Control of Earthquake Damage in Buildings with Masonry Walls

Roberto Meli
Institute of Engineering, National
University of Mexico

ABSTRACT

To establish a rational procedure for deriving maximum interstory drifts that should be allowed by seismic codes, the experimental evidence on damage distortion relations for masonry walls is reviewed. Results of a large number of tests on masonry walls performed in Mexico for different types of units, mortars, reinforcement methods and loading conditions are used to propose critical wall distortions corresponding to different levels of damage.

A probability based optimization criterion is suggested to derive allowable drifts and quantitative results are obtained for conditions representative of Mexico City buildings and design practice. The limitations of the proposed procedure and of the values assumed for some variables are pointed out.

1. *Scope*

Low tensile strength of masonry makes it very vulnerable to in-plane distortions as those caused by lateral displacement of buildings under wind or seismic actions and those produced by vertical movements of supports due to differential settlements of the foundation. In fact, most economic losses in modern construction subjected to strong earthquakes have derived from cracking of masonry partitions or infills and other non structural damage in medium to high rise buildings whose flexibility has given rise to lateral displacements exceeding those tolerable by masonry.

In addition to specifying sets of requirements aiming at attaining adequate lateral strength and ductility, seismic codes specify allowable interstory drifts that should not be exceeded under lateral loads equivalent to the action of a design earthquake, with the purpose of providing structures sufficiently stiff as to avoid excessive non structural damage under frequent and moderate earthquakes. Code allowable drifts are said to be related mainly with damageability of masonry infills, but no quantitative derivation of specified values is known to the writer. At least in Mexico, there is a widespread feeling among structural engineers that lateral stiffness requirements of present codes are too lax and that buildings designed according to the code are exposed to non structural damage cost exceeding what most owners are willing to accept.

A thorough knowledge of the relation between in-plane distortion and amount of damage in masonry walls is needed as a basis of any rational requirements on the subject. Significant information on this respect can be derived from tests performed at the Institute of Engineering of the National University of Mexico in different research projects, namely those on the study of seismic behavior of different types of masonry walls (1,2), and that dealing with cracking-distortion relations for masonry and their use in the formulation of reccomendations

about allowable settlements in low-rise load-bearing masonry construction (3).

The main objectives of this paper are the gathering and interpretation of results of mentioned experimental programs for the purpose of deriving damage-distortion relations for different types of masonry walls. An attempt is made to establish a framework for using this information to derive optimum values of allowable drift consistent with the design philosophy of present seismic codes.

2. *Experimental results on cracking-distortion relation for masonry*

When a frame is subjected to lateral loads, the infill wall deforms in shear; the damage on the latter can be related with in the plane distortion, γ, equal to the ratio between interstory drift and story height, as shown in fig 1. The state of stresses can be reproduced rather accurately if a diagonal compression load is applied to the wall; this is the test most commonly used to study the behavior of masonry infill walls under lateral loads.

Results of a series of tests of this kind are reported in ref 3. The influence of some variables on the distortion that caused first cracking was studied by means of about 200 tests on small square walls (0.4 m side). The influence of other variables and the postcracking behavior were studied on 26 2 x 2 m walls, as shown in fig 2.

In order to ascertain the influence of strain (or load) rate on the mechanical properties, tests on small assemblages were performed at four different strain rates; in the fastest tests, failure was attained in few minutes, whereas in the slowest ones distortion was increased at a constant rate in order to produce cracking of the specimen in about two months. No significant difference was found in the distortion at cracking for different strain rates; slighthy lower cracking distortions were observed for the slowest rate. Even if strain rates for seismic actions are higher than the fastest rates used in these tests, it was assumed that all results could be used to evaluate cracking-distortion relations, irrespective of strain rate.

Results of tests on small assemblages are summarized in table 1. Masonry units tested included solid hand-made clay bricks, perforated extruded bricks with a low void ratio and different types of hollow bricks, plus a sand-lime brick. Three mortar dosifications of cement, lime and sand were used to build the small walls. It can be seen from table 1 that the distortion at cracking is rather variable, being higher for solid than for hollow units; the same results in the average were obtained for 1:0:3 and for 1:1:6 cement, lime, sand mortars; for 1:2:9 mortars this property was in the average 78% of that in the former cases. Bricks with large holes have little area of contact with mortar, and bond between both materials is poor due to the smoothness of the surface of units; both situations give rise to premature bond failure along joints. For very rough surfaces and for bricks with small perforations, the mortar penetrates into the brick producing shear key action and good bonding, failure being governed by tensile strength of mortar or of brick itself, for stresses and deformations higher than those corresponding to hollow units.

In tests on full scale walls the same types of units and mortars were used, the rate of strain was such that failure was reached in about three weeks and the main variables studied were wall reinforcement and vertical load. A very thin concrete frame surrounded each walls in order to simulate the confinement provided in actual buildings by elements as floor slabs and transverse walls. In some tests a thicker concrete frame was used to reproduce the influence of a confining frame; vertical reinforcement was also placed in the hollow units of some walls.

Some results are shown in table 1; the following conclusions are reached. Cracking strength was consistently 80% and cracking strain was in the average 72% of those obtained in small specimens. Differences are attributed to higher confinement provided by the loading setup in small tests, thus increasing tensile strength. For solid units bonded with good mortar, the distortion at cracking, γ_0, was slightly higher than 10^{-3} rad; for hollow unit masonry γ_0 was about half this value.

The strain at cracking was significantly increased by vertical compressive load (about twice for a stress of 0.36 M_{pa}). γ_0 was also slightly increased when walls were rendered with cement or gypsum plaster; in both cases when a crack was observed in plaster it always penetrated into masonry; this finding was contrary to a previous belief that gypsum plaster would be more brittle than masonry and would crack first.

The size of boundary frame and the interior reinforcement did not affect γ_0, but significantly modified postcracking behavior.

As a measure of amount of damage, the widths of the largest diagonal cracks were measured; as seen in fig 3, a linear relation was observed between crack width, ω, and distortion, γ,

$$\omega = \alpha(\gamma - \gamma_0)$$

where α is a constant depending on type of unit, vertical load and reinforcement. The crack width increases faster for hollow than for solid units and vertical load reduces the rate of growth of crack width.

The load-deformations curves of fig 4 show that strength continously decreases after diagonal cracking; the capacity reduction was greatly delayed in walls with interior reinforcement or with strong boundary frame. Damage in boundary frames occured for distortions above approximately 3×10^{-3} rad; less than 50% of the lateral strength was left for a distortion of 10^{-2} rad. The behavior shown in fig 4 was peculiar of tests with sustained load or with distortion increasing very slowly along several weeks. In tests under short term load a much less brittle behavior was observed.

In another research program aimed at finding reinforcement configurations leading to improved ductility and energy absorption capacity, 15 full scale walls similar to those previously described were tested under alternating lateral load (ref 2). In table 2 average distortion at cracking, γ_0, for hollow brick and concrete block walls is reported along with maximum distortion at which cracking strength could be sustained. Values of γ_0 are very similar to those obtained in tests under slowly increasing distortion. ($\gamma_0 \doteq 0.5 \times 10^{-3}$ for hollow units). Nevertheless, postcracking behavior was much more ductile than in previously described tests; in walls with boundary frame and interior reinforcement a distortion of almost 20 times γ_0 could be sustained before load decreased below diagonal cracking strength. Maximum distortion in walls with only interior reinforcement was considerably lower.

Fig 5 shows shear-distortion curves for walls tested under monotonic lateral load (ref 4). Walls with only interior reinforcement suffer drastic reduction of load capacity for distortions exceeding 5×10^{-3} rad, whereas walls confined by boundary frames can withstand distortions of 10^{-2} rad before having their load capacity significantly reduced.

From the point of view of behavior under seismic actions, what matters is the damage that a wall can suffer under repetitions of alternating loads and the progresive degradation of load capacity that can occur under such cyclic

load. From the results of tests reported in ref 1 and 4 it can be concluded that walls built with hollow units suffer much more degradation than those built with solid units, due to crushing and spalling of the shells of the units after a diagonal crack is formed. In framed walls built with solid units, significant degradation starts when a diagonal crack penetrates into the column as a result of the concentration of shear that occur in the corners of the frame after diagonal cracking of the walls. Nevertheless, if the shear strength at the ends of the columns is large enough to avoid propagation of diagonal cracks into the corner, very little deterioration is shown for very large distortions; in fact, the load could be raised to practically the initial maximum capacity after more than 50 cycles of distortion exceeding 30×10^{-3} rad. Fig 6 shows some cycles of shear-distortion curves for walls of hollow units with interior reinforcement and of solid units with weak and strong boundary frame; it can be appreciated that in the first case large degradation appears for $\gamma = 3 \times 10^{-3}$; in the second, degradation is not so large for $\gamma = 5 \times 10^{-3}$, and that the wall with strong frame could sustain a distortion of 15×10^{-3}, with only moderate degradation.

From the results discussed in previous paragraphs some characteristic values determining cracking-distortion relations for masonry walls when subjected to short-time diagonal compression can be proposed (table 3). Solid units are distinguished from hollow ones, allowable distortions being larger in the former case. Distortion at cracking is greater for walls subjected to vertical loads; different values are given for walls with interior reinforcement or with light boundary frame, for infills in a strong frame and for unreinforced walls. The characteristic points of cracking-distortion relations are: distortion at first cracking (γ_0), slope of the curve relating crack-width with distortion curve (α), distortion causing structural damage in interior reinforcement or in boundary frame or in units (γ_s) and distortion corresponding to major loss of lateral capacity of the walls (γ_u).

3. *Observations of damage in actual buildings*

Measurements or reliable estimates of lateral displacements experienced by actual buildings and their relation with non structural damage are scarce. In few cases the displacements could be determined from records of accelerographs located in upper floors of buildings; in spite of lack of accuracy of estimating displacements from double integration of accelerograms, there are clear indications that interstory drifts up to 0.002 or 0.003 times the story height did not produce damage in masonry infill walls.

Indirect information on damage that can be produced by lateral displacements during an earthquake can be obtained from measurements of settlements and for the damage they cause in buildings; nevertheless, great difficulties have been encountered when trying to correlate interbay measured settlements with wall damage, due mainly to lack of an initial point of reference; some rough index of overall distortion, as the differential settlement between center and corner of a building divided by the distance between these points was found better correlated with building damage. A survey of settlements and damage in low rise load bearing masonry wall construction is reported in ref 3; there it was found that for the same settlement, damage diminished with the level of vertical load on walls, corroborating what was found in laboratory tests. From these surveys of differential settlements it has been concluded that in general damage in masonry walls starts when distortions exceed 3×10^{-3} rad.

It seems clear that the distortion capable of producing damage in masonry walls of actual buildings are significantly larger than those found harmful in diagonal compression tests. The difference is attributed mainly to small gaps existing between structure and walls, which allow the structure to experience some

drift before making contact with the infill, as seen in fig 1, and the wall to rotate with respect to its basis. Distortions reported in the previous chapter were obtained from measurements of deformations within the masonry wall (from the changes of length of both diagonals). When the distortion were obtained as the ratios of displacements at the top of infilled frames to story heights in cantilever tests, much larger values of γ_0 were found. For instance, in a series of tests of this kind, Esteva (ref 5) measured average cracking distortions of 2×10^{-3} rad for solid brick infills and 1.5×10^{-3} rad for hollow blocks.

From the point of view of evaluating allowable interstory drifts in buildings, distortions should include relative movements between frame and infill; in order to take into account this effect it is suggested to add a constant value of 1×10^{-3} rad to distortions obtained from direct tests of the infill as those presented in table 3. This addition will affect mainly the value of γ_0 and is consistent with the assumption of an initial gap between frame and infill and also with the difference between measurements made in the infill and in the frame.

With this modification the values of table 3 can be used as the basis for obtaining damage-distortion relations for structures with infilled frames; nevertheless many more direct measurements in buildings are needed to obtain more reliable figures.

4. *Code requirements for drift limitations*

Earthquake resistant design provisions of most building codes require calculation of interstory drift from an elastic analysis of the structure subjected to the same set of equivalent lateral forces used to verify structural strength. Calculated drift should not exceed allowable values; in order to evaluate latter provisions, several aspects of the seismic design must be remembered. Equivalent lateral forces specified by codes are much lower than those that would be generated by the design earthquake if the structure responded within its linear range of behavior; it is admitted that a part of the energy induced by the earthquake can be dissipated by inelastic behavior, and therefore the required strength is smaller for ductile than for brittle structures. Design forces specified by codes are reduced proportionally to the ductility factor that the particular structural system is assumed be able to develop before failure. Nevertheless, maximum lateral displacement that the structure would suffer under the design earthquake is not that calculated with these reduced forces; if the behavior is elasto-plastic displacements would be approximately those calculated by elastic analysis with reduced lateral forces multiplied by the ductility factor implied.

It must also be remembered that what codes are trying to avoid or to limit by this kind or requirements is non-structural damage for earthquakes of intensities much lower than that of the design earthquake. For simplicity sake, drift verifications are made for the same design earthquake used to check structural strength, instead of specifying a different "service earthquake" with lower design forces; therefore, allowable drifts established by codes are higher than those intented not be exceeded under moderate earthquakes and are usually larger than those that produce damage in masonry walls.

Allowable drifts specified by different codes show large variations. Mexican codes state that calculated interstory drift (multiplied by the ductility factor) should not exceed 0.008 times the story height when masonry infills or partition are not properly separated from the main structure. An interstory drift from 0.010 to 0.015 times story height is allowed by ATC-3 code in USA depending on the importance of the building. On the other hand, New Zealand code states that twice the calculated drift should not exceed 0.0006 times the story height, otherwise

partitions should be separated from the main structure. It is evident that rather different considerations underlie such unlike allowable limits.

Drift limitations do not usually govern earthquake resistant design of load bearing masonry structures, where ductility reductions allowed by codes are very low and the area of walls needed in each direction to withstand story shear forces gives the structure sufficient stiffness to meet drift requirements. On the other hand, drift limitation is generally a governing factor in the design of medium to high rise house buildings in zones of high seismic risk, where sizes of column and beams must often be increased beyond those required for strength purposes, in order to provide enough lateral stiffness.

5. *An optimization criterion for defining allowable drift*

A rational criterion to set allowable drift must balance cost of increasing lateral stiffness of structure with consequences of earthquake intensities that could produce cracking of partitions and other non structural damage. A commonly accepted criterion of optimization in this sense involves minimizing the sum of construction cost plus expected monetary losses due to damage.

Let us assume that process of occurrence of earthquake motions at the building site is such that a certain measure of earthquake intensity, i, is related to average annual rate of exceedance, ν, as follows (ref 6)

$$i = A\nu^{-\frac{1}{r}} \tag{1}$$

r is found by fitting curves as eq 1 to statistical data; for data in the range of very low frequency of occurence, r is generally found to be around 2.5; in the range of more frequent moderate earthquakes r is generally around 1.

It can also be assumed that a measure of seismic action (seismic coefficient or spectral ordinate for a strucutre of given period and damping) is linearly related to i; furthermore, if displacements are calculated by means of a linear analysis, the maximum relative interstory calculated drift will be proportional to the seismic coefficient; thus,

$$\gamma = a\nu^{-\frac{1}{r}} \tag{2}$$

This equation gives the story drift that is exceeded in the average at an annual rate ν.

According to the adopted optimization criterion, monetary losses should be assigned to damages produced when the maximum drift in a building exceeds that initiating cracking of a partitions. Ref 3 describes an attempt to obtain a relation between distortion and damage by taking photographs of walls that were carried to different levels of damage corresponding to different values of distortion; these photographs were shown to engineers specialized in cost estimation for building construction, who where asked to assess costs of the repairs required to restablish the initial condition of the walls. Average repair cost were expressed as fractions of costs of building new walls and are shown for a particular case in fig 7; in order to take into account costs associated with troubles inflicted to building tenants, a twofold increase over repair cost was considered. A typical curve obtained according to this criterion is shown in fig 7; it can be expressed as follows:

$$
\begin{aligned}
D &= 0 && \text{if } \gamma < \gamma_o \\
D &= D_o(\gamma/\gamma_o)^n && \text{if } \gamma \le \gamma \le \gamma_M \\
D &= D_M && \text{if } \gamma > \gamma_M
\end{aligned}
\qquad (3)
$$

where D is the ratio between cost of damage and cost of replacement; n was found to lie between 1.5 and 2.

Damage can be expressed in terms of expected frequency of exceedance, ν, through eq 2, leading to a relation as shown in fig 8. The expected damage per unit time corresponding to an intenisty with an exceedance rate, ν, equals the product $D(\nu)d\nu$; hence, the expected value of damage per unit time is the area under the D vs ν curve of fig 8.

$$E[\ D_u\ ]\ \int_0^{\nu_o} D(\nu)d\nu \qquad (4)$$

Costs of damage occuring at different times during the expected life of a building must be transformed to present values by considering a rate ϕ as the difference between interest rate and rate of increase in construction cost; thus, the present value of the expected cost of damage is

$$E[\ D_T\ ] = \frac{1 - e^{-\phi L}}{\phi} E[\ D_u\ ] = L'E[\ D_u\ ] \qquad (5)$$

where L is the expected life of the building.

The cost of construction can be separated in a part independent from lateral stiffness and another that decreases linearly with the allowed lateral drifts; from this assumption,

$$C = C_D(q + p\,\frac{\gamma_o}{\gamma_o}) \qquad (6)$$

where C_D is the cost of construction of the building if the structure is designed so that its calculated distortion, γ_D, under the design earthquake equals γ_o; p is the fraction of cost that is independent from lateral stiffness and $q = 1 - p$.

Establishing as objective to minimize the sum of total cost plus expected damage,

$$C_T = C + E[\ D_T\ ] \qquad (7)$$

The optimum structure is that for which the drift calculated under the design earthquake satisfy the condition

$$\frac{\partial\ C_T}{\partial\ \gamma_D} = 0 \qquad (8)$$

Solving eq 8 for the conditions set in eqs 2 to 7, the following result is found

$$\frac{\gamma_D}{\gamma_o} = \left(\frac{p}{L'\beta\nu_D rK}\right)^{\frac{1}{1+r}} \tag{9}$$

where

$\beta = D_M/C_D$ ratio between maximum damage to non structural elements and total cost of buildings

$\mu = D_M/D_o$ ratio between maximum damage and damage at first cracking

$$K = \frac{1}{\mu^{r/n}} + \frac{\mu^{1-r/n} - 1}{(n/r-1)\,\mu}$$

The following values were considered representative of Mexico City typical buildings and of local code requirements.

ν_D, expected frequency of exceedance of design earthquake	= 0.02/year
L', expected life or building	= 50 years
ϕ , effective interest rate	= 0.03
β , relative cost of non structural damageable elements	= 0.20
$\mu = D_M/D_o$; from fig 7	= 133
n , for infills in a strong frame	= 1.8
r , exponent of eq 1	= 1.5
p , fraction of cost dependent on lateral stiffness	= 0.1

For these assumptions, the optimum value of γ_D/γ_o is 2.5. This means that the drift calculated under design earthquake should not exceed 0.005 for solid unit infills and 0.0037 for hollow unit infill walls.

It must be mentioned that results are rather sensitive to changes in some of the variables, and some of those variables cannot be stated with good accuracy. For instance, if the design earthquake specified by the code has a return period of 250 years instead of 50 years, a twofold increase in allowable drift would be obtained. On the other hand, optimization method described does not take in account, for instance, uncertainties in values of variables involved. Former values of allowable drifts were derived on the basis that no further safety factors are taken in the design; actually, some conservatism is involved in the way lateral stiffness and equivalent load are defined; thus, allowable drifts could be higher and nearer to values of the present code.

6. *Conclusions*

Some information has been presented on the cost of the damage that can be expected in masonry walls when subjected to in plane distortions. Data based on laboratory tests are rather extensive and conclusive; nevertheless, in actual buildings masonry walls seem to be able to afford larger distortions without damage. Careful damage evaluations and analyses of buildings after earthquakes are needed in order to better quantify such discrepancies.

A crude optimization method has been used to find the allowable drifts that should be specified in accordance with present design methods in seismic codes. It is found that values are lower than those specified at present by Mexican codes; nevertheless, results are very sensitive to the values assumed for some variables, and rather arbitrary assumptions were made for some of them; the consensus of different specialists must be obtained in order to set more adequate values. One of the objectives of this paper is to arise interest in studies aiming at finding more rational solutions to the problem.

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TABLE 1. RESULTS OF DIAGONAL COMPRESSION TESTS

Unit	Mortar(1)	Small Specimens		Full Scalle Walls		
		v_o (2)	γ_o (3)	Reinfor-cement (4)	v_o (2)	γ_o (3)
Solid Clay Brick	1 0 3	0.46	1.1	II	0.35	1.6
	1 1 6	0.40	1.1			
	1 2 9	0.38	0.9	II	0.35	0.5
Lobsang Celled Clay Brick	1 0 3	0.46	1.0	I	0.54	1.3
	1 1 6	0.40	1.1			
	1 2 9	0.31	1.0	I	0.27	0.4
Honeycomb Celled Clay Brick	1 0 3	0.82	1.6	I	0.54	1.3
				I(σ = 0.37)	0.75	2.0
	1 2 9	0.37	1.0			
				I	0.37	1.0
Perforated Clay Brick	1 0 3	0.95	0.9	I	0.82	0.5
	1 1 6	0.77	0.8			
	1 2 9	0.44	0.7	I	0.64	0.8
Round Holed Brick	1 0 3	0.41	0.7	I	0.33	0.4
	1 0 3			I(σ = 0.78)	0.56	0.9
	1 0 3			III	0.41	0.3
	1 1 6	0.35	0.8			
	1 2 9	0.30	0.6	I	0.23	0.3
	1 2 9			I(σ = 0.36)	0.45	1.0
Square Holed Brick	1 0 3	0.55	1.4	I	0.32	0.6
	1 0 3			II	0.23	0.6
	1 0 3			III	0.28	0.3
	1 2 9	0.23	0.4			
Sand Lime Hollow Brick	1 0 3	0.66	0.5			
	1 2 9	0.39	0.3	III	0.42	0.8

(1) Mortar dosification by volume; cement:lime:sand

(2) Shear stress at cracking, Mpa

(3) Distortion at cracking (x 10^3)

(4) Reinforcement I: Light confining frame (70 mm width); II: Strong confining frame (150 mm width); III, Light confining frame plus vertical reinforcement in three holes

σ = vetical stress, in Mpa

TABLE 2. CRITICAL DISTORTIONS FOR WALLS TESTED UNDER SHORT TIME LOAD

Type of Wall	γ_o	γ_m
Hollow brick with boundary frame and interior reinforcement	0.7	13.2
Hollow brick with interior reinforcement	0.5	2.9
Concrete block with interior reinforcement	0.5	4.8

TABLE 3. CRITICAL DISTORTION FROM DIAGONAL COMPRESSION TESTS

Type of Unit	γ_o				γ_M			
	$\sigma = 0$	$\sigma > 0.2$	α	γ_s	Unreinforced	Interiorly Reinforced	Ligth Frame	Strong Frame
Solid	1.0	1.5	1.1	3	3	---	15	30
Hollow	0.5	1.0	1.4	3	3	6	10	20

γ_o distortion at first diagonal crack in rad x 10^{-3}

γ_s distortion for beginning of structural damage

γ_m distortion for structural failure

α constant of equation $\omega = \alpha(\gamma - \gamma_o)$

ω crack width, in mm

σ compressive stress due to vertical load on the wall (Mpa)

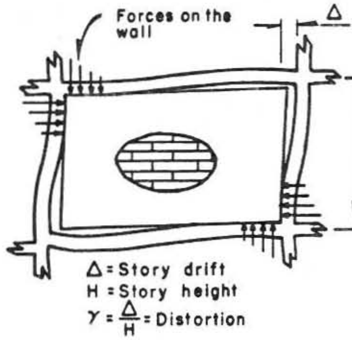

Fig 1. Forces on an infilled frame subjected to lateral loads

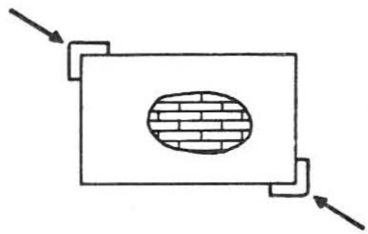

Fig 2. Diagonal compression test

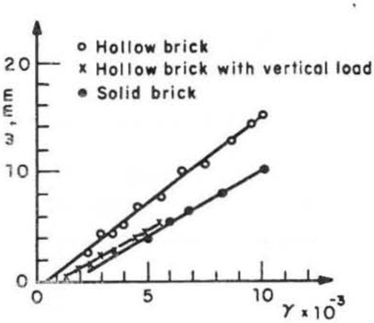

Fig 3. Crack width vs distortion from diagonal compression tests

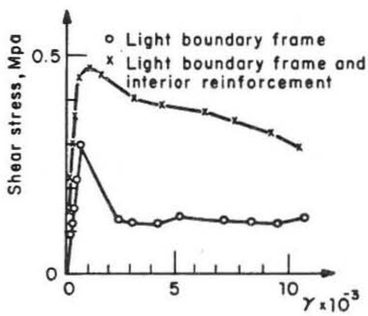

Fig 4. Shear-distortion curves for walls under slowly increasing strain

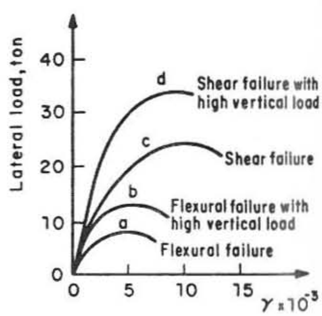

Fig 5. Load-distortion curves for walls under monotonic short term lateral load

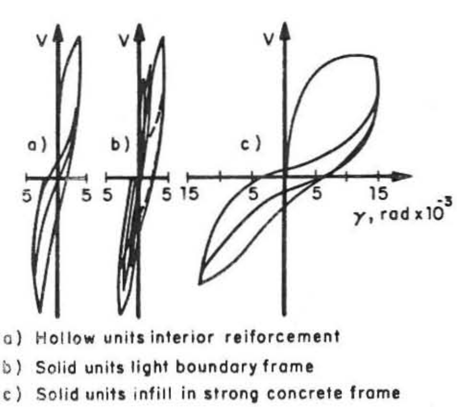

Fig 6. Typical load-distortion curves under alternating lateral load

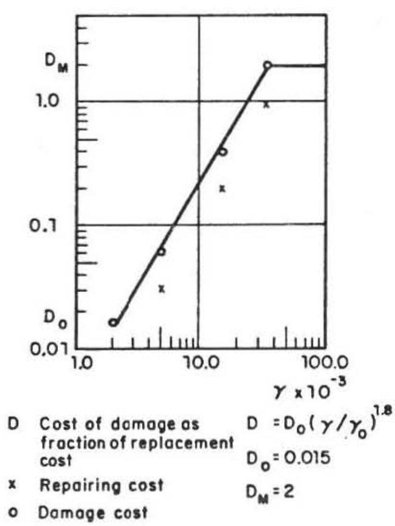

Fig 7. Damage-distortion relation for an infill wall of solid units

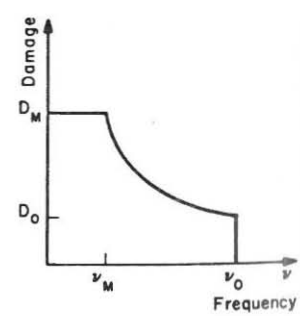

Fig 8. Typical damage-frequency relation