantified herein by computing the proportionality factor σ^* such that the stress point $(\sigma^*\sigma_1, \sigma^*\sigma_2)$ lies on the failure envelope, Eq.(1). So the scaling factor σ^* has the meaning of an "equivalent" stress under biaxial conditions, normalised to its failure value, and the value of $1/\sigma^*$ can be considered as a safety factor against failure of the masonry, with $\sigma^*<1$ signifying a safe stress condition inside the failure envelope and $\sigma^*>1$ implying failure due to fictitious elastic stresses σ^*-times beyond failure.

3. INTERVENTION MEASURES AND THEIR FINITE ELEMENT MODELING

The intervention measures considered in the present study cover the entire range of techniques commonly applied in Greece and in other Southern European countries for seismic strengthening of old masonry structures. As these techniques have been described in detail in [2, 3] they are only listed here, along with some remarks regarding their F.E. modeling.

1. Introduction of through-thickness 0.3m-deep horizontal **reinforced concrete (R.C.) tie-beams** at the levels of the floors and at the top of all load-bearing walls.
2. Construction of vertical **R.C. tie-columns** at the corners and intersections of all load-bearing walls, with horizontal dimensions equal to those of the common area in plan of the intersecting walls.
3. Replacement of timber floors by rigid within their plane **reinforced concrete slabs**.
4. Application of a 60mm-thick shotcrete layer on both sides of the wall, to create a **two-sided or double shotcrete jacket**.
5. As in 4. above, but on the external or the internal face of the exterior walls, to create a **one sided or single shotcrete jacket**.
6. Concentric **horizontal prestressing** of the spandrels over openings of the walls, at a level of prestressing force corresponding to an average horizontal compressive stress in the spandrel equal to 10% or 20% of the uniaxial compressive strength of the masonry, f_w.
7. Concentric **vertical prestressing** of all the piers of the wall, at a prestressing force level corresponding to a mean vertical compressive stress in the pier equal to $0.1f_w$ or $0.2f_w$.

Two-way or three-way combinations of the individual interventions are also considered:

8. 3 plus 1, i.e. R.C slabs at floor levels and a 0.3m deep circumferential tie-beam at the top of the wall.
9. 2 plus 1 at the top, i.e. reinforced concrete tie-columns at the intersections of load-bearing walls and a 0.3 m deep circumferential tie-beam at the top.
10. 7 plus 1 at the top, i.e. vertical prestress of the piers at an average compressive stress level of $0.1f_w$, along with a 0.3 deep horizontal R.C tie-beam at the top of the wall for anchorage of the tendons.
11. 6 plus 7, i.e horizontal and vertical prestressing at the two prestress levels mentioned above, i.e. at nominal average stresses of $0.1f_w$ and $0.2f_w$.
12. 1 plus 2 plus 3, i.e. reinforced concrete slabs at the floor levels, R.C. tie-columns at the wall corners, etc., and a 0.3m deep R.C. tie-beam at the top of the wall.
13. 4 plus 1 plus 3, i.e. one-sided shotcrete jacket combined with R.C. slabs at floor levels and with a R.C. tie-beam at the top.
14. 5 plus 1 plus 3, i.e. two-sided shotcrete jacket along with R.C slabs and with R.C tie-beam at the top of the wall.

Walls are modeled using a dense grid of thick (Midlin) plate bending - plane stress combination 4- to 8-node isoparametric Elements. Element dimensions are about 0.5 to 0.6 m on the average, and over a thousand Elements are used for each building. Reinforced concrete tie-beams and tie columns are modeled by assigning the Elastic Modulus of concrete to the corresponding elements of the F.E. model. As the main effect of reinforced concrete slabs is their diaphragmatic action, they are modeled by kinematically constraining all nodes of the F.E. model at the level of a floor into a rigid body motion within a horizontal plane. Shotcrete jackets are modeled by considering the thick-plate Elements as layered, with the 60mm outer layer(s) assigned the Elastic properties of concrete and inner core assigned those of the masonry. Finally, prestressing forces, horizontal or vertical, are introduced as consistent line loads along those F.E. boundaries where tendons are anchored.

4. RESULTS AND CONCLUSIONS

From each F.E. Analysis nodal stresses σ_x, σ_y and τ_{xy} within the plane of the wall are computed at both surfaces by surface-extrapolation from those at the Gauss points of the Element, and then averaged over the Elements connected to the node. Principal stresses σ_1 and σ_2 computed thereof are used to compute the value of the "equivalent" nondimensional stress σ^*. At each nodal point the maximum value of σ^* on either surface of the wall over all combinations of interest of the gravity load with the seismic action provides a measure of the most adverse biaxial stress conditions there. Contours of this maximum value of σ^* give a picture of the distribution of seismic demand over each wall. At a given point in the wall of the strengthened building the ratio of the maximum σ^*-value as above to that in the unstrengthened building measures the reduction in masonry biaxial stresses due to the intervention, and provides a local measure of the effectiveness of strengthening. The mean value of this ratio over the entire wall, over a storey of the building or over the entire building provides an average measure of the effectiveness of the intervention. The average value of this ratio over the three buildings is listed in Table 1 for each strengthening technique, separately for the walls which are normal to the seismic action to show the effectiveness of strengthening for the most-important out-of-plane behaviour, then for those which are parallel to it for the less-important in-plane one, and finally independent of the direction of the wall relative to the seismic action, i.e. for the most adverse direction of the latter. In the first line, denoted by "everywhere", the average ratio of the σ^*'s over all nodal points in the wall is listed, whereas in a second line, denoted as "critical regions", the average is taken only over those nodal points where the value of σ^* in the unstrengthened building exceeds 0.9. The second line results bear more gravity regarding the effectiveness of intervention than those of the first, and almost invariably show larger effectiveness in the critical regions than overall.

Among the individual interventions not-surprisingly the two-sided jackets come out as most effective, reducing biaxial masonry stresses by about 60% in the critical regions and by more than 40% overall. R.C. slabs, R.C. tie-beams and one-sided shotcrete jackets are almost equally effective, reducing stresses by about 1/3 in the critical regions and by 20 to 25% overall. Prestressing at a mean nominal stress of 0.2 f_w reduces critical region biaxial stresses by about 25%, when applied in the horizontal direction or by 20% when applied along the piers. Reducing the level of prestressing force by half has a less than proportional effect, as critical region stresses are reduced by about 1/6, with horizontal prestressing being slightly superior. R.C. tie-columns have a minor impact on the level of stresses.

Among the two-way combinations the difficult-to-construct horizontal and vertical prestressing at a nominal average stress of 0.2 f_w in both directions is very effective, reducing biaxial stresses in the critical regions by more than 50%, and with a high degree of repeatability among the buildings. This is the result of the beneficial effect of increasing compressive stresses on biaxial failure, as shown in Fig. 1. Adding a tie-beam at the top of the wall significantly increases the effectiveness of R.C. slabs or R.C. tie-columns, as average stress reduction in the critical regions rises to 45% or to 20%, respectively. The corresponding value is about 35% or higher when vertical prestressing at a nominal average stress of 0.1 f_w is combined with a tie-beam at the top of the wall, or with horizontal prestressing at the same level of average nominal stress. Finally, the three-way combinations do not offer a very significant advantage over their individual constituents, as a two-sided jacket plus R.C. slabs and a tie-beam at the top is a little better than the jacket alone, the same combination with the one-sided jacket is slightly better than the slabs and the tie-beam without the jacket, whereas the addition of R.C. tie-columns to the combination of slabs with a tie-beam at the top does not improve the effectiveness of the latter.

Most note-worthy among the results above are a) the relatively low effectiveness of the one-sided jacket; b) the good performance of prestressing in both directions at a nominal average stress level of 0.2 f_w and of the combination of R.C. slabs with a tie-beam at the top; and c) the relatively limited improvement effected by a three-way combination of interventions.

Table 1. Average Ratio of the Equivalent Stress in the Strengthened to those in the Unstrengthened Building

Intervention		Walls parallel to seismic action			Walls normal to seismic action			Irrespective of seismic direction		
		1st story	2nd story	Building	1st story	2nd story	Building	1st story	2nd story	Building
R.C Tie-Beams	everywh.	0.92	0.76	0.82	0.76	0.56	0.73	0.97	0.61	0.77
	cr. reg.	0.86	0.60	0.72	0.51	0.52	0.57	0.77	0.57	0.68
R.C Slabs	everywh.	0.74	0.78	0.79	0.66	0.86	0.77	0.70	0.82	0.76
	cr. reg.	0.59	0.67	0.63	0.42	0.73	0.70	0.59	0.72	0.67
R.C Tie-Columns	everywh.	1.28	1.13	1.24	0.96	0.89	0.96	1.18	0.97	1.08
	cr. reg.	1.06	1.03	1.05	0.87	0.84	0.84	0.93	0.94	0.96
One-sided (Single) Jacket	everywh.	1.02	0.78	0.95	0.76	0.59	0.65	0.93	0.66	0.81
	cr. reg.	0.79	0.68	0.74	0.60	0.57	0.58	0.74	0.65	0.68
Two-sided (Double) Jackets	everywh.	0.87	0.57	0.76	0.46	0.29	0.39	0.74	0.39	0.58
	cr. reg.	0.57	0.38	0.46	0.23	0.26	0.25	0.49	0.32	0.40
Horizontal Prestressing at 0.1 fw	everywh.	0.83	0.81	0.85	0.88	0.86	0.86	0.89	0.87	0.91
	cr. reg.	0.79	0.70	0.75	0.74	0.75	0.80	0.81	0.79	0.83
Horizontal Prestressing at 0.2 fw	everywh.	0.86	0.88	0.91	0.95	0.86	0.96	0.91	0.82	0.91
	cr. reg.	0.68	0.58	0.66	0.64	0.63	0.71	0.72	0.66	0.75
Vertical Prestressing at 0.1 fw	everywh.	0.93	0.91	0.89	1.06	0.83	0.90	0.89	0.83	0.83
	cr. reg.	0.86	0.92	0.88	0.89	0.88	0.85	0.87	0.89	0.84
Vertical Prestressing at 0.2 fw	everywh.	1.01	1.03	0.98	1.25	0.85	0.97	0.92	0.83	0.83
	cr. reg.	0.81	0.93	0.85	0.89	0.86	0.79	0.88	0.87	0.80
Slabs + Tie-Beam at the top	everywh.	0.75	0.74	0.78	0.63	0.52	0.62	0.69	0.57	0.66
	cr. reg.	0.65	0.69	0.59	0.30	0.49	0.45	0.56	0.50	0.55
Tie-Columns+ Tie-Beam at the top	everywh.	1.24	1.01	1.16	0.88	0.63	0.80	1.13	0.77	0.98
	cr. reg.	1.01	0.82	0.92	0.74	0.55	0.62	0.93	0.69	0.80
Horiz.+Vert. Prestressing at 0.1 fw	everywh.	0.72	0.66	0.69	0.87	0.65	0.74	0.73	0.67	0.69
	cr. reg.	0.60	0.60	0.61	0.58	0.61	0.59	0.65	0.65	0.65

Table 1 (continued)

Horiz.+Vert. Prestressing at 0.2 fw	everywh.	0.74	0.53	0.71	1.00	0.59	0.77	0.69	0.55	0.61
	cr. reg.	0.46	0.44	0.50	0.42	0.43	0.42	0.47	0.46	0.47
Vert. Prestr. +Tie-beam at the top	everywh.	0.91	0.71	0.81	1.03	0.55	0.77	0.85	0.53	0.70
	cr. reg.	0.82	0.60	0.69	0.79	0.49	0.55	0.81	0.54	0.63
R.C.Slabs + Tie-Beam + Tie-Columns	everywh.	1.16	1.01	1.15	0.71	0.65	0.73	1.05	0.74	0.93
	cr. reg.	0.88	0.77	0.84	0.38	0.55	0.51	0.77	0.65	0.72
Double Jack. R.C. Slabs + Tie-Beam	everywh.	0.82	0.54	0.71	0.38	0.17	0.30	0.71	0.33	0.54
	cr. reg.	0.50	0.28	0.39	0.12	0.15	0.15	0.42	0.24	0.32
Single Jack.+ R.C. slabs + Tie-Beam	everywh.	0.99	0.72	0.89	0.54	0.30	0.52	0.88	0.46	0.71
	cr. reg.	0.70	0.40	0.54	0.21	0.29	0.32	0.59	0.36	0.49

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REFERENCES

1. KARANTONI F.V. and FARDIS M.N., Computed v Observed Seismic Response and Damage of Masonry Buildings, *ASCE J. Struct. Engrg*, **118**(7), 1804-1821, 1992.

2. KARANTONI F.V. and FARDIS M.N., Effectiveness of Seismic Strengthening Techniques for Masonry Buildings, *ASCE J. Struct. Engrg*, **118**(7), 1884-1902, 1992.

3. KARANTONI F.V. and FARDIS M.N., Assessment of Intervention Techniques for Seismic Strengthening of Masonry Buildings, *Proc. 1st Int. Congr. Restoration of the Architectural Heritage and Building*, Canarias, July 1992.

4. PAGE A.W., The Biaxial Compressive Strength of Brick Masonry, *Proc. Inst. Civil Engrs.*, Part 2, **71**, Paper 8487, 893-906, 1981

5. PAGE A.W., The Strength of Brick Masonry under Biaxial Tension-Compression, *Int. J. Masonry Construction*, **3**(1), 26-31, 1983

6. OTTOSEN N., A Failure Criterion for Concrete, *ASCE J. Engrg. Mech.*, **103**(4), 527-535, 1977.