dissipation capacities of the structural system under consideration, i.e. the response modification factor q and pier ductility factor μ. Some aspects need to be examined, namely: (1) the appropriate value of q and μ representing the real behaviour of unreinforced masonry systems; (2) the correlation between base shear coefficients such to give rise in the two approaches to the same protection for a given seismic area; (3) the effective safety level of code-based designs with respect to a real strong ground motion. These assessments will be attempted in what follows by the interpretation of the experimental results of tests carried out on single stone-masonry piers and on simple buildings both under static loading and base seismic excitation through ISMES shaking table. Base shear forces will be expressed as ratios C to the total weight; moreover a suffix 1 will be used to denote forces at the significant yield point and the suffix 2 the forces at ultimate. A suffix c will additionally denote calculated forces and a suffix x experimental forces.

DEFINITIONS AND EXPERIMENTAL BACKGROUND

Tests on single piers give as a basic product shear-displacement loops whose envelope for shear failure have the typical shape shown in fig. 1(a) (limited to positive forces and displacements). The shear mode of failure is characterized by early flexural cracks at the bases of the pier followed by a diagonal crack which is then augmented up to failure. The first crack occours at shear forces T_f that vary from 85% to 100% of the maximum shear T_M (Refs.1, 2,3) and for height to width ratios up to 1,5 and can be predicted by:

$$T_f = F\,\tau_k\sqrt{1 + \frac{\sigma_o}{1.5\tau_k}}\ ;\ \tau_k = \frac{\sigma_{tr}}{1.5} \qquad (1)$$

with F the pier cross sectional area, σ_o the vertical normal stress, σ_{tr} the diagonal tensile strength and τ_k the referential shear strength. Conservatively the maximum shear force can be considered to be expressed by T_f and ultimate pier conditions to be attained when $\delta = \delta_M$. This leads to a bilinear schematisation of the real curve represented in fig. 1(b) where linear elastic behaviour is assumed up to the diagonal crack. The ultimate displacement δ_M can be expressed as a function of the limit elastic displacement δ_o in terms of pier ductility μ : $\delta_M = \mu\,\delta_o$. Based on this idealized pier behaviour, a simplified method for predicting building non linear response under static lateral load has been proposed (Ref.1). The method was widely used to check the overall lateral resistance of strengthened buildings after the Friuli (1976) and Irpinia (1980) earthquake. Given a masonry wall it consists in determining its lateral resisting capacity at a given storey level by summing up the shear resistances of single piers at equal horizontal displacements. As an example consider the wall of fig. 2(a) whose piers have shear-deflection characteristics shown in fig. 2(b). These result in the wall response curve shown in the same figure. For multiple wall systems the total lateral load is distributed among the single walls according to stiffnesses corresponding to their actual displacements. In the method significant yield is considered to occur at the first diagonal crack in the first yielding pier. In the simple case of fig. 2 it corresponds to point A and to a base shear coefficient $C_{1c}=F_{1c}/W$, with W the total weight. Ultimate wall conditions arise when a pier first reaches its maximum displacement $\mu\,\delta_o$. For the wall of fig. 2 this corresponds to point B and to $C_{2c} = F_{2c}/W$. In all this the role of pier ductility μ is apparent: it determines the extent to which contributions to lateral resistance of single piers can be summed up before one of them attains its limit displacement. Some experimental research work on the seismic behaviour of stone masonry buildings has been carried out during the last years in Europe. It involved (a) the analysis of single piers both under static and cyclic loads (Refs.2, 3, 4); (b) the analysis of single storey simple rectangular buildings under static lateral load (Ref. 5); (c) the analysis of two-storey buildings under static and seismic load conditions (Ref. 6) (base excitation through shaking table). The main results, relevant to the aim of the present work, are now summarised; details and further information can be found in the appropriate references. As to pier behaviour, tests show values of μ ranging from 2.3 to 2.8 if referring to the idealized curve of fig. 1(b). Note that if reference is made to the real experimental envelope the ratio of the dispacement at maximum shear to the first crack δ_M/δ_F ranges from 1. to 2.25. The average value of the referential shear strength τ_k (see eq. 1) for single and two-layered walls is found to be 0.8 kg/cm^2 while for the same grouted walls it is 1.5 kg/cm^2. Static tests on single storey stone masonry houses were carried out on 12 models (scaled 1:2) built under different strengthening conditions (r.c. beams and columns, horizontal and or vertical steel tendons, etc.). Here reference is made only to the results of plain masonry systems. The base shear coefficient at the first diagonal crack, considered as the significant yield point was on average $C_{1x} = 0.318$. This value is about 85% of the ultimate base shear coefficient which was found to be $C_{2x} = 0.375$. Static tests were also carried out on a two storey system, see fig. 3, similar to a prototype which was subjected to seismic base excitation by means of ISMES shaking table. The same base record was used (Sturno (nov. 1980) accelerogram) by scaling all the accelerations of the same factors, ranging from 0.15 to 2.5, thus imposing on the building shocks of increasing intensity up to failure. The static test gave the following shear base coefficient values $C_{1x} = 0.373$; $C_{2x} = 0.433$. The characteristics of the shock immediately before the one causing collapse were (in g-units) EPA = 0.56; $a_M = 0.66$). The static experimental behaviour of the two storey system (fig. 3) can be reasonably predicted by the simplified procedure outlined above provided the true pier characteristics μ and τ_k are used. These were found by single pier tests performed on wall panels built with the same material as the tested houses. A pier ductility $\mu = 2.4$ (following the scheme of fig. 1(b)) and a referential stress $\tau_k = 0.89$ kg/cm^2 were found. The results of the numerical simulation are shown in fig. 4 together with the experimental force-deflection curve. The difference between the predicted and the experimental maximum resistances is less than 5% while for the points of significant yield it is less than 4%. A good agreement holds between the displacements at significant yield (2 mm from the experiments, 2.9 mm for the numerical simulation). The first attainment of the maximum load occurs on the contrary at different displacements (6 mm for the numerical simulation 9 mm during the tests). The overall comparison is however satisfactory.

CODE IMPLICATIONS OF TEST RESULTS

a) Pier properties

The proposed code 2 values of the shear referential strength τ_k and of pier ductility μ are listed in tab. 1 together with the corresponding average values derived by single pier tests. The table shows also the ratio of experimental to code values, they express the degree of conservation for the basic design parameters as assumed by the code under discussion. As to shear strengths this degree is rather uniform for grouted and ungrouted masonry; it must be noted however that the efficiency of grouting (and the corresponding τ_k value)is widely dependent on the quality of the operation and on the diffusion of the cement mixture within the wall. This may suggest a more conservative level for it. The average value of μ obtained by single pier tests is confirmed by the interpretation of the results of the static tests carried out on structural assemblies (Refs. 5 and 6).

TABLE 1

	ungrouted	grouted	
	τ_k (kg/cm^2)	τ_k (kg/cm^2)	μ
code	0.70	1.1	1.5
tests	0.92	1.5	2.4
exp.val./code val.	1.31	1.37	1.6

The analysis is performed as follows, based on the simplified method described in the previsions section. For each model the ultimate lateral capacity C_{2c} is determined for different values of τ_k and μ. This leads to a family of curves which, for the two storey system, is shown in fig. 5. The curves corresponding to the true values of τ_k when entered with the experimental lateral resistance C_{2x} will give the value of pier ductilities μ consistent with the adopted numerical model. By interpreting test results in this way values of μ raging from 2.1 to 2.25 are obtained. Their ratios to the code value range from 1.4 to 1.5, being close to the one obtained by single pier analysis. The code value $\mu = 1.5$ seems to be reasonably appropriate being the experimental ductilities determined by imposing only inplane forces without considering the simultaneous out of plane actions occurring during earthquakes on real buildings.

b) <u>Response modification factor</u>

The response modification factor q accounts for the overstrength capacity of structures above the load level causing significant yield. Basically q represents the ratio of the forces which would develop under a given ground motion if the structure behaved linearly elastic to the forces at significant yield (see Ref. 7). The factor q has a crucial importance in determining the code design force level at significant yield. The typical code relationship given C_{1c} is as follows (for low natural periods):

$$C_{1c} = \frac{r\,A}{q} \tag{2}$$

A being the EPA of the seismic zone under consideration and r an amplification factor depending on soil profiles. This is made equal to 2.5 in ATC 3-06 tentative provisions and ranges from 2.5 to 3.0 in code 1. For unreinforced masonry ATC 3-06 suggests $q = 1.25$ while in code 1 no value has been fixed to date although discussion is focussed on the range $q = 1.0{:}1.5$. Available experimental results allow an estimate of q for the simple tested systems. This will be attempted in two ways: basing on static test results and basing on shaking table output.

In the first approach reference is made to the envelope of the loading-unloading cycles determined experimentally (fig. 4). The envelope is limited to point B of fig. 4 at which the maximum displacement is attained under the maximum lateral load; beyond B displacements increase quickly while lateral resistance decreases and no practical interest related to the survival of real structures holds. Energy absorption evaluated from the experimental process is equated to the one determined by a lateral force level F_e acting on the structure if it behaved linearly elastic. Factor q is evaluated as the ratio of F_e to the force level at significant yield, which was found to be, as already seen, 37.3% of the total vertical load. A value of q = 2.93 id thus estimated. Similar values (q= 2.8÷3.05) are obtained by interpreting the results of static tests of Ref. 5. This procedure considers forces F_e as those to be developed in a linear system by an earthquake causing in the real structure an energy absorption equal to the one involved during in the static test.

If shaking table tests are considered the elastic lateral forces level F_e is determined by entering the response spectrum of the earthquake causing ultimate conditions (EPA = 0.56 g) with the original natural period, obtained from frequency sweeping on the undamaged structure (T_o = 0.135 sec.). The estimate of the earthquake forces F_e is made by using a fraction α of the total weight with the first mode only. By calling S_a (T_o, ν) the spectral acceleration corresponding to the fundamental period and to damping ν this leads to:

$$F_e = \alpha \; S_a \; (T_o, \; \nu) \; . \; M_{tot} \qquad (3)$$

with M_{tot} the total mass of the building. The following expression for α (see Ref. (8)) is used to account for higher mode effects:

$$\alpha = \frac{0.017}{T_o} + 0.686 \qquad (4)$$

A value α = 0.812 is obtained for the building under consideration. By referring to damping ratios of 5% and 10% to critical and by using the significant yield force level given by static tests values of q ranging from 3.03 to 4.05 are obtained. It must be noted that these values are derived by coupling structural properties (such as ν, T_o and C_{1x}) and the properties of the earthquake in question. The above estimate of q has hence to be seen as resulting from the considered base excitation. However it supplies, tagether with the estimates carried out on a static basis, rough reference values with the limitations inherent in the simplified procedures adopted. Moreover it must be noted that in all the tests considered so far no simultaneaus inplane and out of plane forces were acting. In order to account for this the above values of q (3÷4) need to be reduced. The amount of this reduction is questionable and is not supported by experimental evidence. Under the hypothesis that a reduction coefficient 0.6 is appropriate tentative values of q ranging from 1.8 to 2.4 are derived. They are higher than the ones quoted ealier (1.25 for ATC 3-06; 1÷1.5 for code 1). It should be noted that in ref. 8 the ATC proposed value is considered to be excessively conservative.

c) Base shear coefficients

The use of the code design parameters of tab. 1 allows us to calculate the base shear coefficients for the tested buildings. These are denoted as C_{1code} and C_{2code} depending on whether they refer to significant yield or to ultimate. They are compared in tab. 2 with the corresponding experimental quantities. The ratio C_x/C_{code} will be termed overdesign factor (ODF).

TABLE 2

	C_{1code}	C_{1x}	ODF	C_{2code}	C_{2x}	ODF
Bldgs. Ref.5	0.292	0.318	1.09	0.322	0.375	1.16
Bldgs. Ref.6	0.310	0.373	1.20	0.335	0.433	1.29

As can be seen the ODF's are not high, it should however be considered that they refer to static design and to static responses. Under seismic exitation the system of Ref. 6 failed at an EPA value of 0.56 g. Although no simultaneous inplane and out of plane actions were impressed it seems that the effective overstrength capacity of such structures is fairly high. In fig. 6 the force level at significant yield (eq. 2) is plotted for different values of A and q assuming r = 2.5. By entering these curves with the experimental shear coefficient C_{1x} EPA values corresponding to different reduction factors q are obtained. These are listed in tab. 3. The first row refers to tests of Ref.5,

TABLE 3

q	1.	1.25	1.5	1.75	2.	2.25
A (g.units)	0.127	0.159	0.191	0.223	0.254	0.286
A (g.units)	0.149	0.186	0.224	0.261	0.298	0.335

the second one to tests of Ref. 6. The above A values can be seen as representing the characteristics of seismic zones feasible with experimental data and with the assumed response modification factor for the given building. By assuming that q = 2.25 interprets the real behaviour under simultaneous inplane and out of plane actions the values A = 0.286 g and A = 0.335 g are obtained. The ratios of the data of tab. 3 to these can be assumed to represent the ODF's resulting from different choices of the allowable q (fig. 7). A choice of q = 1.5÷1.75 corresponds to ODF = 1.5÷1.3 which seems to be conservative enough. In this range could be selected the code value for plain masonry.

Reference to eq. 2 and to above premises make it possible to compare the levels of protection involved in the two code approaches now under discussion in Italy. For code 1 the EPA values corresponding to the three Italian seismic zones have not yet been defined. A possible hypothesis for them is shown by the first column of tab. 4. The second column show the base shear coefficients C_{2c} used by code 2 for strengthened buildings in damaged areas of Southern Italy. By assuming C_{2c}/C_{1c} = 1.1 (as suggested by the numerical analysis of the buildings considered herein) and q = 1.75, equation 2 gives the EPA

TABLE 4

	A_1	C_{2c}	A_2
zone A	0.35	0.40	0.250
zone B	0.25	0.28	0.178
zone C	0.15	0.16	0.110

values A_2 listed in the third column. As can be seen A_2 and A_1 differ for the same seismic zone, A_1 being 40% greater than A_2. The listed A_1 values are thus more conservative code prescriptions than the ones used, for the same seismic areas, by code 2 for repairs and strengthenings. It must be noted that in turn these were reasonably conservative with respect to experimental results.

CONCLUSIONS

The interpretation of test results discussed above suggests the following tentative conclusions. (1) The suggested values for τ_k and μ for plain stone masonry are consistent with the esperimental results; (2) a response reduction factor of 1.5÷1.75 might be recommended in code 1 for plain masonry; (3) the level of protection assumed by code 2 is less conservative than the one assumed by code 1 for the same seismic area (supposing that A_1 values of tab. 4 hold); (4) buildings designed according to code 2 show however a satisfactory overstrength capacity.

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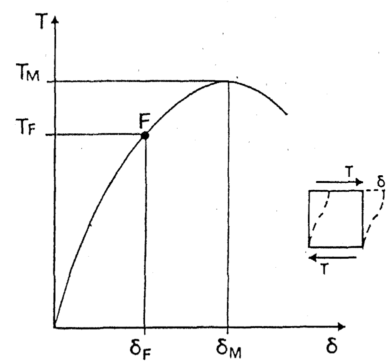

Fig.1(a)

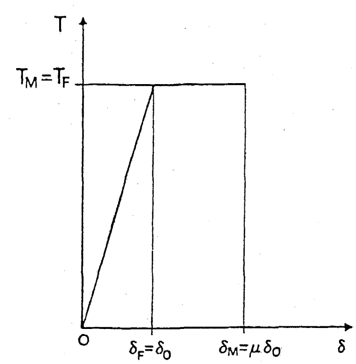

Fig.1(b)

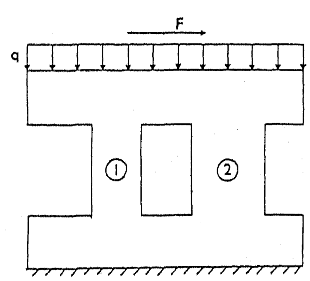

Fig.2(a)

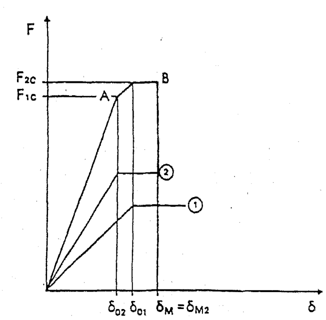

Fig.2(b)

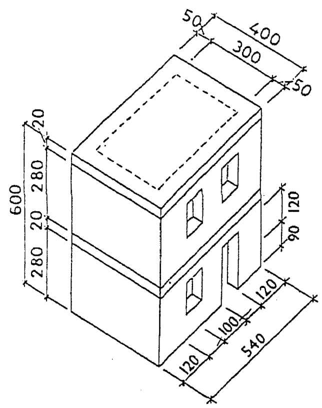

Fig.3

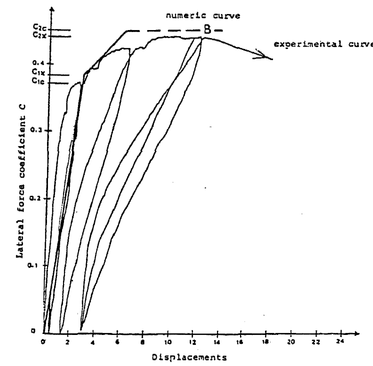

Fig.4

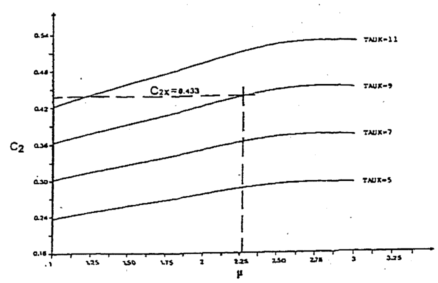

Fig.5

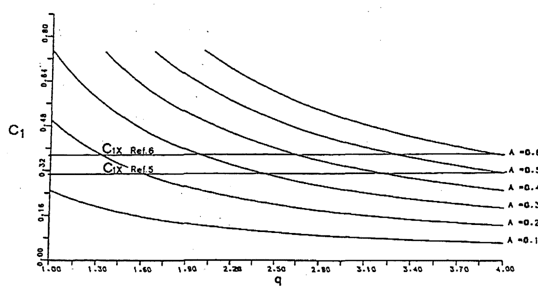

Fig.6

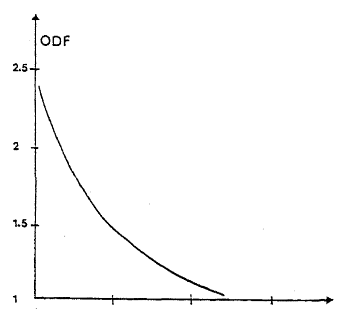

Fig.7

INDEXES