PROPOSAL TO SAFEGUARD EXTERNAL WALLS AND PARTITIONS DURING EARTHQUAKES

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SUMMARY

This paper describes a study on earthquake resistance of partitions and external walls in a normal building having a reinforced concrete structure, with a view to establishing operational proposals for safeguarding these non-structural elements, the repair cost of which accounts for a large proportion of the repair cost of the entire building. By following the stages of collapse of the brick panels via stepwise dynamic analysis, a correlation has been established between seismic intensity and amount of damage. The procedure has been applied to check building in Lioni (Southern Italy), designed solely for vertical loads, damaged during the November 1980 earthquake. It has thus been possible to calibrate the procedure by checking the results of the dynamic against the amount of damage encountered. A relationship linking earthquake intensity with repair costs has been established by evaluating the cost of repairing the brick panels and the RC structures.

STRUCTURAL MODEL AND COLLAPSE CRITERIA

For seismic structural investigations the brick panels have been broken down into the following three types, on the basis of the criteria described here:

a) Infilling panels: walls without any large openings, within the RC frame formed by columns and beams (including flush floor beams)
b) Non-infilling panels: walls with no openings and with plan dimension at least two-thirds the height (between floors)
c) Non-resistant walls: free walls with large openings or plan dimension less than two-thirds the height.

Infilling panels and frames

The analysis of the behaviour of the infilled frame is based on the results of (Ref. 1), bearing in mind the points made in the current Italian code (Ref. 2). The horizontal action is transmitted between frame and infilling through contact in the corner areas alternatively counterposed along a given diagonal assuming linear trend of vertical and horizontal pressures (p_V and

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p_o respectively) at the contact with the beam and the column, the following expression is obtained for the relationship between the maximum values there of (a = length of contact zone):

$$\frac{p_v}{p_o} = \frac{a}{h} \left(\frac{h}{\ell}\right)^2 \left(3 - 2\,\frac{a}{h}\right) \quad (1)$$

For the ratio a/h, by introducing an equivalent square section column (side d_o), since h' = h, then:

$$\frac{a}{h} = 1.2\,\frac{d_o}{h}\,\sqrt[4]{\frac{E_c}{E_m}\,\frac{h/t}{\sin 2\Theta}} \quad (2)$$

In this last expression the ratio h/1 has no marked influence, at least for values that do not depart greatly from unity, for instance between 0.8 and 1.1; expressions (1) and (2) have been used to determine the ultimate strength of hollow-block walls with the holes running horizontally or vertically. The ratio p_v/p_o is greatly influenced by the value of h/1 which, in the range indicated, leads to variations of between 0.64 (broad panels) and 1.21 (tall panels). It is observed that as the ratio d_o/h is reduced, so too is the length a of the contact zone and the ratio p_v/p_o. The ultimate strength of the panel may be reached in different ways, depending on whether there is:

a) Failure due to diagonal cracks. According to the Code (Ref.2) the failure check (Type A) refers to the dimensionless value

$$\left(\frac{H_o}{\tau_k t\, 1}\right)^2 \leq 1 + (0.53h/1 - 0.13)\frac{H_o}{\tau_k t\, 1} \quad (3)$$

having eliminated the factor of safety.

b) Failure due to crushing of the masonry in the contact zones with the frame According to the Code (Ref.2) the failure check is performed with

$$\frac{H_o}{\tau_k t\, 1} \leq 1.6\ \sigma_k/\tau_k\ \cos^2\Theta\ \frac{h}{\ell}\ \sqrt[4]{\frac{E_c}{E_m}\,\frac{I}{th^3}} \quad (4)$$

In the case of hollow-block masonry, failure occurs due to crushing in the vertical or horizontal direction, essentially depending on the parameter d_o/h defined above. Ultimate strength conditions may be reached owing to tensile failure of the column or due to shear failure of the ends of the column, or even due to the joint coming apart because of slippage of the lower bars of the beam. In usual buildings, the limit condition considered here is the shear failure. The shear strength of the columns - taking account both of transversal reinforcement and the strength of the compressed concrete - has been calculated by the formula derived from (Ref. 3), written in dimensionless form thus:

$$\frac{T_{R2}}{f'_c\ b\ d\ 0.416} = \frac{f'_a}{f'_c}\,\frac{A_{sw}}{b\ s}\,2.16 + \frac{\tau}{f'_c}\left(\frac{N}{M}h + 6\right) \quad (5)$$

where T_{R2} is the ultimate shear strength of the column. In place of the τ_{RD}

proposed in (Ref.3) a τ increased by 50% has been adopted. In the infilled frame it has been assumed that attainment of limit conditions of the column leads to failure of the panel, but not vice versa.

Non-infilling panels

It is assumed that the panel is fixed - floor by floor - at the bottom and at the top to the horizontal members and without any vertical load. The limit condition has been taken to be the attainment of a value $\rho\tau_k$ (fraction of the characteristic value of the hollow-block walls) due to the unit action at the contact with the horizontal members. Values between 0.3 and 0.5 have been adopted for the coefficient ρ, depending on the type of wall and the construction method. The remaining walls, defined as non-resistant, have been tied in with the preceding ones, as far as susceptibility to damage is concerned.

Columns

In the case of the columns, checks have also been made of failure due to combined compressive and bending stress. In both cases of failure, i.e. due to shear and to combined compressive and bending stress, in the successive step the ability of these columns to bear the vertical loads has always been assured.

ANALYSIS PROCEDURE

The ETABS program has been used for the seismic analysis. Though the program is limited to the calculation of the dynamic response in linear phase, it permits parametric investigations to be performed on several buildings cheaply. The procedure involves determination of the first six vibration modes and calculation of the structural response for the acceleration spectrum reported in Fig. 1, having put ground acceleration at 1.0 m sec.$^{-2}$. In spatial reproduction of the structure, special care was taken to ensuring correct representation of the brick panels and the specific structural elements, such as staircases. For the latter, equivalent stiffening elements were introduced. For the former - if infilling panels are involved - the infilled frame combination was adopted, and this combination was maintained for the non-infilling panels too, by inserting dummy columns of very low stiffness. The ETABS program was used repeatedly,in a kind of "step-by-step" procedure, determining by direct proportionality the collapse acceleration value and the group of elements supressed in the following step. The collapse accelerations for the individual panels were determined by comparing the shear values on the panels obtained by the processing, with the ultimate strength values derived by application of expressions (3) and (4). The maximum strength T_R of the columns bounding the infilling panels was calculated by applying formula (5). Assuming that the maximum shear stress on the column adjacent to the panel is at least equal to that on the panel itself, by comparing the ultimate strength of the panel and the ultimate strength of the column (the latter depending on stresses M and N at the end and hence varying with acceleration) the actual collapse acceleration of the infilled frame was calculated.

USE OF PROPOSED PROCEDURE

The analysis procedure and the results obtained for one of the buildings situated in the zone hit by the November 1980 earthquake and damaged to a varying extent are given below as an example of how the proposed procedure is applied.

Characteristics of building

The building illustrated here has an RC structure and was almost complete at the time the earthquake occurred (it lacked only doors, windows and flooring). The building has four floors above ground, plus an unusable loft floor. Because of the steeply sloping nature of the site a basement floor occupies about half the plan area of the building on the downstream side;this basement was not damaged and has not been included in the structural scheme, which is considered to be on one foundation level (Figs 2 and 3; Photo 1).The structure has a great number of flush floor beams (h=22 cm). The first two floors of the building were seriously damaged by the earthquake. The partitions were built with hollow bricks (8 holes) measuring 8x25x25 (cm) set vertically with the holes running horizontally. The external walls have two wythes - outer and inner, with 8x25x25 (cm) and 12x25x25 (cm) hollow bricks set vertically and the holes running horizontally. Fig. 2 illustrates the distribution of the partitions and external walls on typical floor. The strength of the masonry was evaluated on the basis of the results of tests and by reference to the literature (Ref.4). The average value adopted was 0.4 MPa, it being held that oscillations due to variations of local conditions need not be considered. As regards the strength of the RC members, the analysis was based on the average strength values of the individual components. Lacking an extensive campaign of tests on samples taken on site, a range of average values was considered in all cases - wider in respect to concrete and narrower where steel is concerned - to take account of local variations in quality that are often encountered. In detail, the values adopted for plain round steel bars were 390 x (1±0.08) MPa, and those for concrete 25 x (1± 0.20) MPa.

Results and comparison with actual damage surveyed

Six computer runs were performed, namely 0, A, B, C, D and E. The results are given in Fig. 4 which indicates the failures of brick panels and columns gradually eliminated in the successive runs. Comparison of runs 0 and A reveals the contribution of the non-infilling panels: these fail at accelerations of around 0.5 m sec^{-2} (so they were eliminated in Run A) resulting in an increase in stresses in the infilled frames (Run A), none of which was damaged in Run 0. In the consecutive runs, A, B and C, "sequential" failure occurred both in the infilling panels and in the columns, always at about the same acceleration of around 1 m sec^{-2}. In Run D, failure of panels and columns occurred at ground acceleration of 2 m sec^{-2}, while failure of some columns due to combined compressive and bending stress occurred at this level of acceleration. In Run E, three infilling panels on the first floor and three on the second remained. Five of these failed even at an acceleration of about 2 m sec^{-2}. Total failure of the first and second floor panels virtually occurred on this run, while damage on higher floors was still slight, with some panels on the verge of collapse. The accompanying photographs of the building provide a general picture and the more significant details of the damage encountered after the November 1980 earthquake (Ref.5). It can be seen from Photo 1 that there was total failure of the brick panels on the 1st and 2nd floors, while the amount of damage to the external walls and partitions of the higher floors is slight. Photo 2 details the shear failure of Column 21 at the 2nd floor (see also Photo 1), detected in Run A of the analysis. Photos 3 and 4 both relate to the 2nd floor. They illustrate the amount of damage that occurred to the partitions, the external walls and the columns.

CONCLUSIONS AND PROPOSALS

Having evaluated the repair costs of the individual structural elements (Ref.6) and taking account of the relationship between ground acceleration (a_t) and damage (Fig.4), the following relation between a_t and repair cost has been obtained:

$$\frac{c}{c_o} = 0.15 + 0.20(a_t - 0.5)$$

which holds good in the 0.5-2 m sec^{-2} acceleration range:

c is the repair cost of 1 m^3 of building, needed to restore it to complete functionality;

c_o is the construction cost of 1 m^3 of building (of middling characteristics) with an RC frame, complete with finishings and utilities.

Buildings of the type considered suffer quite considerable damage even at low seismic intensities. To reduce repair costs the strength of all the non-infilling panels must be increased, thus ensuring an indirect benefit for all the other structural elements too. This fact is evident from Fig.5 which indicates as a function of a_t the trends of the shear forces on three typical elements - each representative of a category - at Run 0 (when all the panels are present) and at the following Run A (when the non-infilling panels have disappeared). Hence, to bring ground acceleration which results in the collapse of the first structural elements (non-infilling panels) to about 1.3 m sec^{-2}, the increase in strength required is defined, with reasonable accuracy, by the ratio H_{03}/H_{01}=2.5, and can be obtained by improving the connection between the sides of the panels and the horizontal structures.

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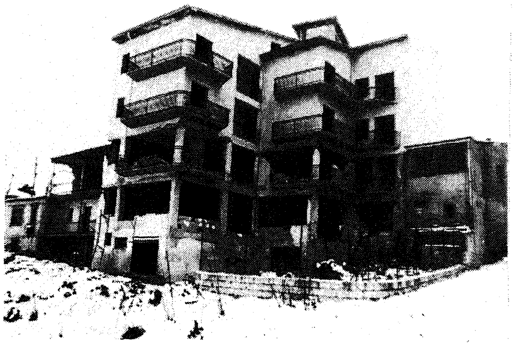


Photo 1

Photo 3

Photo 4

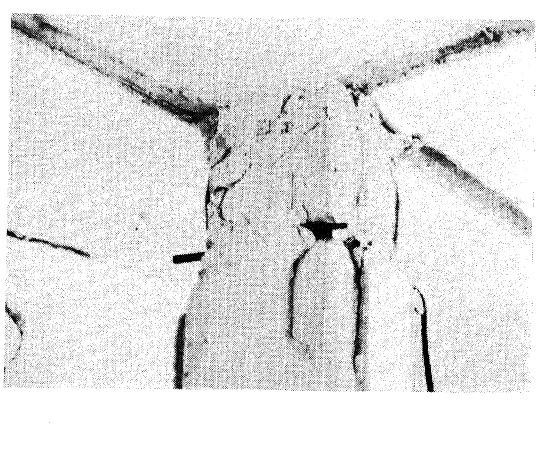
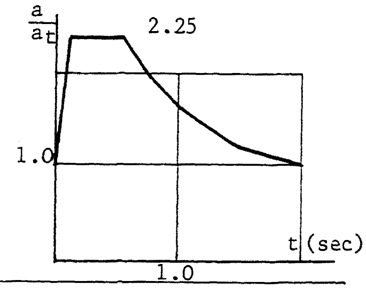


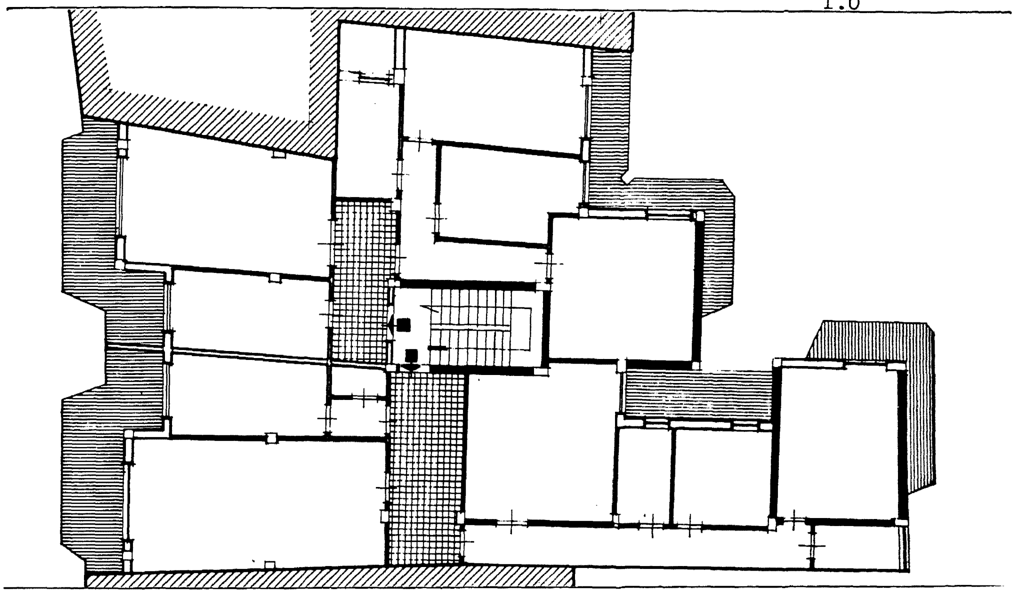
Photo 2

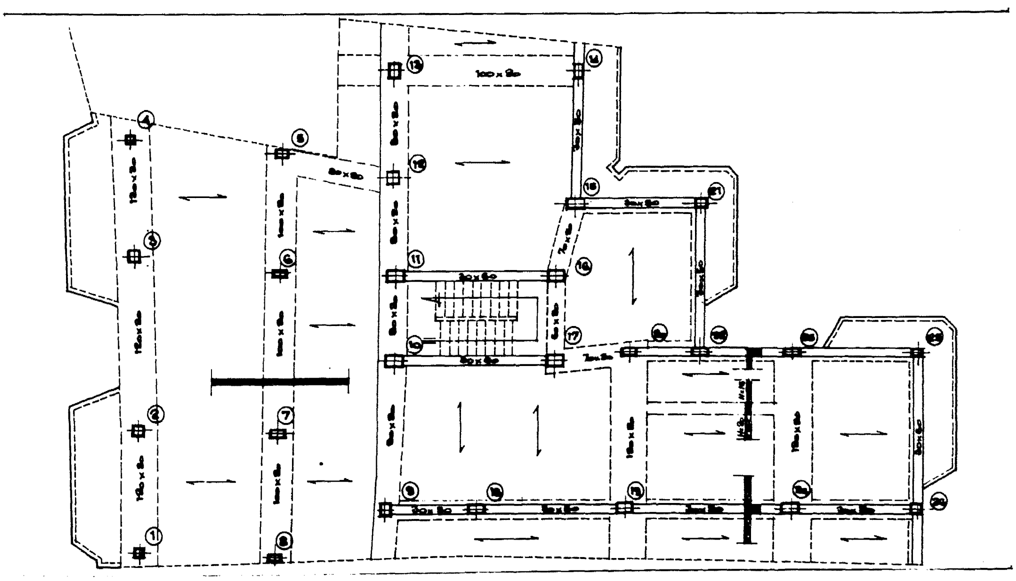
Fig. 1 - Acceleration spectrum

Fig. 2 - Plan of typical floor showing wall distribution (Building at Lioni - Southern Italy)

Fig. 3 - Structural plan of typical floor







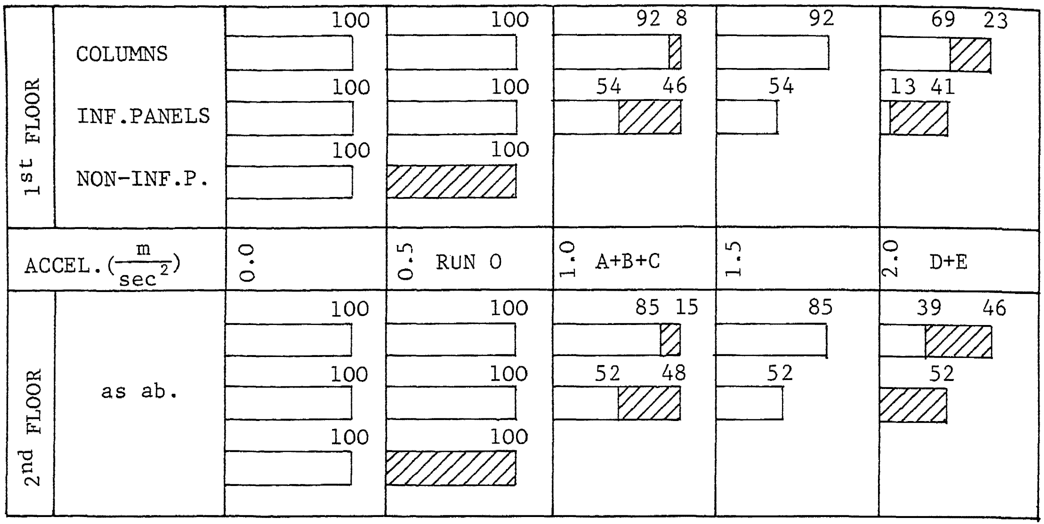


Fig. 4 - Progression of damage to columns and panels in the six steps of the calculation, for the first two floors of the building above ground. The elements damaged are expressed as a percentage of the total (shaded area)

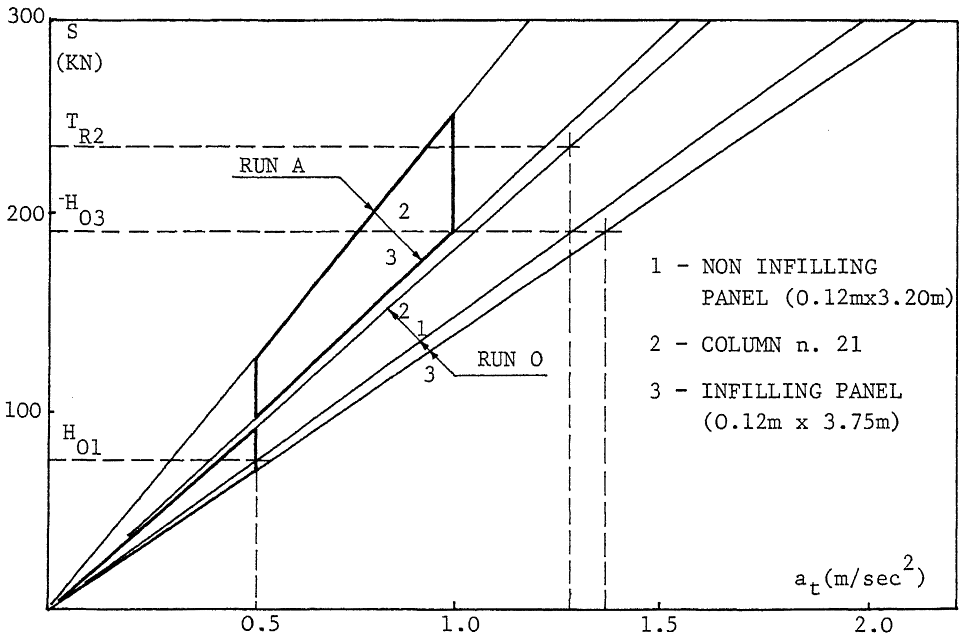


Fig. 5 - Shear forces as a function of ground acceleration, at RUN O and RUN A, on three structural elements (the first to reach collapse, for each category)