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Mechanical properties of old stone masonries

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ABSTRACT: The experimental data of in situ compression and diagonal compression tests have been used to mathematically model the behaviour of old stone masonry panels. The mechanical parameters have been identified by using some simple analytical relationships: elasto-plastic, bi-linear and parabolic. Both experimental results and identified parameters have been presented and discussed. Some simple analytical relationships among stiffness moduli, stresses and strains have been proposed.

1 INTRODUCTION

From the time of the earthquake in Central Italy in 1984, a research has been carried out in Abruzzo to determine and to classify the mechanical characteristics of old stone masonries which are in very common use in Central and Southern Italy. The objectives are: a) to determine reasonable mechanical parameters valid in a wide region; b) to compare them with those available in literature and in Italian codes, where mechanical properties are empirically established on the basis of masonry arrangement and aspect. In fact, data about old stone masonries are still quite poor and it seems proper to integrate a knowledge based on few data, often obtained in other countries, and to verify their reliability.

Compression and diagonal compression tests have been carried out in situ on old stone masonry specimens. They have been isolated from walls of an 18th century building in L'Aquila and of some buildings in an Abruzzon village (Villetta Barrea). The demolition of the building in L'Aquila was planned; the buildings in Villetta Barrea had suffered slight damages during the earthquake in 1984, and had therefore been or were being restored, lightly reinforced or partially reconstructed. Even if L'Aquila rests some distance from Villetta Barrea, about 100 km, the masonry arrangements are very similar. Walls are composed of irregularly arranged rounded calcareous stones, with dimensions varying up to 25-30 cm. They could frequently be considered to be made up of two independent vertical layers joined by mortar: the spaces around the stones are filled, often incompletely, with a rough mortar of very low mechanical properties. Test apparatus and preliminary results were presented in two papers by Beolchini and Grillo (1989, 1990).

A total of thirty tests have been performed: six compression tests (labelled as BC) and six diagonal compression tests (BD) in L'Aquila; nine compression tests (VC) and nine diagonal compression tests (VD) in Villetta Barrea. Each specimen was about 1 m by 1 m and from 0.35 to 0.60 m thick. Eight of them, labelled as VCR (four) and VDR (four), had been lightly strengthened before testing with a welded deformed wire fabric coated with cement grout. All compression tests and BD tests were performed with a cyclic loading; diagonal compression tests VD with fully reversed cyclic loading: every case resulted in collapse. The load at each cycle was predetermined on the basis of certain percentages, usually 1/3 and 2/3, of the probable ultimate strength. The ultimate strength has been evaluated by taking into account previous experiments and the arrangement of the masonry. Stress-strain diagrams have been plotted for each test (see Beolchini and Grillo (1989, 1990)).

Envelope curves are recognizable in every diagram. Typical stress-strain diagrams and corresponding envelope curves are shown in Fig. 1: they can be considered representative of the whole group of diagrams to which they belong. It can be noticed that diagrams VD4 and VD5 appear quite

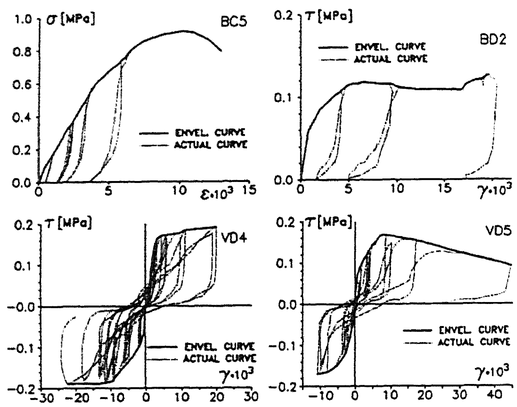

Figure 1. Some examples of stress-strain graphs

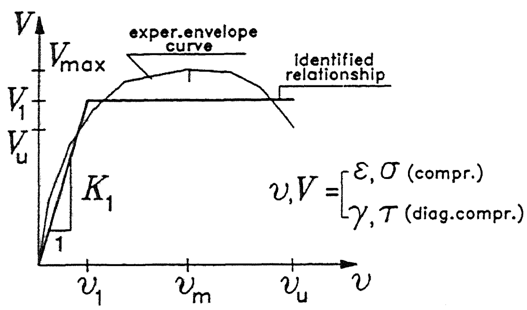

Figure 2. The parameters of the elasto-plastic relationship

different. The first one is basically symmetric with respect to the load directions, whereas the second one shows a clear unsymmetrical behaviour of the specimen. In this case a significant crack appeared during a loading in the positive direction (positive and negative are purely conventional here). The specimen experienced large deformations together with an appreciable decrease both in stiffness and in resistance at the end of all the following cycles in that direction. On the contrary, reversed loads in the negative direction have brought about reduced shear strains and do not have caused degradation.

2 IDENTIFICATION OF MECHANICAL PARAMETERS OF ENVELOPE CURVES

A numerical investigation has been conducted in order to identify three analytical models chosen to describe the envelope curves. A preliminary analysis of experimental data has suggested to direct attention to simple relationships which are frequently used in literature:

a) elasto-plastic:

$$V = K_1 \nu, \qquad \nu < \nu_1$$
$$V = K_1 \nu_1 = V_1, \qquad \nu > \nu_1 \tag{1}$$

b) bi-linear:

$$V = K_1 \nu, \qquad \nu < \nu_1$$
$$V = K_1 \nu_1 + K_2(\nu - \nu_1), \qquad \nu > \nu_1 \tag{2}$$

c) parabolic:

$$V = -p_1 \nu^2 + K_1 \nu, \tag{3}$$

V and ν respectively being shear stress τ and shear distorsion γ when referred to diagonal compression tests, and compression stress σ and longitudinal deformation ε when referred to compression tests. The parameters that had been identified in previous equations were stiffness moduli K_1 and K_2 (K_1 is the initial tangent stiffness modulus in the parabolic relationship), the elastic strain ν_1 and p_1.

The elasto-plastic relationship and its parameters is shown as an example in Fig. 2.

The parameters of each envelope curve have been identified by minimizing the objective function

$$L(p) = \sum_k (z_k - \nu_k(p))^2, \tag{4}$$

z_k being the values of the experimental stress-strain envelopes, and $\nu_k(p)$ being the model responses computed by the appropriate relationship. Furthermore the areas under envelope curves and identified diagrams have been compared: the differences are often less than 1% and in any case less than 4%: so the energetic equivalence of the two diagrams is ensured.

The actual experimental values have been used. In some test the collapse occurred when the load was still increasing, or lightly fluctuating around a certain value: therefore the maximum value of stress or other parameters, normally used to determine dimensionless diagrams, can not always be determined in a clear manner and applicable in every case.

On the other hand some original envelope diagrams have been modified before using the identification procedure. In some cases, in fact, the specimen experienced very large deformations before the collapse, with a certain low decrease in load. In these cases the original envelope diagrams have been truncated when the descending branch crossed a stress value equal to 0.9 the maximum value reached in the test. The occurence of too large deformations in normal conditions is not admissible: therefore the adopted truncation criterion can be considered reasonable, even if it is not the only one.

3 IDENTIFIED MODELS FOR DIAGONAL COMPRESSION TESTS

The fully reversed cyclic loading envelope diagrams have been split before using the identification procedure in order to separate the part corresponding to the positive values of stresses and strains from the negative ones (remember that "negative" and "positive" are purely conventional here). Therefore, two distinct diagrams have been identified for each fully reversed cyclic loading envelope curve: a suffix *p* or *n* is added to the corresponding label.

Identified relationships have been plotted and compared. The analysis showed that:

a) the parabolic law is absolutely inadequate to describe stress-strain relationships both in cyclic and in fully reversed cyclic tests. Identified models are often very different from the experimental ones in shape as well as in values of maximum stress;

b) the bi-linear law gives the best fitting of experimental diagrams. It can be noticed that multi-linear relationships give better approximation as a consequence to the kind of envelope curve to be identified: some trials done using a tri-linear law give better results. The stiffness moduli K_2 are often very closed to zero: their absolute values are a low percentage (less than 10 per cent) of K_1. Only in two cases is K_2 about 30 per cent of K_1: a significant hardening tipifies the corresponding diagrams;

c) the elasto-plastic law gives a very satisfying description of experimental envelope curves. Therefore it seems to be the best choice, taking into account the extreme simplicity of the relationship as

Table 1. Mechanical parameters of diagonals compression tests

	K_1 [Mpa]	γ_1 [$*10^3$]	τ_1 [MPa]	τ_{max} [MPa]	γ_m [$*10^3$]
BD1	17.51	9.60	.168	.175	17.97
BD2	49.05	2.27	.111	.127	19.55
BD3	60.01	2.27	.136	.148	10.24
BD4	47.56	2.15	.102	.108	8.36
BD5	18.40	5.41	.100	.114	16.85
BD6	14.94	5.19	.078	.090	21.61
med	34.58	4.48	.116	.127	15.76
max	60.01	9.60	.168	.175	21.61
min	14.94	2.15	.078	.090	8.36
VD1p	51.24	2.17	.111	.113	3.72
VD1n	29.08	3.95	.115	.127	7.35
VD2p	19.49	5.14	.100	.105	11.43
VD2n	14.30	5.86	.084	.090	16.94
VD3p	18.29	7.64	.140	.153	21.82
VD3n	22.70	6.31	.143	.159	11.01
VD4p	59.10	2.95	.174	.191	19.89
VD4n	58.99	2.82	.166	.189	22.47
VD5p	43.62	3.58	.156	.168	8.78
VD5n	48.30	3.22	.156	.171	10.87
med	36.51	4.36	.135	.147	13.43
max	59.10	7.64	.174	.191	22.47
min	14.30	2.17	.084	.090	3.72
VDR1p	41.79	6.02	.252	.281	8.85
VDR1n	86.17	3.13	.270	.285	12.15
VDr2p	132.31	1.98	.262	.273	11.19
VDR2n	52.06	4.97	.259	.298	11.00
VDR3p	141.50	4.52	.640	.665	5.47
VDR3n	101.68	6.42	.653	.685	12.31
VDR4p	162.20	3.65	.592	.616	11.10
VDR4n	172.62	3.19	.551	.596	8.21
med	111.29	4.24	.435	.462	10.04
max	172.62	6.42	.653	.685	12.31
min	41.79	1.98	.252	.273	5.47

well as the low values of the stiffness moduli K_2, computed by using bi-linear relationship.

The mechanical properties of specimens tested in diagonal compression tests have been given in Table 1 where shear stiffness moduli $K_s = K_1$, elastic shear strains γ_1, and plastic shear stresses $\tau_1 = K_1\gamma_1$ are parameters identified by using the elasto-plastic relationship. Experimental data have been split into homogeneous groups.

The mean values of mechanical properties of unreinforced panels are very similar. It is significant that panels BD and VD have been isolated in sites at a distance of about 100 km, and the dispersion of VD data, related to different buildings, is similar to that of BD data, which have referred to one building.

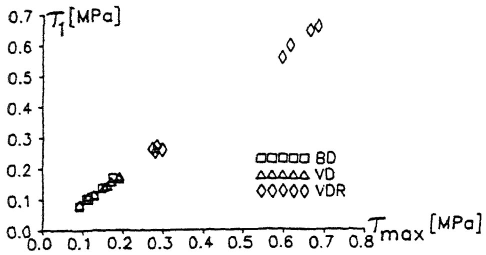

Figure 3. Identified shear stress τ_1 versus experimental τ_{max}

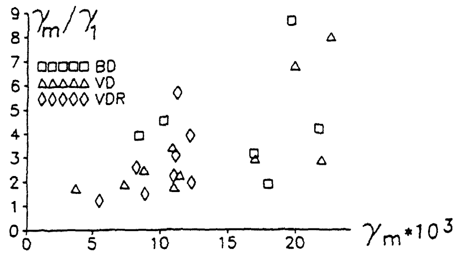

Figure 5. Ratio γ_m/γ_1 versus identified strain γ_m

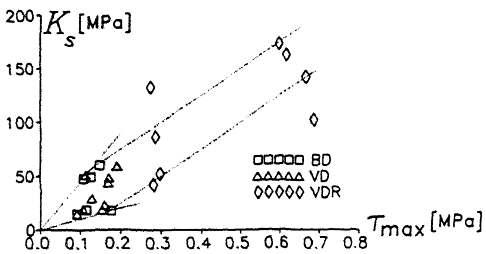

Figure 4. Shear stiffness moduli K_s versus experimental τ_{max}

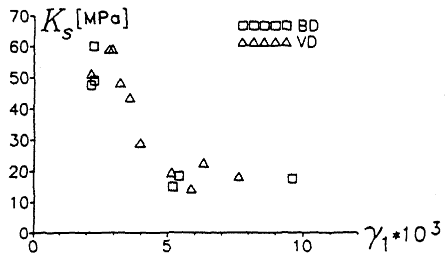

Figure 6. Correlation between shear stiffness modulus K_s and γ_1

The reinforcement, even if light, raises the stiffnesses and the ultimate resistances of the panels significantly, which are about three times higher than those of unreinforced ones. On the contrary, the shear strain γ_m, which could be used to define the ductility of the panel, is lower than those of unstrengthened panels. This is basically due to the behaviour of the reinforced panels. Cement grout reinforcing increases stiffnesses and resistances; when it becomes detached, the values of the mechanical parameters decrease quite rapidly to those of the original wall: panels have still maintained the ductility resources, even if they are at a stress level sensibly lower than that previously attained.

The Figure 3 shows the strong correlation between τ_{max} and τ_1: the mean value of the ratio τ_1/τ_{max} is equal to 0.92, and is in agreement to that usually suggested in literature, as well as for other kinds of masonry.

The connection between the identified shear modulus $K_s=K_1$ and τ_{max} is clearly recognizable in Fig. 4, even if there is some difficulty in establishing a precise relationship. A reasonable proposal seems to consider all the data enclosed in the area shown in Fig. 4. The exclusion of only two points, which are in any case related to rather odd strengthened panel tests, permits the proposal of an option to $K_s = 1100\ \tau_k$, the most frequently used empirical relationship between shear stiffness modulus and ultimate shear strain:

$$K_s = (100 \div 450)\ \tau_{max}, \qquad \tau_{max} < 0.15 \quad (5)$$

$$K_s = 250\ \tau_{max} + a, \qquad \tau_{max} > 0.15 \quad (6)$$

where a varies from -25 to 25. The shear stiffness modulus K_s can still vary in a non-negligible range, but it can at least be established taking into account an actual mechanical parameter.

The ratio between strain corresponding to maximum stress γ_m and elastic strain γ_1 can be considered as a prudential measure of ductility, taking into account that in general the panels have tolerated deformations higher than γ_m, without significant reduction in the strength. The plot in Fig. 5 reveals that the greater part of these ratios vary from about 2 to about 4. Lower values occur in diagrams like that corresponding to negative loads of the test VD4 (Fig. 1).

The interesting connection between K_s and γ_1 is shown in Fig. 6: the shear stiffness modulus decrea-

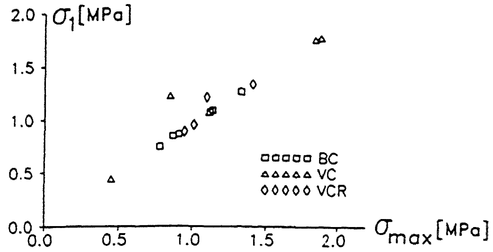

Figure 7. Identified compression stress σ_1 versus experimental σ_{max}

ses quite linearly until γ_1 is more or less about a value of 5, and remains basically constant for higher values. The strengthened panels data are not indicated in Fig. 6 because they are completely out of range. However they follow, in any case, the same trend in the same range of strains.

It is worth revealing that all the above diagrams show that the results do not depend on the site where panels have been tested.

4 IDENTIFIED MODELS FOR COMPRESSION TESTS

All relationships used in the identification procedure are apt to describe experimental envelope diagrams.

In particular, the parabolic law fits accurately envelope curves in every section, therefore the results of other researchers are confirmed. As a consequence, identified initial stiffness moduli and maximum compression stresses, not reported in this paper, agree in a excellent way with experimental data: in some cases a difference between strains corresponding to maximum stresses has been found, depending on the actual curve truncated due to a sudden collapse of the specimen.

As in the diagonal compression tests, the elasto-plastic law seems to be a reasonable compromise between accuracy and simplicity. The stiffness moduli are about 20 per cent lower than initial tangent moduli, identified by the parabolic relationship: this can be considered acceptable if the progressive reduction of stiffness in the parabolic law is taken into consideration. Furthermore the identified values of plastic stress σ_1 are only a few per cent lower than the experimental values, as shown in Fig. 7 where the good correlation is evident.

In Table 2 experimental values of maximum compression stresses σ_{max} and corresponding strains ε_m are reported together with parameters identified by the elasto-plastic relationship: longitudinal stiffness moduli $K_l = K_1$, elastic longitudinal strains σ_1, and plastic stresses $\sigma_1 = K_1\ \varepsilon_1$. Experimental data are split into homogeneous groups.

The data in Table 2 show that the panel behaviour does not seem to be conditioned by the reinforcement. Maximum stresses in tests VCR are in the same range of those of unstrengthened panels and stress-strain relationships, not reported in this paper, are very similar: only the strains corresponding to maximum stresses reached in the tests are slightly higher in reinforced panels. This is most likely due to: a) the light type of reinforcement used and the low bond between the reinforcing cement and the wall; b) the manner of the panels collapsing, which tended to break apart thus separating the two vertical layers of which the wall was composed. As a consequence, panel behaviour in compression tests essentially relates to the mechanical properties of the panel and the reinforcement only delays the collapse by confining the masonry.

Table 2. Mechanical parameters of compression tests

	$K1$ [Mpa]	ε_1 [*10³]	σ_1 [MPa]	σ_{max} [MPa]	ε_m [*10³]
BC1	124.80	8.76	1.09	1.14	10.07
BC2	97.76	8.67	.85	.87	10.81
BC3	130.80	8.23	1.08	1.12	12.10
BC4	306.80	4.13	1.27	1.33	7.31
BC5	136.90	6.32	.87	.91	10.16
BC6	171.70	4.34	.75	.78	9.06
med	161.46	6.74	.98	1.02	9.92
max	306.80	8.76	1.27	1.33	12.10
min	97.76	4.13	.75	.78	7.31
VC1	70.70	6.26	.44	.45	13.36
VC2	219.50	4.87	1.07	1.11	6.97
VC3	209.40	5.85	1.24	.85	9.88
VC4	191.10	9.30	1.78	1.88	16.18
VC5	170.00	10.35	1.76	1.84	18.39
med	172.14	7.33	1.25	1.23	12.96
max	219.50	10.35	1.78	1.88	18.39
min	70.70	4.87	.44	.45	6.97
VCR1	101.60	13.16	1.34	1.41	17.16
VCR2	72.90	12.21	.89	.95	27.20
VCR3	84.50	11.29	.95	1.01	14.77
VCR4	214.30	5.67	1.22	1.10	18.08
med	118.33	10.58	1.10	1.12	19.30
max	214.30	13.16	1.34	1.41	27.20
min	72.90	5.67	.89	.95	14.77

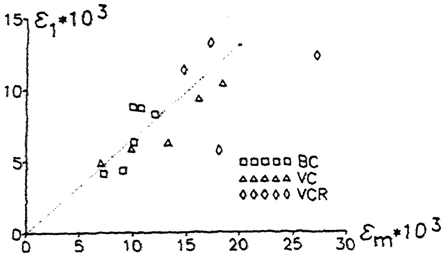

Figure 8. The correlation between identified elastic strain ε_1 and ε_m

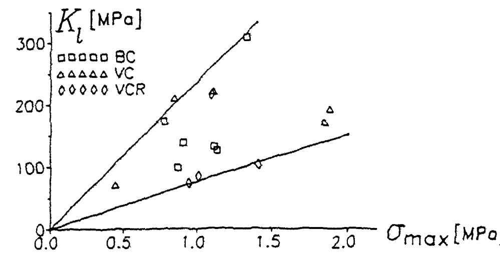

Figure 9. Identified longitudinal stiffness moduli K_l versus maximum compression stress σ_{max}

The identified elastic strain ε_1 seems to be dependent on the strain ε_m, corresponding to maximum stress, as shown in Fig. 8: if the data of strengthened panels are excluded, the following relation seems to be suitable for directly evaluating directly the elastic strain once the strain ε_m is known:

$$\varepsilon_1 = 0.65\, \varepsilon_m, \tag{7}$$

Other data are sensibly scattered, in a manner which is not dependent on the site where the panels have been tested. No general relationships among them are recognizable, with the exception of the identified longitudinal stiffness moduli $K_l = K_1$ and the experimental maximum stresses plotted in Fig. 9. In this case the points are lying in an area limited by two straight lines: the equation

$$K_l = (75 \div 235)\, \sigma_{max}, \tag{8}$$

gives a range of variation which is still generally large, but can be considered more precise than other empirical relationships.

CONCLUSIONS

The following main conclusions can be drawn when analyzing the previous results:

- the ultimate strengths are basically in the range suggested in the published literature, even if they are sensibly higher than that estimated from the masonry aspect or its arrangement; it could be necessary to investigate the influence of dimensions of the tested panels;

- the scatter of experimental data depends on the quality of masonries rather than the site where tests have been performed; as a consequence, the results can be considered suitable for framing the mechanical characteristics of masonries in a large region;

- the scatter nevertheless, makes it possible to define simple relationships in order to mathematically model the behaviour of masonry panels, both in compression and in diagonal compression tests;

- the analytical relationships such as (5), (6) or (8) give a range of variation which is large in an absolute sense: nevertheless they are related to a specific masonry and therefore can be considered more appropriate than other empirical expression which have referred to a generic masonry.

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