EARTHQUAKE DAMAGE TO NONSTRUCTURAL ELEMENTS

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ABSTRACT

Partitions and exterior masonry walls assumed to be "nonstructural" elements by the structural engineer in the design stage can modify considerably the response of buildings to earthquakes if they are constructed neglecting the recommended gaps between these elements and the structure or if these gaps are small compared with the expected motion. Additional experimental studies have to be made in order to establish the best method to eliminate this effect and the way these walls should be supported on the structure.

INTRODUCTION

For the purpose of this paper, nonstructural elements will be defined as those architectural parts of a building that do not contribute "theoretically" to the strength and stiffness required to take the applied loads, such as walls, facades, ceilings, ducts, piping and many other items because the structural engineer has so decided. At the beginning of the design stage the structural engineer has to define which part of the building is going to be used to resist the loads and which one is not, and elaborate mathematical models according with these assumptions; walls, facades, and vertical ducts may sometimes be considered part of the structure and its contribution to strenght and stiffness should be included in the analysis.

In a broader sense, some authors regard also as nonstructural elements, mechanical and electrical equipments and other fixtures attached to the building. However, the ones considered here may considerably change the response of the building during earthquakes when they have been constructed violating the assumptions made by the structural engineer with respect to their contribution to stiffness and strenght.

Modern architecture has been oriented to skeletal structures that use mainly columns as vertical supporting elements, in order to have freedom for the distribution of spaces in each floor of the building. This has modified the traditional building construction scheme that used some partition and facade walls, continuous throughout the height of the building, to support a good part of the vertical and horizontal loads to which the building is subjected during its useful life. In some cases these bearing walls were very thick, leading to very stiff structures for the lateral movements caused by wind and earthquake forces, as compared with the stiffness of skeletal structures.

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In modern buildings a majority of the partition walls are made of light materials but exterior walls and permanent walls around stair and elevator shafts are usually made of masonry or concrete. It is usually specified that they should be constructed in such a way that they do not interfere with the motion of the structure caused by lateral forces, in order to avoid their damage and colaboration to stiffness when they are nonstructural.

Frequently the gap between the structure and the wall is small and the separation is not efficient to prevent the participation of and damage to those elements during strong or medium earthquakes. A large proportion of earthquake damage in recent seismic movements has occurred on "nonstructural" partition walls, facades, and ceilings with high repair or replacement costs (Ref. 1-4).

LATERAL STIFFNESS OF SKELETAL STRUCTURES

The determination of the lateral stiffness of skeletal or framed structures is a simple matter using one of the different computer programs available. The mathematical model may be simplified to a series of plane frames or the complete structure analized in three dimensions. The lateral stiffness depends mainly on the elastic properties of the materials used and on the actual dimensions of the beams and columns which form the skeleton, either for reinforced concrete or steel buildings; however, there are some uncertainties as to the contribution to the stiffness of different concepts: for instance, it is not very clear what is the effective contribution of reinforced concrete slabs, cast monolithically with concrete beams or anchored to steel beams by means of shear connectors. There are also occasions in which it is necessary to evaluate the effects of the dimensions of the intersection of beams and columns at the nodes in the stiffness of the frames, as large errors can be introduced by analyzing the frame as though beams and columns had constant properties along their center lines, neglecting the modification of these properties at intersections; this effect is very pronounced in deep spandrel beams at the facades, (Ref. 5). Other factor that can greatly influence behaviour are: the consideration of effective moment of inertia of cracked sections along the bars; the ratio of the height to effective width of the frame (neglecting cantilivers), which can produce large axial forces in the end columns and their vertical deformations which reduce lateral stiffness of the frame; shear deformations in the members and at the nodes should also be taken into account in some occasions, as well as the effects of axial forces on the angular stiffness of columns.

There are computer programs that use finite element techniques to take into account some or all of the foregoing effects and provide a good estimate of a structure's lateral stiffness for a set of prescribed applied forces.

When the beams are stiff as compared with the columns, lateral stiffness tends to be independent of the applied forces, but as beam stiffnesses decrease, lateral stiffness of the frame changes considerably for different sets of lateral forces. Blume, (Ref. 6), has proposed the use of a nodal rotation index to estimate the type of frame we are dealing with, that is:

a real frame, with the columns bent in double curvature and a virtual hinge in each, or a "cantilever disguised as a frame", where the columns are bent in single curvature and there is no virtual hinge along them; the usual situation is to have beams that are flexible compared with the columns. There are some structural systems where the floor-system flexibility can be critically high; for example, hollow flat plates supported only by columns, forming "equivalent" rigid frames to resist vertical and horizontal loads. There are some codes that do not allow this situation and require the addition of shear walls to improve the behaviour of the system; however, it is permitted in others, like the Mexico City code. This leads to very flexible structures, which generally are near the upper limits of deformation allowed by codes or exceed them.

Skeletal structures have limited application due to the large lateral deformation that may result from seismic loads; however they have been used for buildings with more than 40 stories, like the Latino Americana Tower in Mexico City, (Ref. 7). Great care has to be taken concerning the gaps that should be left between the structure and the nonstructural elements in order to avoid damage to them in medium and strong earthquakes, and to satisfy the assumptions made about the behaviour of the structure.

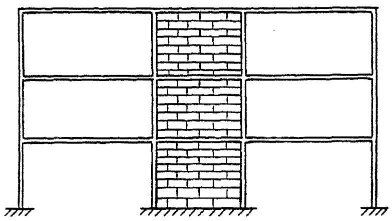
LATERAL STIFFNESS OF INFILLED-FRAME STRUCTURES

Several studies have been realized to take into account the modification in lateral stiffness of framed structures where some bays are filled with masonry or concrete walls. In general, it has been observed that lateral stiffness is increased considerably, as walls are very rigid in their plane and their stiffness may be on the order of ten times that of the enclosing frame. Unfortunately their strenght is not compatible with this stiffness in most cases. There is also a definite change in the form in which the frame will resist lateral loads; flexural effects will decrease substantially. The wall in a bay may be idealized as a diagonal strut oppossing the lateral deformation of the frame; there are expressions for the dimensions that should be considered for this "equivalent" diagonal member, (Ref. 8).

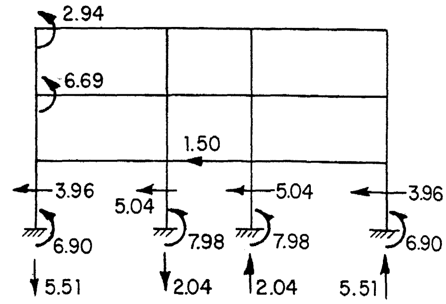
A computer analysis may be performed taking into account these additional members, which transform the rigid frame into a trussed frame. Figure 1, taken from ref 8, shows clearly the drastic changes in bending moments and axial forces obtained when a masonry wall is included in the analysis of a frame or when it is neglected. Obviously, the design of the frame elements with the moments and forces obtained without the wall is useless when the wall is constructed attached to the frame, and some damage may be expected when the structure is subjected to earthquakes, as shown in figs 2 and 3. Damage occurs not only in the weaker masonry wall; it can also take place in the columns or beams of the enclosing frame, usually due to the additional shear forces at the zone of intersection.

ANALYSIS VERSUS CONSTRUCTION

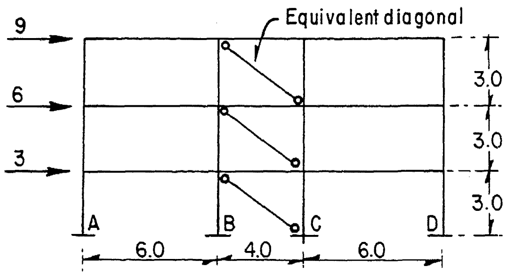
Building design should be made according to different limit states that include strength and deformation aspects. Usually strength requirements allow consideration of ductility reduction factors for the computation of



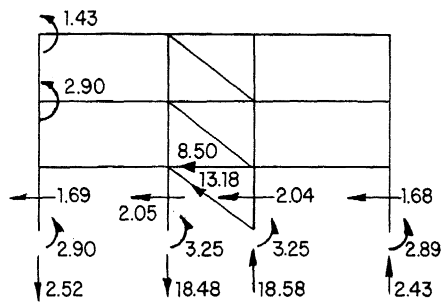
Column sections 0.30 x 0.30 ; Beam - sections 0.25 x 0.50 , f'c = 210 kg/cm²
Masonry walls, 0.15 cm thick



a) Without diagonals



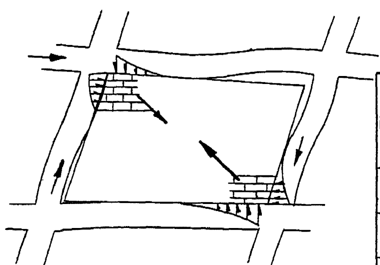
Forces in ton; lengths in m.



b) With diagonals

Forces in ton ; bending moments in m.

Some results from the analysis of the frame



Masonry wall confined by a frame

LEVEL	a) WITHOUT DIAGONALS		b) WITH DIAGONALS	
	Lat. Displ. cm	Stiffness Ton/cm	Lat. Displ. cm	Stiffness Ton/cm
3	3.72	10.2	1.47	25.4
2	2.84	10.3	1.11	27.2
1	1.39	13.0	0.56	32.1

Fig 1 Frame with masonry walls

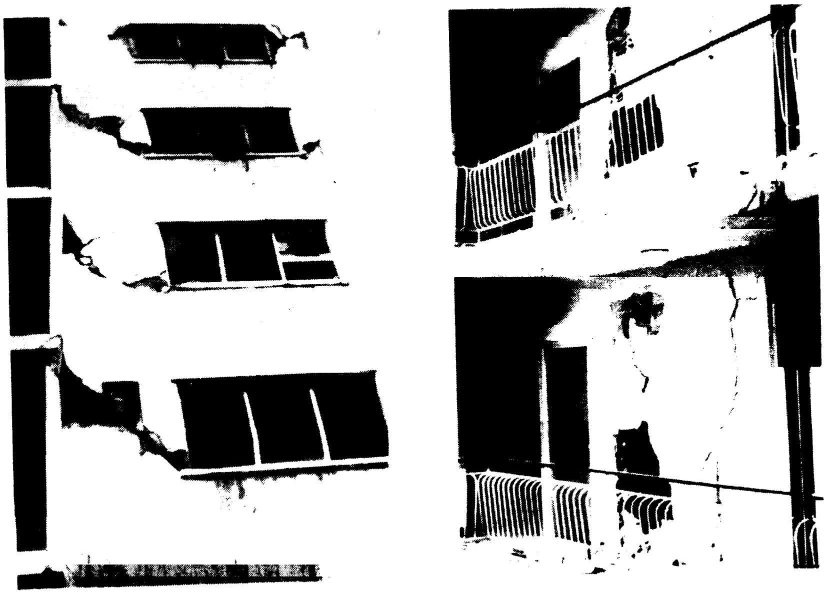


Fig. 2 Damage to nonstructural facade walls

Fig. 3 Damage to nonstructural walls as well as to structural elements. (Note the width of the "equivalent" diagonal)

the design seismic forces, to take into account the excursions into the inelastic range of some parts of a structure and the formation of plastic hinges during the design earthquake. However, for the computation of the displacements that the structure will experience with the design earthquake, it is considered that the displacements produced by the reduced forces should be multiplied by the reduction factor in order to obtain displacements equal to the elastic displacements that would be obtained with the elastic non-reduced forces, (Ref. 9). Codes usually specify the limit displacement that can be tolerated as a fraction of the story height. All the elements that contribute to the stiffness of the structure should be included in this calculation so that total computed force for each floor of the building is distributed between all resisting elements in proportion to their stiffness.

Therefore, when the structural engineer considers that partition and exterior masonry walls are "nonstructural" and neglects them in his calculations, his analysis can be seriously wrong if the construction engineer builds those walls integrated to the structure making them structural also.

This frequently happens due to lack of adequate communication between the structural engineer and the construction engineer. The drawings for construction should include clear details of how the nonstructural elements are to be built; otherwise the construction engineer, who ignores the assumptions made in the design, could build those walls improperly changing completely the structural behaviour.

It may also happen that the nonstructural walls are constructed correctly, but the gaps left between them and the structure are filled with noncompressible materials or that these materials absorb cement paste during construction and harden or that the wall veneer, such as plaster or ceramic tiles, cover also the gap and nullify it.

The state of the art, reflected in code specifications, may be as well the reason for inadequate performace of these gaps. The Mexico City code, for instance, specified in its 1966 version reduced seismic forces, which were used for the computation of displacements without the amplifications mentioned before; the gaps between structure and nonstructural walls were left according with these displacements, which were tolerated if they did not exceeded 0.002 times the story height. The 1976 version of the same code, still in use, specifies elastic seismic forces that can be reduced according to a ductility reduction factor that varies between 1 and 6, depending on the materials used, type of structure and care exercised in its detailing, and where the total displacements are computed multiplying those obtained with reduced forces by the ductility reduction factor; these total displacements are compared with tolerable displacements obtained as 0.008 times the story height. Actual reduced forces are approximately the same specified in 1966 for the case of rigid frame buildings; therefore, the gaps left prior the 1976 are insufficient for this type of buildings, according to the new code and the failures are waiting for the earthquake that will cause them.

In the last fifteen years many tall and medium height buildings have been built in Mexico City using structural systems consisting of flat plates supported only by columns, forming "equivalent" rigid frames to resist vertical and lateral loads. This leads to very flexible structures, as was mentioned before, and during the March 14, 1979 earthquake, many of these buildings suffered considerable damage in partitions and facade walls, (Ref. 4). Some of the buildings were stiffened to reduce future damage, but the majority were only superficially repaired and will be damaged again in future strong earthquakes.

Thus, the fact that the structure behaves in a manner completely different from that assumed by the structural engineer, due to lack of congruence between the mathematical models used in the analysis and the prototype, can be very dangerous. There are special cases when this is more critical: one of them is for corner buildings with exterior walls forming an angle, where large torsional effects may damage the whole building if these walls are made structural during the construction. Another case is the one frequently encountered in school buildings that have two longitudinal frames, with walls of partial height in one of them and without walls in the other, where the shortened columns are stiffened several times (this stiffness varies roughly with the cube of the effective length, so if this is one third of the length considered in the analysis, the increase in stiffness is on the order of 27 times) which causes shear failure of them, fig 4, and torsional effects in the entire building.

I consider that research projects are needed to determine the best way to avoid the above mentioned situation, and to recommend practical details to construct these walls, taking into account the points of view of the structural engineer, of the architect and of the construction engineer. Although some details have been proposeed, (Refs. 8-9), they are sometimes expensive and difficult to build.

ACKNOWLEDGEMENTS

Dr. Emilio Rosenblueth read the original version of this paper and made valuable suggestions.

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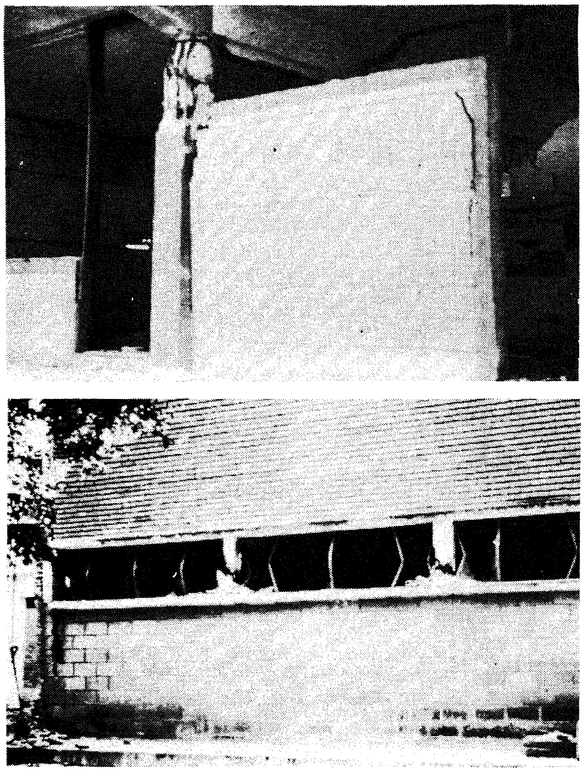

Fig 4. Damage due to partial height walls

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