REINFORCED HOLLOW CLAY BRICK MASONRY SHEAR WALLS UNDER SEISMIC ACTIONS

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SUMMARY

The main purposes and the first results of a research program on the seismic behaviour of reinforced hollow clay brick masonry shear wall systems are presented.Experimental and numerical analyses have been executed.Based on the results of tests carried out on panels subjected to vertical constant load and to cycles of horizontal,quasi-static,in-plane displacements or forces,an analytical model for non linear dynamic analyses has been used to evaluate the design forces and the corresponding ductility requirements for the walls.The overall structural behaviour and the design criteria of the considered structural systems are discussed.

INTRODUCTION

The employment of reinforced masonry structural systems has been considered in Italy too,in the last years, for the construction of buildings in seismic areas. A research program has been then promoted by the National Association of Clay Brick Manufacturers (A.N.D.I.L) to study the behaviour under seismic actions of reinforced masonry built up with units of the type just usually employed in unreinforced masonry constructions. In particular clay, normal or lightweight, vertically or horizontally perforated bricks with high percentage of holes (greater than 40%) are of interest, owing to the remarkable thermal insulation capacity they provide to the masonry walls.

The most frequent apartment building types built in Italy are regular, lowrise constructions. The possible structural systems that can be considered, corresponding to different inelastic behaviour under strong earthquakes are the following (Ref. 1, 2) :

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

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

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

High resistance and high ductility are required to the coupling spandrel beams respectively in system 2 and in system 3.

Considering that,owing to the architectural solution in actual buildings, a rather reduced masonry spandrel beam height is frequently obtained, and that there are some technological difficulties to properly detail the connection between the beams and the walls, it seems not possible to consider a reliable fulfillment of such requirements until sufficient experimental data on the subject shall be collected, not available at the present program research phase.

On the other hand , the coupling efficiency,with respect both to the strenght and to the ductility demands, of the r.c. floor slabs is often greater than the efficiency the masonry beams could provide, expecially considering that, in this case, the shear resistance is critical.

Moreover, inelastic dynamic analyses of coupled r.c. shear walls (Ref. 3) seem to indicate that little response reductions with respect to the elastic response are to be expected when the coupling beams enter in the plastic range.

It seems to be justified, at this research stage, to consider only the inelastic behaviour of the walls in order to evaluate the actual seismic response of a masonry wall system (Ref. 2, 4).

The coupling effect of the slabs, and eventually of the masonry beams, can be taken into account only for strength verifications.

DESIGN FORCES

As usual in modern codes for the design of structures in seismic zones (Ref. 5, 6), the design forces are obtained deviding the elastic acceleration response spectrum corresponding to the earthquake intensity that, with a given probability in a fixed time period, produces the collapse of the structure, by a behaviour coefficient wich takes into account the inelastic characteristics of the structure.

The behaviour factor, here called K, generally depends on the natural period of vibration of the structure and on the parameters defining its inelastic properties (Ref. 7, 8).

While many studies are available for r.c. structures (Ref. 8, 9), rather little experimental and numerical research work has been done on masonry structures , expecially taking into account the high variability in the various countries of the material properties and of the constructional solutions .Particularly discussed are the possibility to obtain ductile behaviours both in flexure and in shear modes of failure,and the influence of the amount and of the disposition of the steel reinforcement.

As a consequence, the available code provisions regarding the forces for the aseismic design of masonry structures are rather discordant (Ref. 2, 5, 9, 10, 11, 12).
Specific experimental analyses on walls built up with the materials actually employed, inducing both types of failure, on wich to base the analytical description of the observed inelastic behaviour for dynamic analyses, are then necessary.
The test methods, the experimental results on some of the masonry types used in Italy, the analytical model and the dynamic inelastic analyses wich have been carried out, are described and discussed in the following.

TEST METHODS AND EXPERIMENTAL RESULTS

The basic strength and deformability parameters of the materials and of the masonry assemblages are obtained by means of some standard tests (uniform compression test, diagonal concentrated compression test) and of a special test, this one executed to determine the strenght and, most important, the deformability characteristics of walls under in-plane vertical forces and horizontal cyclic displacements or forces. By means of a purpose made equipment (Fig.1), selected slow cycles of lateral displacements or forces can be applied to the bottom edge of panels up to storey height, subjected to constant vertical load. Flexure (Fig.2a), shear (Fig.2b) and mixed flexure and shear (Fig.2c) type of failure can be induced varying the edge restraint conditions, the panels dimensions, the amount and the disposition of the steel reinforcement and the vertical load intensity.
The tests were performed applying cycles of horizontal displacements of continuosly increasing amplitude. Two typical lateral force versus lateral displacement diagrams are displayed in Fig. 3a,b. Fig. 3a refers to a panel subjected to a vertical load corresponding to the mean value of the permanent action in a typical building (about 0.5 MPa), while Fig. 3b refers to a vertically unloaded panel.
The gross section of the panels is 1250x250 mm^2 ; the reinforcements percentages are : 0.076 % (f_y=440 MPa) in the vertical direction, and 0.090 % (f_y= 320 MPa) in the horizontal direction.
The compressive mean strength of the corresponding unreinforced masonry is 5.5 MPa.
In panel SP1 (Fig.2b and Fig. 3a) a shear failure was obtained ; in panel SP8 (Fig. 2c and Fig. 3b) a mixed flexure and shear failure was observed.
Comparing figures 3a and 3b the following remarks are of interest. The available ductility is comparable in the two cases, but when flexure mode of failure occurs a less stiff and more stable behaviour is obtained.
When only shear mode of failure occurs, the maximum strength is reached after a phase of progressive diagonal cracking wich produces a material degrading, controlled by the reinforcing bars, but producing a subsequent more pronounced strenght reduction beyond the displacement corresponding to the maximum strength.

NUMERICAL ANALYSIS

Analytical model

The analytical model employed to simulate the cyclic diagrams lateral force (F) versus lateral displacement (d) experimentally obtained is shown in Fig. 4. A dimensionless representation (f, μ) has been derived (Fig.5) for parametric dynamic analyses;stiffness and strength degrading is taken into account,and a maximum strength reduction is accepted less than or equal to 30% of the maximum strength.
The four parameters characterizing the model are derived from the experimental results as shown in Fig. 4 and in Fig.5. First a trilinear envelope curve is drawn, then the values of the lateral force and of the lateral displacement indicated in Fig. 4 are derived,from wich the dimensionless parameters can be calculated (Fig. 5).
Generally the experimental diagrams are not symmetrical;then the parameters of the symmetrical model here used are otained averaging the corresponding values in the positive and in the negative regions.

Dynamic analyses

A single degree of freedom system has been employed, wich is considered to well fit the actual behaviour of the studied structural systems,as previously discussed. Ten generated accelerograms from USNRC response spectrum were used for step by step dynamic analyses.
The parameters P1 and P2 showed a very slight influence on the dynamic response,and P4 was assumed equal to 0.1μ (Fig.5) for μ greater then μ_c.
The influence of the parameter P3 on the variability of the K factor,usually considered function of T and μ (Ref.7),is shown in the figures 6a,b,c,d and figures 7 a,b,c,d,e.
Values of T greater than 0.5 sec. were not considered,owing to the general stiffness characteristics of the buildings to wich we are referring.
Some well known characteristics of the inelastic response of SDOF systems are self evident in the figures.I.e.,the K factor increasing rate progressively reduces with decreasing T values;as a consequence,the P3 influence becomes slight at the lower T values. Farthermore, in the considered range of T values,K very seldom exceeds μ.
The particular characteristics of the model response,to be connected probably to the proposed method to evaluate the ultimate strenght and the available ductility, can be easily observed too,comparing the obtained curves with two typical laws, derived from models usually employed for r.c. structures analysis, wich give,in the low range of T values , the K factor as function of T and μ :

a – the Newmark-Hall law for the acceleration region $K = \sqrt{2\mu - 1}$,independent from T (Fig.6 and 7);

b - the law suggested in (Ref.7) $K=1+2\,(\mu-1)^{0.87-0.5\,T_o}\,\frac{T}{T_o}\,(1-0.5\,\frac{T}{T_o})$; in Fig.6 $T_o=0.5$ sec. was assumed.

The first could be acceptable only for T values greater than 0.3 sec.; elastic dynamic analyses of typical three storeys buildings gave values of the natural period of vibration about 0.2 sec. (Ref.13).

The second law would be unsafe ,particularly for high μ values,except when P3=1.

A proposal is now in discussion in Italy to assume for reinforced masonry structures $K=0.5\,\mu$ (Ref.14),wich seems to be more reliable,but for the lowest values of T and P3 (curve c in Fig. 6 and Fig.7).

As to P2,wich indicates where cracking occurs when an hardening behaviour is observed, and wich does not influence the value of K,it could be an interesting factor when considering the damage of the structure.

At this stage of the research,the problem of the definition of the acceptable damage level is not sufficiently investigated,particularly taking account of the available techniques and of the costs of repairing. The proposed limiting to 30% of the strength degrading is a rough estimate of the acceptable damage level.

It is interesting to observe, moreover, that when considering higher strength degrading,higher K values are obtained.In Fig.8 the increase of K corresponding to a strength degrading level equal to 50% of the maximum strength is shown;it is worthnoting that to each value of the P3 factor correspond very different values of the required ductility.

CONCLUSIONS

The experimental tests carried out on reinforced hollow clay brick masonry walls demonstrated a reliable cyclic behaviour for aseismic constructions, showing a considerable and stable inelastic phase, both in flexure and in shear mode of failure.

A procedure to evaluate the behaviour factor K (by wich reducing the linear elastic acceleration response spectrum to take into account the actual inelastic behaviour) with a direct reference to the experimental results on wall panels, is proposed.

The analytical description of the experimental behaviour is obtained by means of four parameters,of wich only one showed a remarkable influence on the K factor values.

The obtained numerical results of the dynamic analyses evidenced that the rules generally employed to evaluate the K factor for r.c. structures can be unsafe for the considered reinforced masonry structures.

Further investigations are required particularly to define acceptable damage levels, and their correlation to the model parameters.

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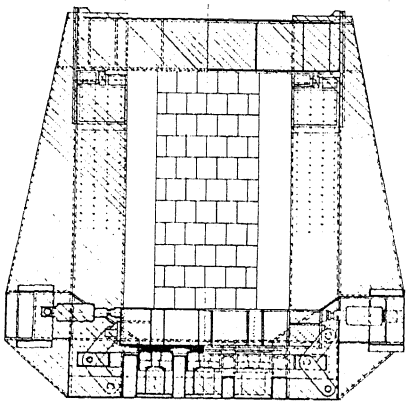


Fig. 1 : Testing equipment for in-plane cyclic lateral loads or displacements and constant vertical load tests

Fig. 2 : Wall panel SPHPF1 failed in flexure mode

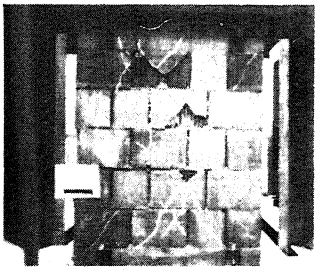


Fig. 2b : SP1 wall panel failed in shear mode

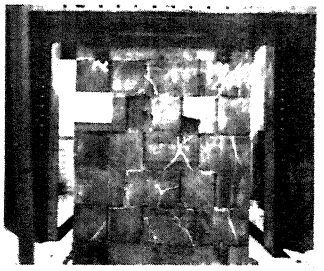


Fig. 2c : SP8 wall panel failed in a mixed flexure and shear mode

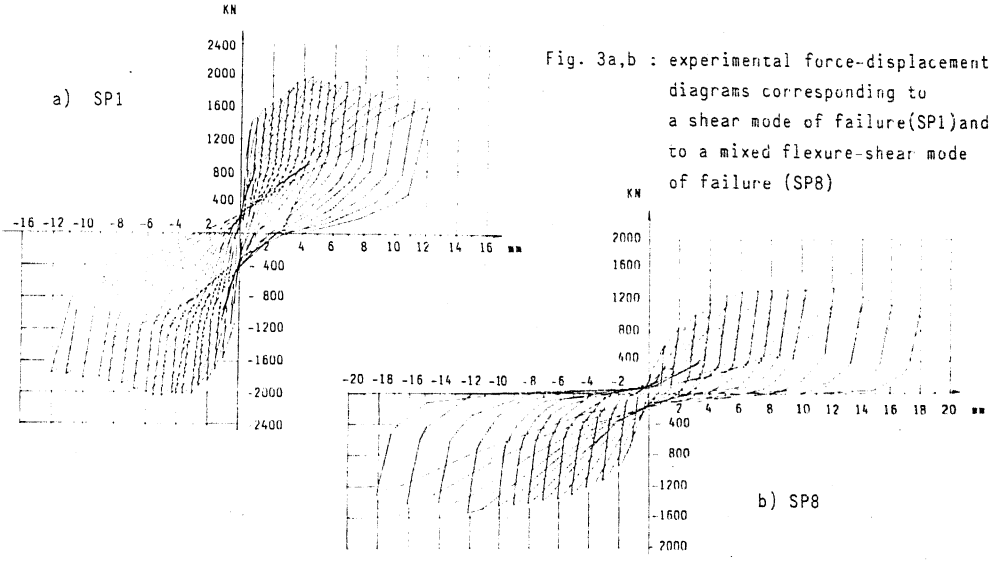


Fig. 3a,b : experimental force-displacement diagrams corresponding to a shear mode of failure(SP1)and to a mixed flexure-shear mode of failure (SP8)

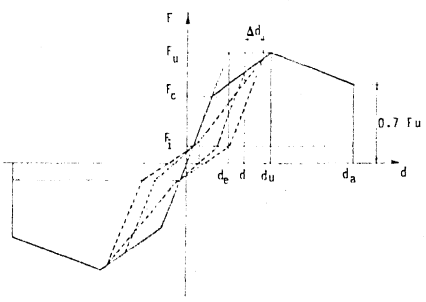


Fig. 7 : analytical model assumed to represent the experimental results

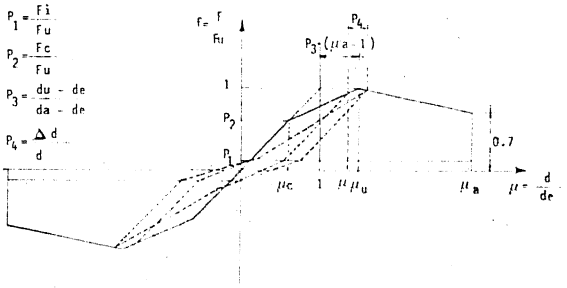
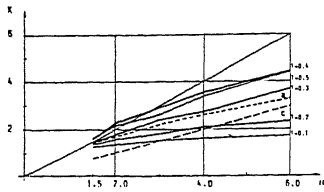
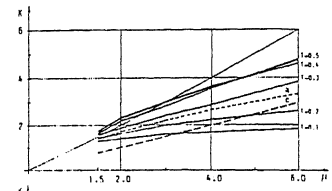


Fig. 8 : analytical dimensionless model used for numerical analyses

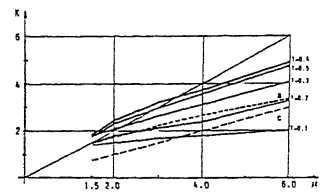
a)
P3=0.25



b)
P3=0.50



c)
P3=0.75



d)
P3=1.00

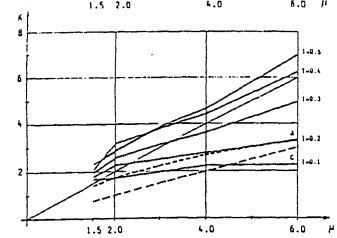
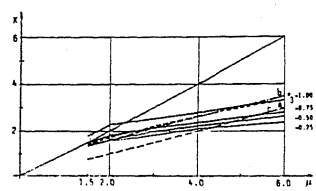
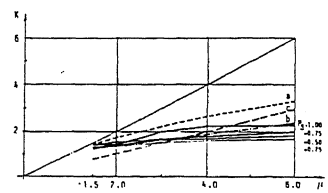


Fig. 6

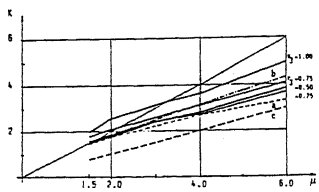
a)
T=0.1 s



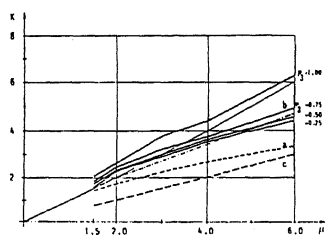
b)
T=0.2 s



c)
T=0.3 s



d)
T=0.4 s



e)
T=0.5 s

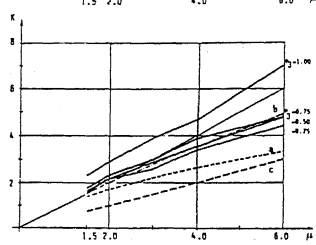


Fig. 7

Fig. 6,7 : K-μ diagrams numerically obtained for different values of T and P3, compared with the rules of Newmark-Hall(curve a) and of references (7) (curve b) and (14) (curve c)

Fig. 8 : relative increments $\Delta K=(K_{50\%}-K_{30\%})/K_{30\%}$ of the K values, corresponding to a strength degrading increase from 30% to 50% of the maximum strength