ASEISMIC DESIGN AND BEHAVIOUR OF REINFORCED HOLLOW CLAY BRICK MASONRY SHEAR WALLS

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ABSTRACT

A research program to study the seismic behaviour and suitable design criteria for hollow, normal and lightweight, concrete and clay brick or block reinforced masonry is presented.
The available, partially not concordant, experimental data and codes' provisions are reviewed.
The main purposes and the first results of the experimental and numerical investigations are described.

INTRODUCTION

Only in the last years the use of reinforced masonry (R.M.) as structural system for the construction of buildings in seismic areas, has been taken into consideration in Italy.
Research programs are beeing carried out to study the behaviour under seismic actions of R.M. built up with units just usually employed in unreinforced masonry constructions.
The present paper deals particularly with vertically or horizontally perforated bricks or blocks, with high percentage of holes (about 45%), by means of which remarkable thermal insulation capacity of the masonry walls can be reached.
The structural design of R.M. systems in seismic zones requires the following steps to be checked:
a) choice of the masonry structural system;

b) prediction of strength and inelastic behaviour associated with each mode of failure of walls under inplane forces;

c) prediction of the design forces.

The principal research works in the field are referred in (1,2,3,4,5, 6,7,8).

Modes of failure

Two fundamental modes of failure, shear and flexural, have been experimentally recognized.
The flexural mode of failure is generally characterized by the yielding of the vertical steel reinforcement followed by the crushing of the compressed toe. A desirable inelastic behaviour can be obtained with a suitable choice of steel percentage and, sometime, with the masonry confinement (4,7) Both for high and low steel percentages, especially with the higher values of normal forces, a brittle failure could be reached, particularly when hollow ungrouted bloks are used.
The shear mode of failure generally occurs when diagonal tensione strength (just like in unreinforced masonry panels occurs) is reached; nevertheless, particularly when hollow units of the type here considered are employed, a diagonal compressive crushing may be found (6).
The judgements on the inelastic behaviour of this type of failure are not concordant. Priestley recognized brittle shear failure on concrete masonry cantilever panels (3) and suggested to avoid it "at all costs", by means of capacity design approach (4). McNiven and Clough (5) observed a "less desirable" flexural inelastic behaviour in fixed ends piers, and recognized a positive effect of less amount of horizontal reinforcing steel than that corresponding to Priestley's suggested design procedure. Macchi (6), considering the tests results obtained on vertically perforated brick masonry, observed the occurrence of diagonal brittle crushing when too high percentage of steel reinforcement had been employed.
With regard to the shear strength, furthemore, there is not general agreement about the influence of the amount and the direction of the reinforcing bars. In the opinion of Meli and Esteva (1), only high percentage of steel, particularly horizontally placed, can improve both strength and ductility of masonry walls, but only slightly, probably due to the bad bond conditions of the bars in the mortar joints. McNiven and Clogh observed "a trend toward an increase" in the shear strength with increasing horizontal steel percentages form 0 to 0,2%. Greater amount seems not to give significant strength increase, and even to worsen the inelastic behaviour. Vertical reinforcement seems to give no appreciable improvement both in strength and ductility properties (5). Macchi, in the above mentioned research work (6), found relevant ductility values just with low steel percentages (0,07% in the horizontal direction and 0,09% in the vertical direction), near to the minimum

values suggested in most codes, and, furthermore, a positive influence of the vertical reinforcement. Turnsek (7) observed considerable strength increases with horizontally placed steel bars, especially when sufficient ductility properties are performed by the corresponding unreinforced masonry.

Structural systems

The three possible structural systems described in (4), corresponding to different inelastic behaviour under strong earthquake motions of reinforced loadbearing masonry buildings, are here considered:

1 - the single cantilever shear walls systems, in which the inelastic deformation only accurs at the base of the walls;

2 - the perforated shear walls systems, in which plastic, shear or flexural, deformations are concentrated in the piers of one, generally the lowest, storey;

3 - the coupled shear walls systems, in which firstly the spandrel beams enter in the plastic range, and then base hinging of the walls occurs.

The particular inelastic dynamic response, on which is based the choice of the design forces, strongly depends upon the expected failure mode of the structural elements, and, then, upon the design philosophy. Suitable numerical analyses of R.M. structures are not available, but some informations may be drawn, even if with caution, from the r.c. shear walls systems studies. E.g., inelastic dynamic analyses of suitable and rather complex structural models were conducted by Fintel and al. (9,10), Paulay and Taylor (11), Bertero and Mahin (12). In particular, Fintel and al., considering high rise building (more than 16 storeys), found little response reduction when the structure enter in the plastic ranges, with respect to the elastic response, both in single cantilever shear walls and coupled shear walls.
Priestley, on the contrary (4), consider for the cantilever shear walls systems a reduction of the elastic response equal to the local rotational ductility factor μ. In Priestley's opinion, ductility demand in structural model 2 is very large and strongly increasing with the number of storeys, and the model 3 is not to be considered owing to the practical difficulty to properly design masonry spandrel beams capable to withstand the expected ductility demands.

Design forces prescribed in codes

The large uncertainty and disagreement about the inelastic behaviour corresponding to the different modes of failure and to the non linear dynamic response of the different structural systems, are again found when comparing some codes' prescriptions regarding R.M. structural systems in seismic zones.
In the table 1 the "characteristic" (0.95 fractile) strengths corre-

sponding to the design forces prescribed in some different codes for apartment buildings in comparable seismic intensity level areas (peak ground acceleration 0,30÷0,35 g) have been evaluated as fractions of g applied to the mass of the construction.The safety coefficient for the allowable stresses prescribed in the UBC (13) and ATC (14) codes was assumed equal to 3.5.
The values given by the New Zealand code are variable depending on the axial load level (flexural strength) and the expected flexural overstrength (shear strength); furthermore, S=1 corresponds, to a value of the local ductility demand $\mu=4$, which means that wall panel must be able to execute 4 load cycle to such ductility level without losing more than 20% of the strength.For the structural systems 2 and 3 the code prescribes S factor values equal to 4 and 2.4.
It is worthnoting that the aim of the U.S. and N.Z. codes is to avoid shear mode of failure, with a quasi-elastic design procedure or a capacity design approach.Such limitation seems to be too severe (5,7),even if justified by the lack of sufficient experimental data.
The assessment of the actual behaviour of R.M. structural systems, considering the masonry type used in Italy,is the main objective of the experimental and numerical research works here described;the first result are presented in the following.

A RESEARCH PROGRAM FOR THE DEVELOPMENT OF SEISMIC R.M. STRUCTURAL SYSTEMS IN ITALY.

Materials, technologies and structural systems

The structural types here considered are bearing walls low rise buildings (e.g. 3÷4 stories) connected by flexible floor slabs,corresponding to the three structural system above described.
The masonry is built up with perforated, concrete or clay,normal or ligthweight,bricks or blocks.The perforation direction may be parallel or normal to the bed joints. The perforation direction may be parallel or normal to the bed joints.The compressive strength of this type of units is generally less than 10 MPa and greater than 5 MPa.
Masonry can be built traditionally or by assembling prefabricated storey height panels by means of grouted concrete joints.
The "diffused" reinforcement can be obtained with horizontal and vertical steel bars provided in bed joints and in special grouted pockets in the units (Fig.1).A concentrated reinforcement can be obtained,when block's geometry doesn't allow for vertical diffused reinforcement,by masonry confinement with vertical (and horizontal too,if necessary) r.c. elements included in special blocks. (Fig.1) (17).

Design criteria

A linear structural analysis is proposed,with respect to vertical loads W_i (at floor i) and static lateral loads

$$F_i = \gamma_i \, A \cdot R \cdot W_i / K \quad \text{where}$$

γ_i: is a distribution factor,depending onthe height of the floor measured from the building base;

A : is the peak ground acceleration (as a fraction of g) for the seismic zone of interest;

R : is the maximum elastic (5% damping) amplification factor;

K : is the behaviour factor,depending on non linear characteristics of the structural system.

As pointed out in the introduction,the main purpose of the research regards the rational choice of the K values justified by experimental data on inelastic characteristics of the structural components and on numerical non linear dynamic analyses of the different structural systems. The amount and the distribution of the steel bars will be principally devoted to the increase of the behaviour factor with respect to the low value suitable for unreinforced masonry walls systems,rather than to improve strength.

It is worth noting that according to some codes,a minimum amount of horizontal reinforcement is requested to carry all shear force (13;14), particularly in the hinging zone of slender wall panels(4),and that this is somewhat conservative with respect to some experimental results(e.g.7) More experimental results are required to define the optimum amount of vertical and horizontal reinforcement,taking account of:

- the contribution of the masonry shear strength in post-cracking stages;
- the contribution of the vertical reinforcement by arch action, particularly in squat walls;
- the possible shear strength degrading in the hinging zones.

A second group of design requirements will be related to the structural damages due to earthquake actions of different intensity and to the repairing conditions, considering in particular:

- vertical load bearing residual capacity;
- crack width and possible brittle failure of perforated brick walls (18);
- reinforcing steel failure.

Experimental techniques and first data

The strength parameters of the material and of the masonry assemblages are determined by means of standard tests:particularly uniform compression tests and concentrated diagonal compression tests (21).

The strength and,especially,the inelastic characteristics of the walls up to storey heigh are experimentally evaluated applying ciclic shear forces and compression forces.By means of the experimental purpose made equipment,fit for simulating contraflexure loading, (Fig.2) selected slow cycles of lateral forces or displacements can be applied to the panel bottom edge. .

The standard procedure corresponds to the application of three cycles for each value of the imposed maximum displacement,defining the enve-

lope curve of the maximum measured lateral forces and the strength degrading associated to the selected values of the ductility factor. Standard tests will be used to determine the residual compression strength. The testing programm will be applied also to repaired panels.
In fig.3 the diagram of the first test,on a lightweight concrete blocks masonry wall,is reported;several cycles of lateral forces were imposed to increasing values of the ductility factor.The envelope curve,and the strength degrading caused by the standard three cycles,can be derived to improve the numerical model parameters.

Numerical modeling

With reference to the structural systems 2 and 3,as previously described, within limites of the buildings here considered (low rise constructions with "regular" distribution of the walls) a suitable analysis of the non linear response can be obtained by means of a nonlinear single degree of freedom model,whose parameters derive from experimental results. A stiffeness and strength degrading model,well fitting the experimental data,is presented in fig.4,related to the adimensional parameter of force $f=F/F_y$ and the "ductility factor μ".The assumed strength degrading level DR is defined in the figure.It is worthnoting that increasing values of DR are obtained at each load reversal after reaching the maximum strength. First results are obtained with model parameters derived from experimental results (19) regarding unreinforced hollow clay masonry panels. The mean values of the maximum ductility demand,and of the attained strength degrading level DR,produced by ten generated accelerograms are reported in the upper and in the lower part of the fig.5.The different curves are related to different value of the design factor $\eta=F_y/MA$ which is the ratio between the maximum structural strength and the inertia force produced by the peack ground acceleration A.
In fig.6 the design spectra related to several ductility demands μ and strength degrading level DR are given.The results attained using models with and without strength degradation at load reversal are compared. It can be noticed thath the strength degrading model produces a lower design spectrum.It can be also noticed that,if a ductility demand $\mu=3.5$,or a strength reduction DR=30% could be accepted an even lower design spectrum would be available.Of course, the damage level attained by the structure,and its residual resistance to vertical loads, set up a limit to the available ductility.

Evaluation of the behaviour factor K

The results presented in fig.6 can be used to evaluate the suitable value of the behaviour factor K related to the allowable ductility .
E.g. if the ductility $\mu=2.35$,which corresponds to the beginning of the decreasing branch of the model,is wanted not to be exceeded,the value $\eta=F_y/MA=1.5$ will be derived from the design spectrum in the range of the periods interesting low rise buildings.Putting this value in the design equation $F_y=(R(T)/K)AM$,where the value $R(T)=2.5$ can be accounted

for in these range of periods, the value K=1.6 will be derived. (20) The numerical procedure can then be used to obtain, for the experimental data collected as previously described on each masonry type, the behaviour factor K taking account of both ductility and strength degrading.

In fig. 7a,b the typical time histories of f and DR parameters and its corresponding f,μ diagrams are presented. The number of cycles of strong amplitude, giving rise to the maximum ductility demands and to the maximum strength degrading, doesn't exceed 3 or 4, as it generally occurs. It can be concluded that, e.g., a value af K=1.5 seems to be suitable for the masonry type above considered, when 3 cycles with μ=2.35 and DR<20% can be experimentally obtained.

It seems so demonstrated that the proposed experimental tests will be fit for giving enough data in order to find out a suitable value for the behaviour factor of every masonry sistem.

As work hypothesis the following figures will be adopted: the K value 1.5 will be accounted if the test gets through the ductility $\mu=\mu_y$ with less than 30% of degradation; K=2 if the test gets over $\mu=2\mu_y$: K=2.5 if the test gets over $\mu=3\mu_y$.

A wider numerical investigation, accounting for the actual non linear characteristics shown by the test, will provide better specifications about this figures.

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TABLE 1

	Yugoslav (15)	UBC	ATC	New Zealand(16)
		Box Systems	Bearing Wall systems (R=3.5)	Cantilever Walls with $h_w/l_w>2$(S=1)
Flexural strength	0.2	0.67	0.29	0.21÷0.28
Shear strength	0.2	0.98	0.74	0.26÷0.39

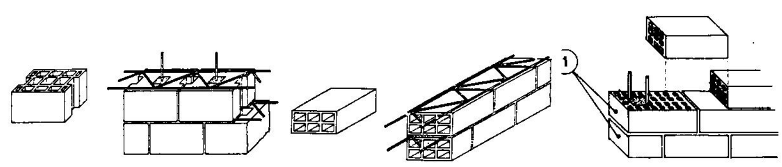

Fig.1 - Typical reinforced masonry assemblages to be used in Italy

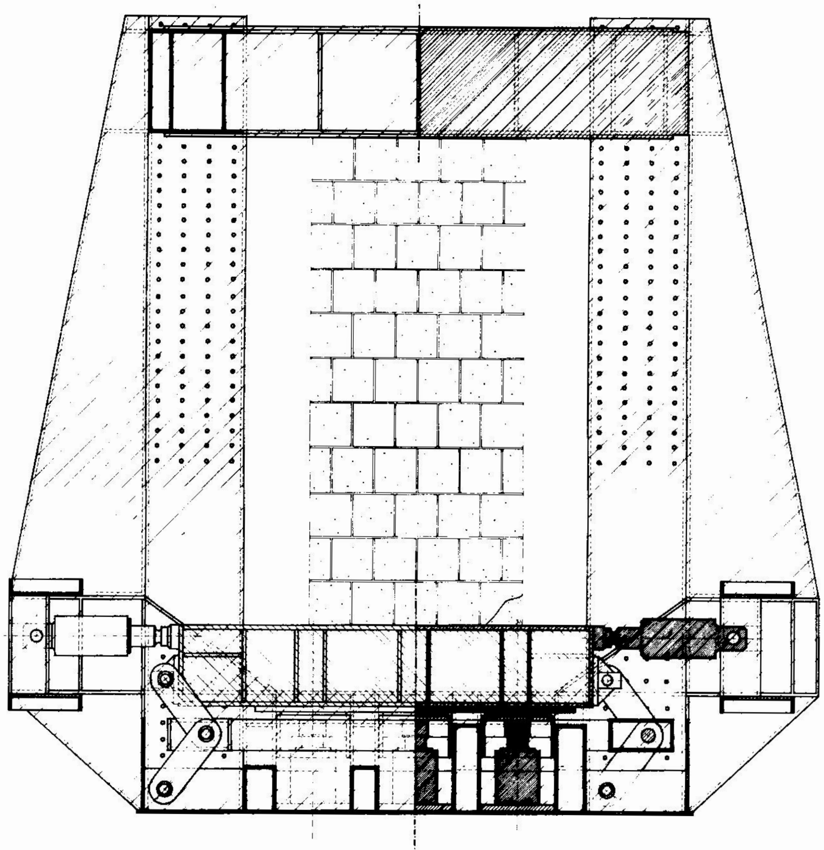

Fig.2 - Testing equipment for in-plane cyclic load tests in contraflexure

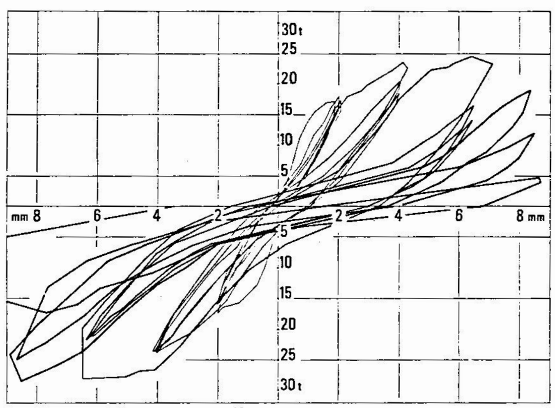

Fig.3 - Load deflection diagram for light weight concrete blocks reinforced masonry

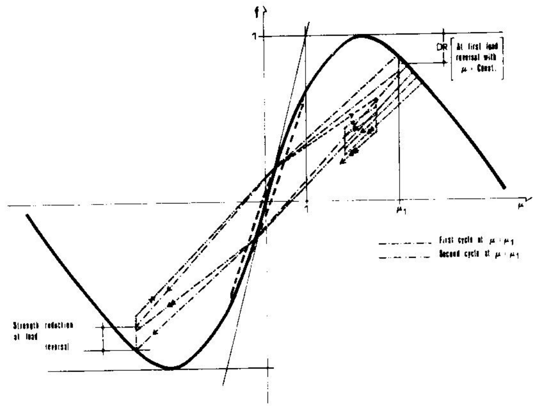

Fig.4 - Non linear degrading model

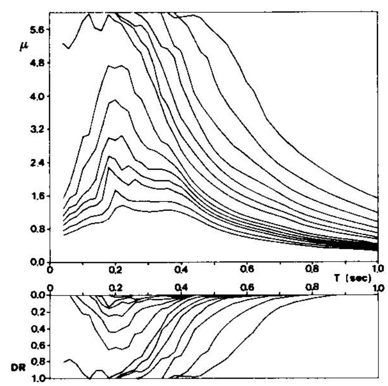

Fig.5 - Mean value of ductility demand (μ) and of strength degradation (DR) for several values of the design parameter $\eta = F_y/MA$

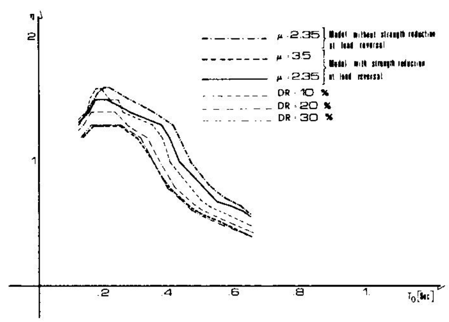

Fig.6 - Design spectra for selected values of μ and DR

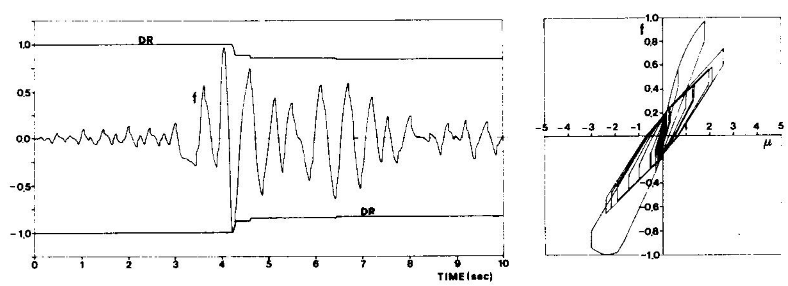

Fig.7 - Time history and f-μ diagram for a S.D.F. degrading model (T=0.3 sec., η=1,2)