dissipation capacities of the structural system under consideration, i.e. the response modification factor $q$ and pier ductility factor $\mu$. Some aspects need to be examined, namely: (1) the appropriate value of $q$ and $\mu$ 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 ISMHE 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 occurs 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 can be predicted by:

$$T_f = F \cdot \tau_k \sqrt{1 + \frac{\sigma_0}{1.5\tau_k}} \quad \tau_k = \frac{\sigma_{tr}}{1.5}$$

with $F$ the pier cross sectional area, $\sigma_0$ the vertical normal stress, $\sigma_{tr}$ the diagonal tensile strength and $\tau_k$ 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 $\delta_M$ can be expressed as a function of the limit elastic displacement $\delta_0$ in terms of pier ductility $\mu$ : $\delta_M = \mu \delta_0$. 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. 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 $C_1 = F_1/W$, with $W$ the total weight. Ultimate wall conditions arise when a pier first reaches its maximum displacement $\mu \delta_0$. For the wall of fig. 2 this corresponds to point B and to $C_2 = F_2/W$. In all this the role of pier ductility $\mu$ 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 ma-
sonry buildings has been carried out during the last years in Europe. It in-
volved (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 ap-
propriate references. As to pier behaviour, tests show values of \( \mu \) ranging from
2.3 to 2.8 if referring to the idealized curve of fig. 1(b). Note that if ref-
ereence is made to the real experimental envelope the ratio of the displace-
ment at maximum shear to the first crack \( \frac{\Delta y}{\Delta p} \) ranges from 1. to 2.25. The aver-
age value of the referential shear strength \( \tau_k \) (see eq. 1) for single and two-
layered walls is found to be \( 0.8 \, \text{kg/cm}^2 \) while for the same grouted walls it is
1.5 \text{ 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 ultima-
te base shear coefficient which was found to be \( C_{2x} = 0.375 \). Static tests we-
re 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 (Storno (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) \( E_p = 0.56 \); \( a_m = 0.66 \). The static experimental behaviour
of the two storey system (fig. 3) can be reasonably predicted by the simplifi-
ed procedure outlined above provided the true pier characteristics \( \mu \) and \( \tau_k \n\) 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 \( \mu = 2.4 \) (follo-
wing the scheme of fig. 1(b)) and a referential stress \( \tau_k = 0.89 \text{ kg/cm}^2 \) were
found. The results of the numerical simulation are shown in fig. 4 together
with the experimental force-deflection curve. The difference between the pre-
dicted 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 betwe-
en the displacements at significant yield (2 mm from the experiments, 2.9 mm
for the numerical simulation). The first attainment of the maximum load oc-
curs on the contrary at different displacements (6 mm for the numerical simula-
tion 9 mm during the tests). The overall comparison is however satisfactory.

CODE IMPLICATIONS OF TEST RESULTS

a) Pier properties

The proposed code values of the shear referential strength \( \tau_k \) and of
pier ductility \( \mu \) are listed in tab. 1 together with the corresponding average
values derived by single pier tests. The table shows also the ratio of ex-
perimental to code values, they express the degree of conservatism for the basic
design parameters as assumed by the code under discussion. As to shear stren-
gths this degree is rather uniform for grouted and ungrouted masonry; it must
be noted however that the efficiency of grouting (and the corresponding \( \tau_k \) va-
ue) is widely dependent on the quality of the operation and on the diffusion

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of the cement mixture within the wall. This may suggest a more conservative level for it. The average value of \( \mu \) 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**

<table>
<thead>
<tr>
<th></th>
<th>ungrotured</th>
<th>grouted</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \tau_k ) (kg/cm²)</td>
<td>( \tau_k ) (kg/cm²)</td>
<td>( \mu )</td>
</tr>
<tr>
<td>code</td>
<td>0.70</td>
<td>1.1</td>
</tr>
<tr>
<td>tests</td>
<td>0.92</td>
<td>1.5</td>
</tr>
<tr>
<td>exp. val./code val.</td>
<td>1.31</td>
<td>1.37</td>
</tr>
</tbody>
</table>

The analysis is performed as follows, based on the simplified method described in the previsions section. For each model the ultimate lateral capacity \( C_{2e} \) is determined for different values of \( \tau_k \) and \( \mu \). 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 \( \tau_k \) when entered with the experimental lateral resistance \( C_{2e} \) will give the value of pier ductilities \( \mu \) consistent with the adopted numerical model. By interpreting test results in this way values of \( \mu \) ranging 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 experimentaluctilities determined by imposing only inplane forces without considering the simultaneous out of plane actions occurring during earthquakes on real buildings.

**b) Response modification factor**

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 \( C_{1e} \) is as follows (for low natural periods):

\[
C_{1e} = \frac{F_{A}}{q}
\]  

(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 $q = 2.93$ is thus estimated. Similar values ($q = 2.8 \pm 3.05$) 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 ($T_0 = 0.135$ sec.). The estimate of the earthquake forces $F_e$ is made by using a fraction $\alpha$ of the total weight with the first mode only. By calling $S_a$ ($T_0$, $\nu$) the spectral acceleration corresponding to the fundamental period and to damping $\nu$ this leads to:

$$F_e = \alpha S_a (T_0, \nu) \cdot M_{tot}$$

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

$$\alpha = \frac{0.017}{T_0} + 0.686$$

A value $\alpha = 0.812$ 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 $v$, $T_0$ and C1x) 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, together 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 simultaneous 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 earlier (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 C1code and C2code depending on whether they refer to significant yield or to ultimate. They are compared in tab. 2 with the corresponding experimental quantities. The ratio $C_x/C_k$ will be termed overdesign factor (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 excitation
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>
<thead>
<tr>
<th>( q )</th>
<th>1.</th>
<th>1.25</th>
<th>1.5</th>
<th>1.75</th>
<th>2.</th>
<th>2.25</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A ) (g. units)</td>
<td>0.127</td>
<td>0.159</td>
<td>0.191</td>
<td>0.223</td>
<td>0.254</td>
<td>0.286</td>
</tr>
<tr>
<td>( A ) (g. units)</td>
<td>0.149</td>
<td>0.186</td>
<td>0.224</td>
<td>0.261</td>
<td>0.298</td>
<td>0.335</td>
</tr>
</tbody>
</table>

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 \( C_{2x} \) used by code 2 for strengthened buildings in damaged areas of Southern Italy. By assuming \( C_{2x}/C_{1x} = 1.1 \) (as suggested by the numerical analysis of the buildings considered herein) and \( q = 1.75 \), equation 2 gives the EPA 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 \( \tau_k \) and \( \mu \) for plain stone masonry are consistent with the experimental 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.
REFERENCES

1 - Technical recommendations for repairs "Regione Friuli-Venezia Giulia" DT-2, 1978


7 - ATC 3-06 NBS Special publication 510, U.S. Govt. printing office, 1978.

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