BEHAVIOUR OF MASONRY WALLS UNDER LATERAL LOADS

by

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SYNOPSIS

Results of tests on full scale masonry panels subjected to lateral loads in one direction and to cycles of alternating lateral loads are shortly presented. Walls incased in concrete frames, walls with concrete tie columns and interiorly reinforced walls were studied. Strength, stiffness, modes of failure and the postcracking behaviour are discussed. Methods for the prediction of behaviour based on mechanical properties of masonry are proposed and typical values of the main properties are given for different cases. Deterioration of cracked walls due to the repetition of loads depended mainly on the type of failure and on the type of reinforcement. Models of the hysteretic loops are proposed and experimental values of the parameters are given. General recommendations about the seismic design of the different types of masonry walls studied are also formulated.

INTRODUCTION

Seismic behaviour of masonry construction has been very frequently unsatisfactory and it is often unfavorably compared with the performance of steel and concrete structures. Most catastrophic failures have been of unreinforced masonry buildings with obvious structural defects, such as lack of proper joints, scarcity of walls in one direction and asymmetric layout of resisting elements. This seems to indicate that very often masonry construction is not subjected to a careful structural design as is common practice for concrete and steel structures. On the other hand, although it is true that masonry walls are generally very stiff and brittle elements with low resistence to seismic action, specially for shallow nearby earthquakes, it is also true that with proper reinforcement and confinement quite large deformations can be accepted by masonry with some cracking but without collapse.

To properly specify design seismic forces based on the ductility and energy absorption of reinforced masonry systems, behaviour under lateral loads must be studied, specially the postcracking behaviour and the effect of alternating loads. A long range experimental programme has been carried out at the Institute of Engineering of the National University of Mexico with that aim. Tests of full scale wall panels subjected to lateral load acting in one direction and to cycles of alternating lateral loads were performed. To study the mechanical properties of masonry and the modes of failure under elementary actions, series of tests on small specimens were also carried out.

Detailed progress reports of experimental results have been already published.[1,2,3] The complete report of a first phase of the study on infilled frames made by Esteva has also been published.[4] The objectives of this paper are to make a short presentation of the experimental results and to outline the conclusions, relative to seismic design, resulting from research and from the observed behaviour of masonry structures in recent earthquakes.

MECHANICAL PROPERTIES OF MASONRY

The aim of this part of the project was to identify those mechanical properties that most affect the behaviour of masonry under common types of actions, to find simple tests that could give indexes of these properties and to try to predict the structural behaviour of masonry from the results of these simple tests.

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Tests on small assemblages of masonry units subjected to axial compression, diagonal compression and shear were performed. Detailed results are reported in ref 3. To study the effect of axial compression, piers with a height to width ratio of approximately 4 were tested (fig 1a). The effect of tensile stresses in different directions and of shear stresses in the joints was studied with diagonal compression tests on small panels of differents shapes. For square panels (fig 1b), failure was usually through the joints, except for very weak units where a pure tensile crack caused the failure. For specimens with different shapes the mean shear stress at failure increased with the height to width ratio, due to the higher compressive stresses normal to the joints. To investigate bond and friction in the joints shear tests of joints with different levels of precompression were carried out, as indicated in fig 1c. A linear variation of strength with precompression level was found, defining a friction coefficient almost independent of the type of mortar. Bond strength was very variable depending on the type of mortar and of unit. Attempts to predict, using simple formulas, results of diagonal compression tests from bond and friction coefficients obtained in shear tests were made, but for some type of units results were quite different from predicted values.

Mean values of the different properties obtained for more usual types of units are recorded in table 1.

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Test programme. Walls of approximately 2 x 2 m built on stiff concrete beams were tested as cantilevers (fig 2a). Concrete blocks and hollow and solid clay bricks were the main types of unit studied. Types of reinforcement used were: interior reinforcement cast in the holes of the units and tie columns and bond beams of the same thickness as the wall. Vertical load was also preimposed on walls in several cases. A few tests were also performed subjecting the wall to diagonal compression, as in fig 2b. The number of walls tested was 56.

Experimental behaviour. The behaviour of walls tested as cantilevers is described in the following. Walls encased in concrete frames behaved as monolithick elements for small loads until separation occurred in the lower tensile corner and later also in the opposite corner. Major stiffness reduction was due to progressive flexural cracking in the frame or in the wall itself. Subsequent behaviour depended on the type of failure.

For walls with low vertical reinforcement and low vertical loads, failure was governed by flexure and behaviour was similar to that of an underreinforced concrete beam. Extensive flexural cracking occurred (fig 3a) and strength was limited by yielding of the reinforcement followed by a rather long plateau (fig 4a). Failure was finally due to crushing of the compressive corner or to rupture of the extreme bars. Precompression on the wall caused an increase in strength but for high vertical stresses, behaviour tended to change to a brittle shear failure.

For high reinforcement ratios failure was governed by shear. A relatively high stiffness was maintained until a diagonal crack occurred at angular deformations between 0.001 and 0.002. The crack formed generally through the joints. The diagonal crack crossed the units only for low strength units or for high precompression loads. For walls with interior reinforcement load increased after cracking depending on the amount of reinforcement. If there was no precompression on the wall, maximum load could be maintained for high deformations due to the friction in the crack and to the dowel action in the reinforcement, and behaviour was rather ductile. On the contrary, for high vertical loads failure was extremely brittle (fig 4a).

For walls with exterior reinforcement, post-cracking behaviour depended mainly on the strength, stiffness and ductility of the exterior columns. Such walls can be modeled as two adjacent rigid triangles that can take lateral load by friction and interlocking depending on the confinement provided by the frame. High concentrations of stresses occurr in the corner of the frame. A weak reinforcing element fails in shear for loads slightly higher than cracking load but it is still able to give proper confinement and maintain that load for high angular deformations (up to 0.01). A strong column whose shear strength in the corner avoids the propagation of the diagonal crack into the column, causes deviation of the diagonal crack to a more horizontal position (fig 3d). The system thus works as short columns pushed by the two masonry blocks (fig 5). Strength is in this case higher than the cracking load and is limited by bending failure of the short columns. Very large deformation can be taken by the system if proper longitudinal and transverse reinforcement is provided. Crushing of masonry of low strength units can reduce ductility.

When vertical precompression was applied to walls with exterior reinforcement both cracking and maximum load increased and ductility decreased but less drastically than for walls with interior reinforcement (fig 4b).

Prediction of strength. Strength and general behaviour of walls with bending failure could be satisfactorily predicted with usual hypotheses for reinforced concrete. For usual reinforcement ratios, simplified procedures give reasonable accuracy. Such procedures may consist in considering that the resultant of compressive interior forces is located in the center of the extreme reinforcement on the compressive side, and that all the remaining reinforcement is yielding at maximum load.

Shear strength was more difficult to predict. Cracking load was found to be less variable than maximum load, for which the failure mechanism depends on more factors. Contribution of the exterior reinforcement to the cracking load was found to be negligible and prediction was based on the properties of masonry as determined in tests on small assemblages. The property that could best be related with wall strength was the average shear stress obtained in diagonal compression tests of small square panels (fig 1b). For walls tested in diagonal compression, average shear stress at diagonal cracking was found to be 85% of that obtained in small assemblages. The reduction can be attributed to the effect of confinement introduced by the loading plates in test of small panels and to the greater number of possible cracking trajectories in the walls. For walls tested as cantilevers, flexural stresses reduced shear strength and the ratio between mean shear stress in walls at diagonal cracking and that obtained in small tests was 0.5 in the average. Many attempts were made to relate wall strength with other mechanical properties of masonry. For low strength bricks, where the diagonal crack crossed the units, shear strength was found to be proportional to the square root of masonry compressive strength (the ratio was 0.8 for diagonal compression and 0.55 for cantilever tests). When cracking was through the joints, expressions in terms of bond and friction between mortar and units were found,[2] but their accuracy is very poor.

Strength was increased by the addition of reinforcement in intermediate holes, due to the effect of grout cast in holes and of the steel reinforcement. No general methods could be found for the prediction of those contributions; reinforcement seemed to be more effective for concrete blocks than for clay bricks due to the poor bond between clay and grout in the last case. The increase in cracking load due to precompression was found to be approximately 40% of the total vertical load applied. This result is limited to vertical load not exceeding one third of the wall capacity.

Prediction of the load-deflection curve. To describe the load deformation behaviour a trilinear relation was chosen (fig 6). By changing the parameters of this curve a wide range of cases could be covered. The stiffness of the first branch is not supposed to represent the initial uncracked stiffness of the wall, but an average stiffness before diagonal cracking or severe flexural cracking. Such a stiffness can be calculated with reasonable accuracy by ordinary strength of materials methods considering cracked transformed section for bending deformations and only the contribution of masonry for shear deformations. Remaining parameters defining postcracking behaviour and ductility depend mainly on the type of failure, reinforcement and vertical load. Experimental values for typical cases are reported in table 2.

As can be inferred from the table, for bending failure ductility ratios exceeding 4 were obtained even for relatively high precompression. For shear failure of interiorly reinforced walls, ductility ratios were between 2 and 3, while for walls with tie columns, ductility ratios were always higher than 4. In every case precompression increased the initial stiffness of the wall due to the reduction of flexural cracking.

BEHAVIOUR UNDER ALTERNATING LOADS

Test programme. Cantilever and diagonal compression tests were performed on 3 x 3 m panels alternating the direction of load. In some cases the maximum deformation of each cycle was kept constant and 12 to 60 cycles were applied. In other tests, the maximum deformation was progressively rised until failure occurred. Tests were mostly on concrete block walls with interior reinforcement (26) and also on brick walls with tie columns (4 tests). Results of 28 tests on walls with concrete frames reported in ref 4 are also considered.

Experimental behaviour. When subjected to cycles of alternating loads that cause cracking, walls suffered deterioration of stiffness and strength. The load-deformation curve changed significantly from the first to the second cycle; in most cases the curve tended to a stable pattern in subsequent cycles and difference after the sixth cycle were neglegible. The amount of deterioration depended mainly on the type of reinforcement and on the mode of failure. The type of unit and the vertical load applied affected also the behaviour.

Concrete block walls whose failure was goberned by bending showed little deterioration before yielding of the reinforcement; after yielding important reduction of stiffness occurred in subsequent cycle but strength was not affected (fig 7a). For high deformations progressive crushing of the unconfined compression corner gave rise to major deterioration.

When strength was governed by diagonal cracking the hysteretic loop was characterized by an initial branch of very low slope, corresponding to the closing of the cracks due to the load applied in the opposite direction, followed by a branch of higher stiffness similar to that of the first cycle in the cracked stage. For that reason the load was considerably lower than that of the first cycle for the same deformation, although the initial strength was usually reached for higher deformations. Behaviour was significantly influenced by the type of reinforcement.

Walls with interior reinforcement only, whose failure was governed by shear, showed very important deterioration after diagonal cracking (fig 7b). Often stabilization of the curve did not take place and initial strength could not be attained again, due to the progressive shearing off of the interior reinforcement. Increasing the amount of interior reinforcement did not give rise to a clear improvement of the behaviour.

Walls with tie columns and bond beams deteriorated after diagonal cracking much less than interiorly reinforced walls due to the confinement provided; nevertheless for high deformations, exceeeding that of maximum load, shear failure of the tie column progressively reduced confinement and very important deterioration occurred (fig 7c).

Walls with concrete frames whose section and transverse reinforcement were sufficient to avoid propagation of the diagonal crack into the corner and to give rise to a failure mechanishm as in fig 5, showed a definitely better behaviour. Deterioration was small and strength could be raised to practically the initial maximum load even after more than 50 cycles of angular deformations exceeding 0.03. The former was only true for walls of solid units; for hollow masonry, local crushing and spalling of the shell of the units caused a gradually increasing deterioration and load capacity was greatly affected for high deformations.

When a low level of precompression (3 to 5 kg/cm^2) was applied to the walls deterioration decreased for all types of reinforcement and modes of failure.

Analytical description of the behaviour. Load-deformation behaviour under repeated loads can be represented by curves as those in fig 8. Experimental results justify the adoption of a single hysteretic loop after the first cycle. The most important characteristics of the hysteretic loop are the ratio between the area contained in that loop and that contained in the first cycle and the ratio between the loads corresponding to the maximum deformation for the two cycles. The first parameter is a measure of the loss in energy absorption capacity and the second of the strength deterioration. A constant value of these parameters for each branch of the assumed trilinear load deformation curve can be considered. For the first branch results justify the adoption of a non deteriorating elastic behaviour; for each of the remaining two stages values of the parameters calculated from test results for different types of reinforcement, modes of failure and units are shown in table 3.

For the non linear deteriorating systems proposed for masonry, proper factors have to be found to modify seismic coefficients or ordinates of the design spectra used in ordinary seismic designs that assume linear elastic behaviour. The former can be done with a step by step analysis of the linear degrading systems for the effect of a certain number of real of simulated earthquakes representative of the types of movement expected in the zone of interest. Comparing the strength needed by the degrading system to withstand the earthquake without failure with that of a linear system with the same initial stiffness, proper reduction factors can be found. Analyses of this type are presently being performed in another stage of the research.

CONCLUSIONS

In the following, general remarks on the seismic behaviour of different types of masonry shear walls are given based both on the experimental results and on the direct observation of the effect of earthquakes.

Structures of unreinforced masonry have a completely brittle type of failure and linear elastic behaviour has to be considered for the analysis of seismic effects. Use of unreinforced masonry in seismic zone is objectionable due to the experience of catastrophic failures.

For walls with interior reinforcement, where failure is governed by bending, behaviour is nearly elastoplastic with remarkable ductility and small deterioration under alternating load, except for very high deformations where important deterioration is caused by progressive crushing and shearing off of the compressive corner. If failure is governed by diagonal cracking, ductility is smaller and, when high vertical loads are applied, behaviour is frankly brittle. Furthermore walls with this type of reinforcement showed important deterioration after diagonal cracking. Possibly, behaviour could be improved by using much higher ratios of vertical and horizontal closely spaced reinforcement, or more effectively by confining grout and vertical reinforcement in the corners by means of closely spaced ties. In conclusion, for adequate behaviour under alternating loads, the layout, aspect ratio and reinforcement of walls have to be chosen, when possible, in such a way as to give rise to a bending failure.

For walls of solid units strengthened by tie columns and bond beams, large ductilities are reached in spite of important damage in the wall itself and in the column. Deterioration is still important for high deformations due to the loss of confinement caused by the progressive destruction of the column in the corner. Nevertheless behaviour is definitely better than that of unreinforced or interiorly reinforced walls as has also been made evident by their better seismic behaviour.

For walls working as diaphragms of strong concrete frames, large ductilities are attained and deterioration is small, if the column has the proper section and reinforcement to avoid propagation of the diagonal crack into the corner. The former is true only for walls of solid units; for hollow masonry crushing and spalling of shells causes great deterioration.

For walls acting as diaphragms in strong concrete frames, large ductilities are attained and deterioration is small when solid units are used; for hollow masonry, crushing and spalling of shells causes great deterioration. To avoid propagation of diagonal cracking into the corner of the frames it has been suggested[4] that the column section must be designed to withstand one half of the wall shear strength. Due to the peculiar stress distribution in the corner, shear strength of the concrete section can be assumed as twice that calculated with the formulas for diagonal tension of beams.

As a tentative recommendation resulting from the research, it is proposed that the seismic coefficients for linear elastic behaviour be divided by the following factors: for unreinforced masonry and for hollow thin walled units masonry 1.0; for reinforced masonry (of units with no more than 50% of voids and with thick shells) failing in shear, 1.5; for walls of solid units confined by tie columns, 2.0; for flexural failure and for walls of solid units acting as diaphragms of strong frames, 2.5.

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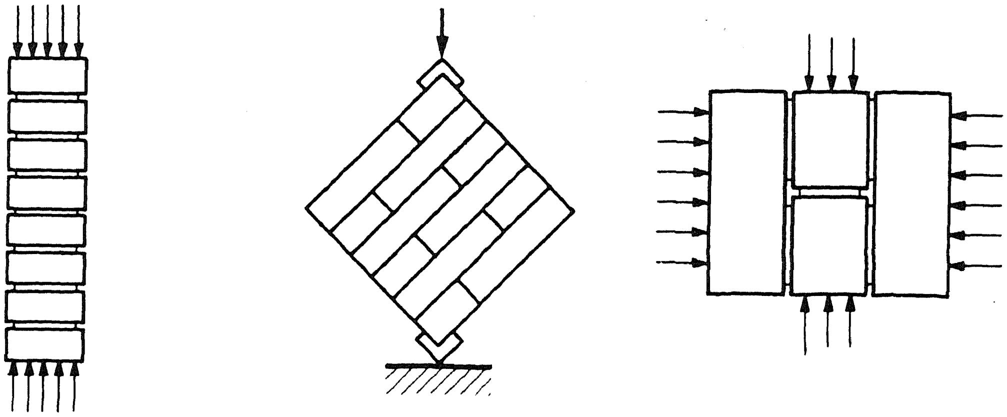
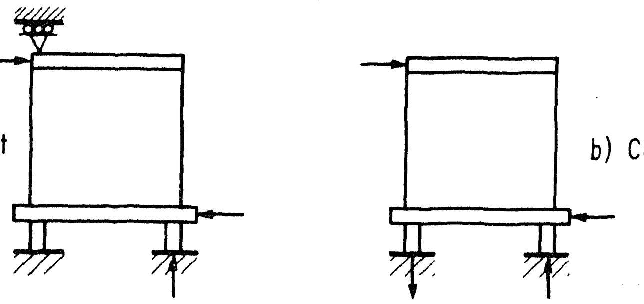


a) Axial compression test b) Diagonal compression test c) Shear test of joints

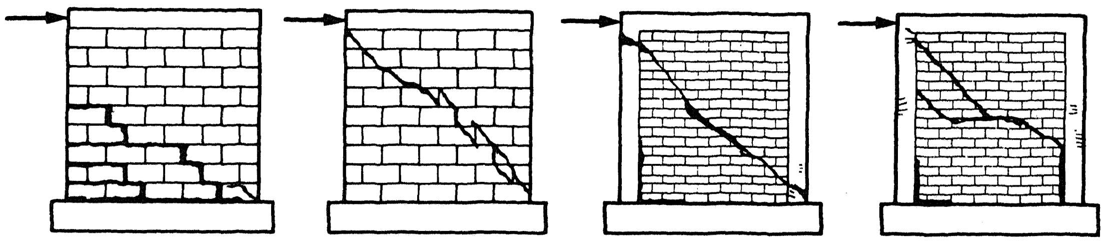
Fig 1 Tests for mechanical properties



a) Diagonal compression test

b) Cantilever test

Fig 2 Types of tests



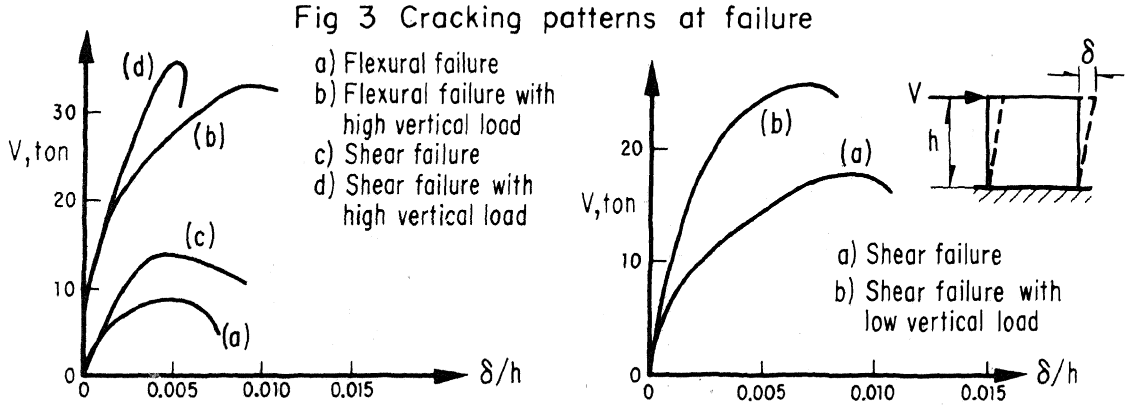
a) Flexural failure

b) Shear failure Interior reinforcement

c) Shear failure Ligth frame

d) Shear failure Strong frame

Fig 3 Cracking patterns at failure



a) Walls with interior reinforcement

b) Walls with frame

Fig 4 Load-deformation curves

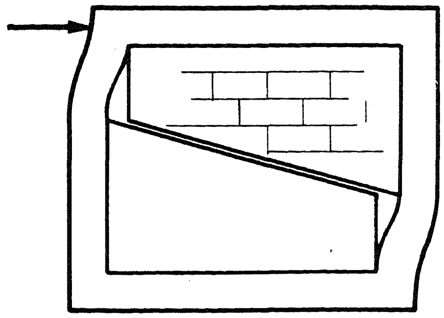


Fig 5 Mechanism of shear failure of walls with frame

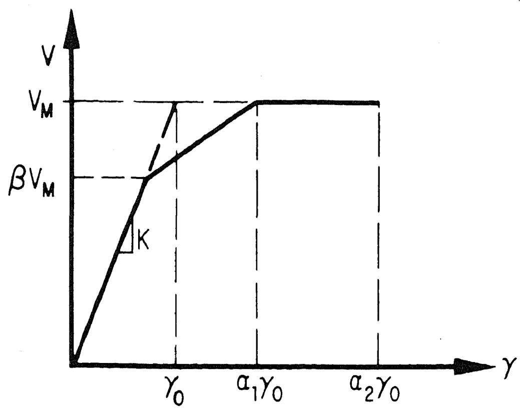
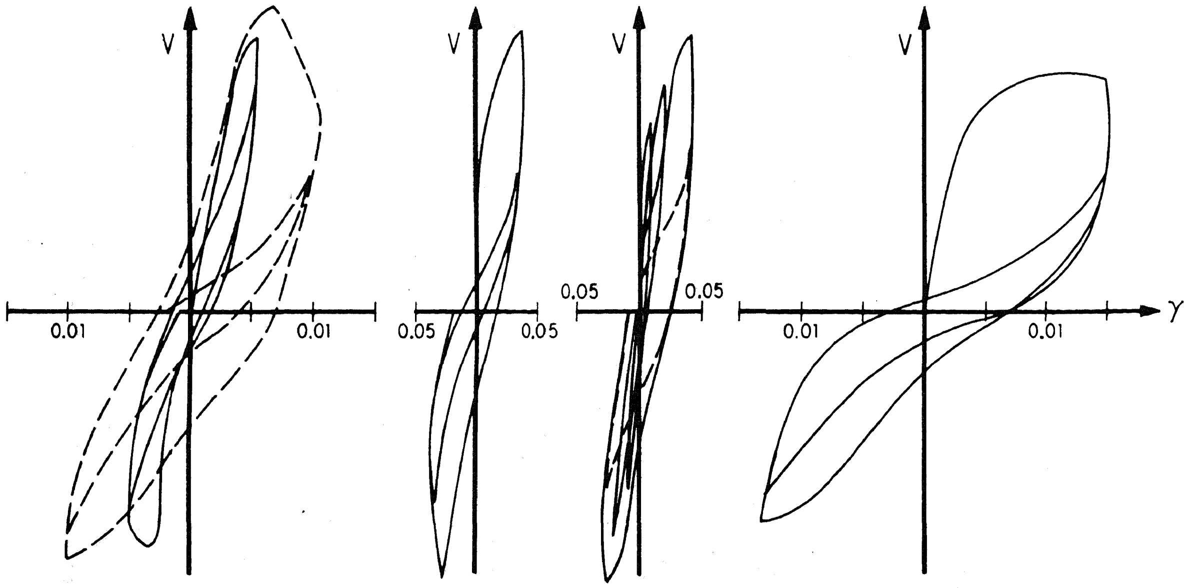


Fig 6 Proposed load-distorsion curve for masonry walls



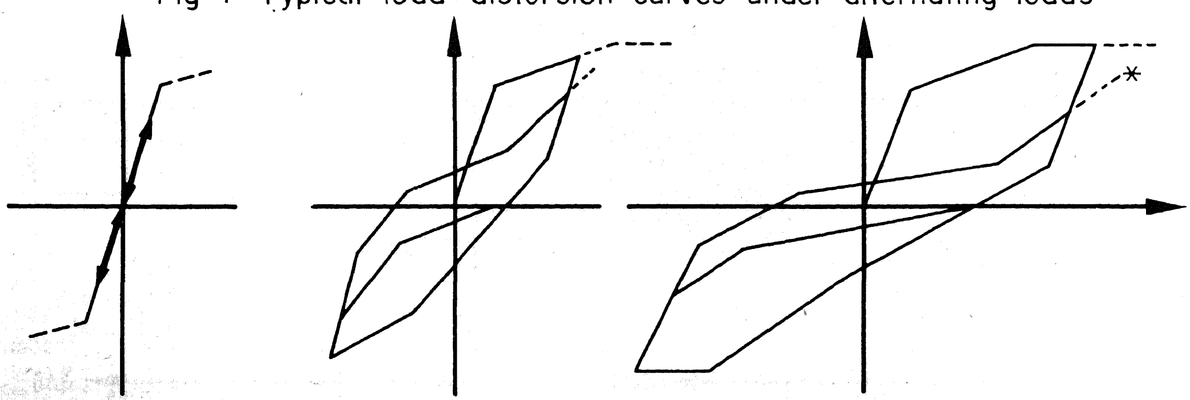
a) Flexural failure

b) Shear failure Interior reinforcement

c) Shear failure Solid brick with tie columns

d) Shear failure Strong frame

Fig 7 Typical load-distorsion curves under alternating loads



a) Elastic range

b) Post cracking range

c) After maximum load

Fig 8 Idealized hysteretical behavior

TABLE 1. MECHANICAL PROPERTIES FROM TESTS OF SMALL ASSEMBLAGES

Type of Unit	Compressive strength of Unit	Compressive strength of masonry	Shear strength	Bond strength	Friction coefficient
Common Brick (Burnt Hand Made Clay Brick)	60	35	5.5	2.5	0.75
Solid Brick (Notched)	150	65	7.0	1.2	0.7
Perforated Clay Brick	200	85	5.8	4.5	0.7
Hollow Clay Brick (Two round holes)	215	100	3.4	1.3	0.45
Hollow Clay Brick (Three rectangular holes)	150	90	5.0	1.6	0.7
Concrete Block	65	40	5.8	1.8	0.8
Sand Lime Brick	150	95	6.4	2.5	1.0

All stresses in kg/cm^2 over gross area. Shear strength determined in diagonal compression test on squared panel
Mortar Proportion 1 cement : 3 sand

TABLE 2. SUMMARY OF RESULTS OF WALLS TESTED WITH ONE APPLICATION OF LOAD

Reinforcement	Type of Unit	Other test charactheristics	Precompression kg/cm^2	Stength, ton: Cracking	Stength, ton: Maximum	Stiffness ton/rad	β	a_1	a_2
Interior Reinforcement	Concrete Block	Bending Failure	0	–	8.5	6,000	0.65	1.3	6
			3.5	–	14.4	12,000	0.45	2.2	7
			10	–	32.3	24,000	0.50	3.5	4
		Shear Failure. Only Extreme Holes Reinforced	0	8.7	13.5	5,200	0.65	1.5	2
			3.5	13.1	23	9,500	0.55	1.75	2
			10	18.4	32	17,000	0.45	2.5	3
		Intermediate Reinforcement Fully Grouted	0	11.3	20	10,000	0.6	1.75	3.2
			0	15.1	35	12,500	0.5	2	2.7
	Hollow Clay Brick	Shear Failure. Only Extreme Holes Reinforced	0	5.3	7.3	7,000	0.5	2	4.5
			6	10.9	13.3	14,000	0.6	2	4
			12.5	11.9	19.6	10,000	0.6	1.75	1.75
		Intermediate Reinforcement	0	8.9	12.5	7,500	0.5	2	3
Tie Columns and Bond Beams	Hollow Clay Brick		0	6.3	7.5	8,000	0.6	4	>6
		Intermediate Reinforcement	0	9.2	12.7	6,000	0.65	3	4
	Solid Brick	Shear Failure	0	5.2	8.8	5,500	0.7	2	–
			3	11.6	13.4	10,000	0.5	2	6
	Perforated Brick	Shear Failure	0	6.9	9.3	5,200	0.4	3	6
			3	11.9	13.7	6,500	0.5	1.5	4
	Hollow Clay Brick	Shear Failure	0	6.6	7.3	6,600	0.6	2.5	7
			3	12.3	12.3	13,500	0.6	2	5
	Sand Lime Brick	Shear Failure	0	4.6	5.8	4,000	0.5	2.5	4
			3	11.8	13.1	10,500	0.4	2	?

β, a_1, a_2 are the parameters of idealized load deformation curve (see fig 6)

TABLE 3. EXPERIMENTAL PARAMETERS OF HYSTERCTIC LOOPS

Reinforcement	Type of Unit	Type of Failure	Precompression	Maximum Deformation	V/V_o, %	A/A_o, %
INTERIOR	Concrete Block	Flexure	NO	0.0015	85	80
				0.0030	85	70
			YES	0.002	95	80
				0.005	95	70
		Shear	NO	0.0015	75	35
				0.004	40	20
			YES	0.0015	75	60
				0.0035	50	30
	Hollow Brick	Shear	NO	0.002	35	20
			YES	0.002	30	35
TIE COLUMNS	Solid Brick	Shear	NO	0.001	90	80
				0.003	70	40
			YES	0.001	100	80
				0.0025	100	60
FRAME (25 x 40 cm)	Solid Brick		YES	0.015	70	40

V/V_o: Ratio between the load at maximum deformation in the hysteretic loop and the load corresponding to the same deformation in the first cycle.

A/A_o: Ratio between the area contained in the hysteretic loop and the area contained in the first cycle.