SEISMIC ANALYSIS OF STRUCTURES WITH MASONRY WALLS

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SUMMARY

The paper summarizes analytical studies on seismic behavior of masonry walls, carried out for the Ph D thesis of the first author. Models of non linear behavior under alternating loads were formulated, single degree of freedom systems with the proposed models were analyzed under different strong motions and the response compared with elastic behavior in order to obtain inelastic spectra and reduction factors for elastic design spectra.

Masonry behavior in the linear and postcracking stages was modeled and failure criteria were established in order to perform finite element analyses leading to load-deformations curves for walls under monotonic lateral load until failure. The usefulness of analyses of this type is discussed and simple rules to determine lateral stiffness of masonry walls are proposed to be used in linear elastic seismic analyses.

INTRODUCTION

A long term research program on the seismic behavior of masonry walls has been carried out at the Institute of Engineering of the National University of Mexico. Results of the experimental program, reported by Meli (ref 1 to 3), show that with proper design a rather ductile behavior can be achieved in masonry structures, specially if shear failure is avoided.

Analytical studies were also carried out to obtain theoretical models of the observed behavior and to interpretate and enhance experimental results with the aim of proposing practical methods for the seismic analysis of structures with load bearing or infill masonry walls. This paper summarizes the results of the analytical studies that have been reported in greater detail in ref 4.

SEISMIC ANALYSIS OF MODELS REPRESENTING INELASTIC BEHAVIOR OF MASONRY WALLS

In practice, linear elastic behavior is assumed in seismic static or dynamic analysis of buildings, although it is admitted that considerable amounts of energy can be dissipated through inelastic excursions during an earthquake; this situation is taken into account in modern codes (ref 5, 6) by modifications of the elastic design spectra. When the hysteretic behavior of the structure can be assumed as elastoplastic, design spectra are obtained by dividing ordinates of elastic spectrum by factors that are directly correlated with the ductility factor. Hysteresis loops obtained in static alternating load tests of masonry walls show (ref 2, 7, 8) that, although fairly large ductilities can be attained in monotonic load test, hysteresis loops are characterized by large amounts of deterioration in stiffness, strength and energy-dissipation capacity. Inelastic behavior depends mainly on whether failure is governed by shear or bending, on whether units are solid or hollow, on the level of vertical compressive stresses on the wall, and on the type of reinforcement. Masonry can be reinforced either by horizontal

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and vertical steel bars, or by confining bond columns and beams, or by stiff concrete or steel frames as in infill walls.

A trilinear degrading model, as shown in fig 1, provides a rather accurate description of the wide range of types of behavior that can occur for different combinations of the aforementioned variables. The model is characterized by seven parameters that define three stages of behavior. Before cracking the behavior is assumed to be linear; between cracking and maximum load a slight degradation is considered, and between first yield and maximum deformation, a larger degradation of stiffness, and in some cases also of strength, occurs. Different combinations of variables affecting inelastic behavior can be summarized in the four cases shown in fig 1.

In the same figure values for parameters defining hysteresis loops are proposed based on the behavior observed in tests of ref 1 to 3. Case 1 represents walls with interior reinforcement failing in bending with levels of axial load not exceeding 30% of their compressive strength. The same parameters correspond to infill walls of solid units if the frame is strong enough to withst and shear concentrations that occur in the corners after diagonal cracking of the wall, ref 2. A rather ductile behavior is obtained in this case and large amounts of energy can be dissipated by fat hysteresis loops after yielding. Case 2 represents interiorly reinforced walls with shear failure with or without axial load; ductility and dissipation of energy are very low in this case. Case 3 describes behavior of walls reinforced by slight concrete tie columns and beams in the boundary of the walls forming a confining frame; this type of reinforcement is used in many countries and is very efficient to enhance seismic safety of masonry walls; this case covers the situation when bending governs failure and an excellent behavior is described by parameters of fig 1. Case 4 corresponds to walls confined by tie columns and beams when failure is governed by shear; it also represents infill walls of hollow units; the behavior is much less satisfactory than in case 3. As has been shown by tests and by observations of failures due to earthquakes, behavior of hollow unit unreinforced masonry walls is very poor due to crushing and spalling of the shells of the units.

The proposed trilinear models can be reproduced by superposition of two simpler elastoplastic models: Clough's stiffness degrading model and a model with total degradation as shown in fig 2. Inelastic seismic response of the proposed models was studied by means of step by step dynamic analyses of one degree of freedom systems. Records of three strong motions on firm ground were used: El Centro N-S, (1940), Managua (1972) and Acapulco (1965). Systems with initial periods between 0.1 and 0.8 seg were studied; this range covers most buildings whose main structural elements are masonry walls. Newmark's method was used with integration intervals between 0.001 and 0.01 seg depending on the system's period. The maximum load capacity (V_M) needed by the inelastic system such that its maximum deformation during the earthquake was equal or slightly less than its deformation capacity ($\alpha_2 \gamma_0$ in fig 1) was obtained by trial and error. This load capacity was compared with that of an elastic system with the initial stiffness of the inelastic model (maximum load V_0 corresponding to stiffness k_0 of fig 1) and for an elastic system whose stiffness is the secant stiffness of the inelastic model (V_1 corresponding to k_1 in fig 1).

Shown in fig 3a are the ratios V_M/V_0 and V_M/V_1 averaged for the three accelerograms studied and for model 1 defined in fig 1; also shown are the mean plus one standard deviation of the aforementioned ratios. It can be observed that values of V_M/V_1 are more uniform than those of V_M/V_0 and that their standard deviation is lower; similar results are obtained in the remaining three cases, thus showing that a more reliable representation of the inelastic response can be obtained if in the elastic analysis the secant stiffness is used. Fig 3b shows averages of V_M/V_0 for the four cases considered; it can be seen that when bending governs the failure (cases 1 and 3) larger reduction of elastic spectral ordinates can be allowed than when shear governs (cases 2 and 4). Results also show that reduction factor is almost constant for periods higher than 0.3 seg and that for lower periods a linear variation can be assumed, reaching a reduction factor of one for zero period systems, giving rise to the same type of representation adopted by the Mexican Code.

To recommend reduction factors for the analysis of a whole structure it must be considered that points shown are averages of the response to three strong motions and that for a given earthquake ductility demands can be much more severe; furthermore the reduction factor must represent the inelastic behavior of the whole structure thus requiring more stringent ductility demands for individual walls. On the other side, parameters assumed give rise to models that represent rather conservatively the experimental behavior of walls. Taking into account the aforementioned factors it is proposed that upper limits representing the average plus one standard deviation of the calculated points be considered for design. With this criterion inelastic design spectra are obtained by using reduction factors proposed in fig 3c for the four cases considered. Calculations were performed with strong motions recorded in firm ground; different, and probably more conservative results, would be obtained if records on soft ground were used.

INELASTIC FINITE ELEMENT ANALYSIS OF MASONRY WALLS

Non linear finite element analyses of masonry walls were performed for monotonic load up to failure. The aim was first to reproduce the behavior observed in the tests and then to cover a wider range of the variables in order to obtain estimates of initial and post cracking stiffnesses. Considerations made for modeling materials behavior in different stages of damage are summarized below.

Before cracking, masonry was considered an elastic material. Anisotropic behavior due to the layered construction can be easily considered with finite element methods. In this work the same value of Hooke modulus (E) was considered for horizontal and vertical directions, because tests on small assemblages have shown negligible differences; however shear modulus G varied between 0.1 and 0.3 depending on the types of unit, joint and mortar (ref 1). From the analysis of small square penels subjected to diagonal compressive forces it was found that the value of the G/E ratio significantly affects the stress distributions; in particular maximum tensile stresses 50 per cent higher than those corresponding to isotropic materials ($G/E = 0.4$) were obtained when G/E equals 0.1.

Elastic behavior of masonry stands until cracking or compression failure occurs. Experimental results (ref 1) show that the tensile strength of masonry varies from a minimum for stresses perpendicular to the layers to a maximum for stresses parallel to layers. A bilinear variation of tensile strength with the direction of stresses was assumed and, in order to predict the formation of a crack, the direction where the ratio between acting and resisting stresses was maximum was investigated. Very often cracks appear following the mortar joints; the applicability of Coulomb sliding shear criterion was also explored to represent this type of cracks, but better results were obtained with the above tensile stress criterion. After cracking masonry was considered as a no-tension material resisting stress only in direction parallel to the crack; shear stiffness was also annulled.

Compressive failures were also checked, comparing principal compressive stresses with the strength obtained in small masonry assemblages. In agreement with experimental results after compressive failures, spalling of the units was assumed; thus, all stiffness properties were made equals to zero. Steel reinforcement was considered able to support only longitudinal stresses, and an uniaxial elastoplastic behavior was assumed. To take into account its contribution to the stiffness, reinforcement was considered as uniformly distributed in the reinforced material.

A computer finite element program was written aiming at reproducing the lateral load-deformation curve and at following failure patterns of masonry walls. If any, vertical load effects were first analyzed; then small amounts of lateral loads were applied. For each increment the adopted failure criteria for masonry and reinforcement were checked in each element, using centroidal stresses. When a failure was detected, the corresponding changes in stiffness and stress redistribution were performed, and again, failure criteria were checked. When no failures occurred next load increment was applied.

Several experimentally tested walls were analyzed with the computer program. Analytical and experimental load-deformation curves are presented in fig 4 for a concrete block, interior reinforced wall tested by Meli (wall 508 in ref 3); required mechanical properties were taken from results obtained in small masonry subassemblages. The comparison of both curves shows that the analytical approach gives a good approximation to experimental results. However the cracking pattern obtained from the analysis was more evenly distributed that the experimental one, due to the continuous, and not concentrated, modeling of cracks.

Another computer program was written for the case of masonry infilled frames, allowing automatically separation or sliding between elements representing wall and frame, when tensile or excessive shear stresses occur. Then, by a reanalysis, it was checked if displacement compatibility requirements were met, to avoid overlapping between separated nodes. It was found that this procedure is not always convergent, and for this reason failure criteria for wall or frame elements were not incorporated in this program. An examination of analytical results allows to conclude that the no-tension material approach gives a good representation of masonry cracking. However, analytical procedures are yet limited because other failure modes, as bond deterioration, or other deteriorated states, as interface shear strength in cracks, cannot be properly represented with a practical approach.

Furthermore, an important amount of computational effort is required to have a good prediction of failure patterns, because great refinement in finite elements meshes is needed and the analysis must be made several times. Despite these limitations, finite element methods can be used to perform parametric studies of stiffness degradation and stress distributions corresponding to several stages of behavior.

LATERAL STIFFNESS OF STRUCTURES WITH MASONRY WALLS

As mentioned above, usually for seismic design purposes an elastic analysis of the whole structure is performed. Either static or dynamic methods are used to determine the effect of seismic actions. In this approach it is impractical to represent each wall of a building with several finite elements, due to the large amount of computational effort that would be involved. Then, it is necessary to have a macroscopic representation of masonry panels as equivalent single elements.

For uncraked elastic panels the accuracy of the wide column concept (a column in which both flexural and shear deformations are taken into account) has been checked by comparison with results of finite element analysis of isolated and more complex sets of masonry walls, and very good results were obtained (ref 4). Experimental results also confirm the validity of this approach(ref 2).

In the case of concrete frames infilled by masonry walls it is necessary to consider the reduction in the lateral stiffness due to the separation or sliding between frame and wall. The extension and pattern of these separations or sliding depend on the way the loads are applied to the frame-wall systems; however, a lateral load acting at the top of the frame can be considered representative because vertical loads and overturning moments are mainly resisted by the frame columns. With such a top load several infilled frames were analyzed with the finite element method, allowing separation or sliding between frame and wall when tensile or excessive shear stresses (0.3 times compressive stress) occurred in the interface. A typical resulting pattern is shown in fig 5a. Equivalent wide columns were defined so as to obtain the lateral stiffness computed with the finite element analysis. In these columns the flexural stiffness is given by the concrete columns ($EI = EA_c \ell^2/2$), and the shear stiffness by a masonry wall with reduced area A . Values of A , as a fraction Ω_0 of the total area ($\Omega_0 = A / 2A_c + A_m$) are presented in fig 5b in terms of the nondimensional parameter $E A_c / G A_m$. E and A_c are the modulus of elasticity and the cross-section area of the columns, and G and A_m are the shear modulus and the horizontal area of the wall; these properties are most important for the composite frame-wall behavior.

After its separation or sliding over the frame the infill wall works essentially as a compression strut; for this reason several authors have proposed the use of such a strut to represent infilled frames (ref 10); thus, equivalent widths, w_0 , for diagonal struts were also obtained. These strut widths were computed to provide the difference between the lateral stiffness of the infilled frame and that of the frame alone, and are presented in fig 5c, as a fraction of the wall height h, and as a function of the parameter $E A_c / G A_m$.

In the first part of this paper it is concluded that the use of a secant stiffness (k_1 in fig 1) allows a better representation of inelastic effects in seismic analysis of masonry structures. To follow this approach it is necessary to have simple procedures to evaluate that secant stiffness. An obvious form to do it, is to compute the initial stiffness k_0 with the procedures suggested above, and then reduce this value with factors k_1 / k_0 obtained from tests and given in the table of fig 1. Otherwise more refined calculations can be performed. For walls with interior reinforcement, whose behavior is governed by flexural failure wide columns concept can be employed, provided that transformed cracked sections are used to evaluate flexural stiffness. This method has been verified in this study by comparison with finite element results, and also when compared with experimental results good approximation was found (ref 2).

For infilled frames the secant stiffness can be considered as corresponding to the state in which a diagonal crack exist in the infill wall. Thus, the same infilled walls analyzed to study the initial stiffness k_0 , where reanalized considering a pre-existent crack in the wall's diagonal as shown in fig 6a. The no-tension material method was used to represent the cracks; separations or slidings between wall and frame were allowed where tensile or excessive shear stresses appeared. In fig 6b the ratio k_1 / k_0 is plotted against the parameter $E A_c / G A_m$; it can be noted that the values of that ratio are almost constant and very close to 0.67 which is the average experimental value given in table of fig 1 for this case.

The wide column and the diagonal strut approaches, can be used for calculating k_1 . The corresponding values of the reduced shear area, A_1 are presented in fig 6b as a fraction of the total area and in terms of $E A_c / G A_m$. Values of equivalente width w_1 are presented in fig 6c.

CONCLUDING REMARKS

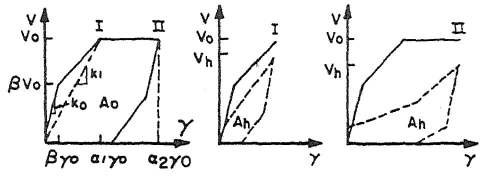
By combinations of simple well known models, rather complicated hysteresis loops for masonry walls were represented. Models proposed can be used to describe behavior of other structural elements with important stiffness or strength degradation, specially those whose behavior is governed by shear, as, for instance, in flat slab-column connections.

Non linear finite element analyses of masonry walls under monotonic lateral load used in this paper gave rise to good prediction of load-deformations curves and of cracking patterns. Nevertheless the numerical effort involved limits at present the usefulness of this approach. A better use of finite element analysis consists in parametric studies of stiffness and stress distribution for different stages of pre-established damage in masonry walls. The same conclusions apply to other planar structures as concrete panels or deep beams.

In practice, seismic analysis of building with masonry walls must be performed with simple idealizations such as wide column or diagonal strut. Equivalent shear areas or strut widths were obtained in this work to determine either initial or secant stiffness.

REFERENCES

1. Meli, R (1973), "Behaviour of Masonry Walls under Lateral Loads", Proc V World Conf on Earthquake Eng, Rome
2. Meli, R (1975), "Comportamiento sísmico de muros de mampostería", Publ No 352, Institute of Eng, Nat Univ of Mexico, Mexico
3. Meli, R and Salgado, G (1969), "Comportamiento de muros de mampostería sujetos a carga lateral", Publ No 237, Institute of Eng, Nat Univ of Mexico, Mexico
4. Bazán, E (1980), "Análisis sísmico de estructuras con muros de mampostería", Ph D Thesis, Grad School of Eng, Nat Univ of Mexico, Mexico
5. Departamento del Distrito Federal, (1976), "Reglamento de Construcciones para el Distrito Federal", Mexico
6. Applied Technology Council (1978), "Tentative Provisions for the Development of Seismic Regulations for Buildings", Nat Science Found Publ 78-8, Washington
7. Williams, D (1971), "Seismic Behaviour of Reinforced Masonry Shear Walls", Ph D Thesis, Univ of Canterbury, Christchurch, New Zealand
8. Mayes, R L, Omote, Y and Clough, R W (1976), "Cyclic Shear Tests on Masonry Piers, Volume I. Test Results", EERC Rep No 76-8, Univ of Calif, Berkeley
9. Cervenka, V (1970), "Inelastic Finite Element Analysis of Reinforced Concrete Panels Under In-Plane Loads", Ph D Thesis, Univ of Colorado, Boulder, Colorado
10. Stafford Smith, B (1962) "Lateral Stiffness of Infilled Frames", ASCE Struct Div Journal, Dec



First cycle Posterior cycles

Experimental parameters of hysteresis loops

Case	Reinforcement	Vertical load	Governing failure		V_h/V_o Stage I	V_h/V_o Stage II	A_h/A_o Stage I	A_h/A_o Stage II
1	Interior Frame (solid units)	Yes - No No	flexure —	0.6 2.5 7.0 0.67	1.00	0.30	0.80	0.50
2	Interior	Yes - No	shear	0.6 3.0 4.5 0.56	0.80	0.40	0.40	0.15
3	Tie colums	Yes - No	flexure	0.6 2.5 10.0 0.67	1.00	0.90	0.85	0.70
4	Tie colums Frame (hollow units)	Yes - No No	shear	0.6 3.0 6.0 0.56	0.90	0.70	0.70	0.40

Fig 1 Models of hysteresis loops for masonry walls

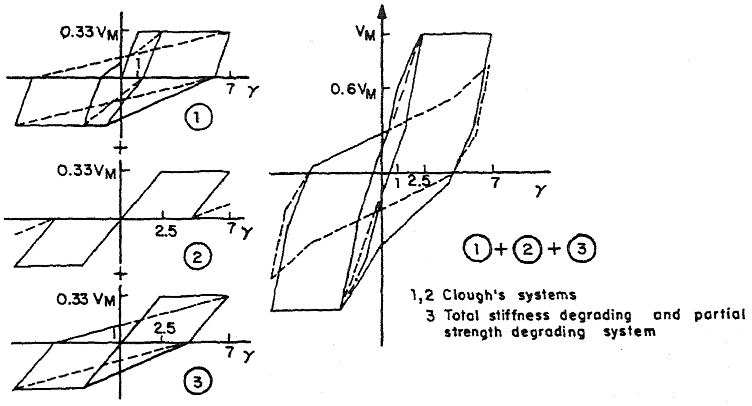
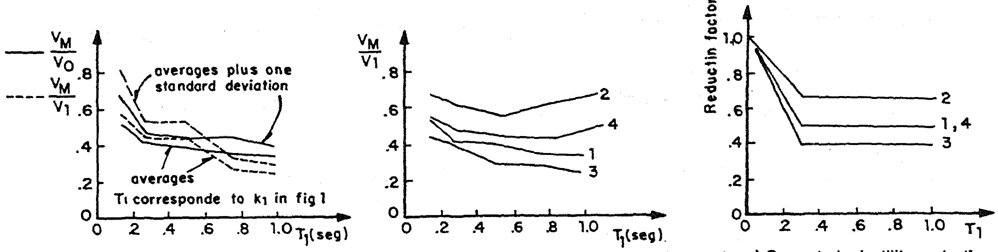
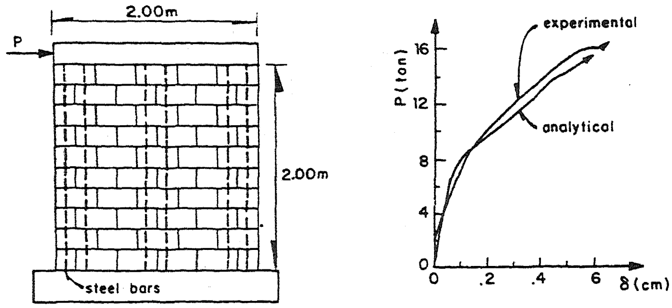


Fig 2 Superposition of simple degrading systems to obtain model for case 1 in fig. 1



a) Case 1 in table of fig 1 b) All cases of fig 1 c) Suggested ductility reduction factors

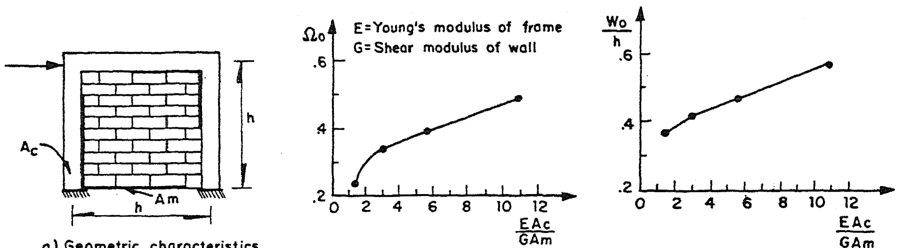
Fig 3 Ratios between elastic and inelastic responces of trilinear deteriorating systems representing several cases of masonry walls hysteretic behavior



a) Characteristics of the wall

b) Load—deformation curves

Fig 4 Comparison of calculated and experimental load—distortion curves

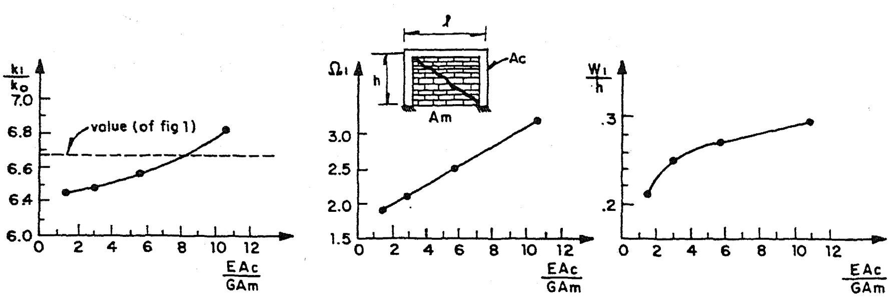


a) Geometric characteristics of the walls

b) Reduced shear area for wide column representation

c) Width of equivalent diagonal strut

Fig 5 Representations for the lateral stiffness of square infilled walls



a) Ratio betwen cracked and uncracked stiffness of infilled walls

b) Reduced shear area for wide column representation

c) Width of equivalent diagonal strut

Fig 6 Lateral stiffness of square, diagonal cracked, infilled walls