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ESTIMATION OF BASE SHEAR COEFFICIENT FOR THE CHILEAN EARTHQUAKE OF MARCH 3, 1985, BASED ON THE RESPONSE OF REINFORCED MASONRY BUILDINGS

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SUMMARY

This paper presents the results of a study to estimate the level of earthquake excitation that would justify the damage observed in three reinforced masonry buildings as a consequence of the Chilean earthquake of March 3, 1985. The estimates are given in terms of base shear coefficients for the city of Santiago, where no reliable accelerogram records were obtained during this event. Experimental data obtained from cyclic in-plane shear tests is used to estimate the capacity of the reinforced masonry walls.

INTRODUCTION

The study of structures damaged by strong earthquake ground motions has usually led to very useful conclusions about the analysis and design methods used for those structures. In this case, the damage experienced by three 4-story reinforced masonry buildings has been used to estimate the level of earthquake excitation in the city of Santiago during the Chilean earthquake of March 3, 1985, defined through a given response spectrum shape. No reliable accelerogram records were obtained in Santiago during this event, the most important that has ocurred in Central Chile in the last 80 years. Santiago, the capital of the country, has a population close to 4 million people.

The study presented herein makes use of the results of experimental research carried out in the last 15 years on the seismic behavior of reinforced masonry wall elements, of analytical research developed after this earthquake to explain the damage suffered by some buildings, and of the results of the inspection performed after this event on the three buildings considered. This allowed to estimate the base shear coefficient spectrum for this earthquake by comparing the damage observed in the reinforced masonry buildings with the results obtained from the analysis of mathematical models of them. These buildings, identified as Duble Almeyda, Los Alamos, and Emilia Tellez, were designed and constructed between 1980 and 1984, and exhibited minor damage during the earthquake.

ANALYSIS TO DETERMINE THE INTERNAL FORCES

A standard response spectrum analysis using the Chilean seismic design code spectrum (Ref. 1) was performed for each of the buildings and internal forces were computed for each of the shear walls in them. The linear elastic behavior implicit in this type of analysis is adequate since these buildings suffered only minor damage during the earthquake.

The analytical models of the buildings were selected in order to use the computer program COMBAT (Ref. 2). Several modeling issues were discussed and defined before performing the analyses; namely: actual E and G values for masonry and reinforced concrete, torsional stiffness of beams using the ACI formula, details about the consideration of masses due to dead and live loads, the use of panel elements to model perforated shear walls, and effective widths of slabs and walls. This study considered a rigid base, rigid floor diaphragm model and assumed the center of mass at the calculated plan location for all floors, without consideration of any shift caused by the live load contribution.

SHEAR STRENGTH OF MASONRY WALLS ASSOCIATED TO FIRST DIAGONAL CRACKING

This shear strength was derived from the ultimate shear strength of masonry walls, and included the effect of axial compression as explained below. The ultimate shear strength of reinforced masonry walls was derived from the experimental data obtained in the research programs carried out both at U.C. Berkeley and at the Catholic University; the details of these tests and of the data processing are described elsewhere (Ref. 3). The mathematical model to predict the ultimate shear strength v_u for walls with zero axial compression stress is shown in Fig. 1, and was selected as a lower bound of the experimental values obtained from tests where the actual axial compression stresses were between 0 and 200 psi (1.4 MPa) at the time the ultimate shear strength was attained. This ultimate shear strength is a function of the masonry compressive strength, f'_m, the aspect ratio (M/Vd) of the wall, and the amount of horizontal reinforcement. The influence of f'_m and M/Vd is represented through v_m, the UBC 85 allowable shear stress for the wall when masonry takes all the shear (Ref. 4), as indicated in Fig. 1.

The shear force V°_{cr} associated to first diagonal cracking was determined from experimental evidence obtained during the Catholic University research program. It was observed that V°_{cr} is about one half of the ultimate shear force, for walls having horizontal reinforcement ratios of the order of 0.0006 and zero axial compression stress. V°_{cr} remains practically constant for reinforcement ratios under 0.0006 and increases slightly for reinforcement ratios above 0.0006, as shown in Fig. 2. Therefore, the initial shear cracking strength for walls without axial compression was estimated as

$$v^\circ_{cr} = \frac{V^\circ_{cr}}{bd} = 1.375\ v_m \qquad \rho_h < 0.0006 \tag{1}$$

$$v^\circ_{cr} = v_m\ (1.335 + 67.1\ \rho_h)\ ,\ 0.0006 \leq \rho_h \leq 0.006 \tag{2}$$

where ρ_h is the horizontal reinforcement ratio. The shear cracking value, v_{cr}, for walls with non-zero axial compression was obtained from v°_{cr} by equating the principal tensile stresses for the cases with and without axial compression. The corresponding values of these principal tensile stresses may be obtained from a Mohr's circle analysis as

$$\sigma_1{}^\circ = \tau^\circ = 1.5\ \frac{V^\circ_{cr}}{bd} = 1.5\ v^\circ_{cr} \tag{3}$$

for zero axial compression stress, and

$$\sigma_1 = -\frac{\sigma}{2} + \left(\frac{\sigma^2}{4} + \tau^2\right)^{\frac{1}{2}} \tag{4}$$

for axial compression stress σ, where $\tau = 1.5$ V/bd $= 1.5\ v_{cr}$ is the shear stress at the neutral axis of a rectangular cross-section with dimensions b and d. Therefore, the shear cracking value for walls with axial compression stress σ may be obtained from v°_{cr} using the expression

$$v_{cr}{}^2 = (v^\circ_{cr})^2 + \frac{v^\circ_{cr} \cdot \sigma}{1.5} \tag{5}$$

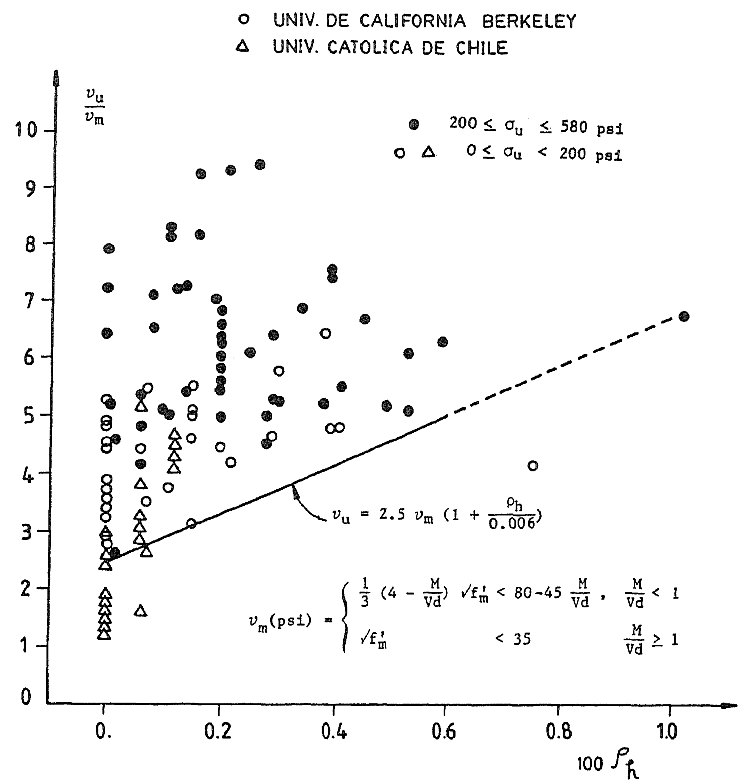

FIG. 1 ULTIMATE SHEAR STRENGTH OF REINFORCED MASONRY WALLS

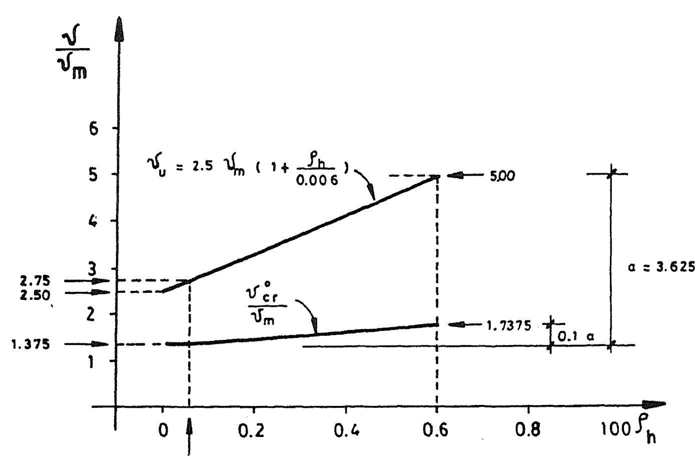

FIG. 2 SHEAR STRESS ASSOCIATED TO FIRST DIAGONAL CRACKING IN REINFORCED MASONRY WALLS

The procedure described above allowed the evaluation of the shear strength associated to first diagonal cracking, for the walls in each of the three buildings considered in this study. This required the consideration of the actual values of f'_m, ρ_h, and the compressive stress σ at each of the walls. The other significant parameter influencing the contribution of masonry to the shear strength of reinforced masonry walls is the aspect ratio of the element, represented by the M/Vd ratio. The bending moment M and the shear force V were obtained from the analysis; d is the length of the wall.

AMPLIFICATION OF THE CHILEAN CODE SPECTRUM TO JUSTIFY FIRST SHEAR CRACKING

Comparison of the shear cracking strength associated to first diagonal cracking with the shear force obtained from a dynamic analysis based on the Chilean code design spectrum (Ref. 1), allowed to compute the amplification factor needed to justify the observed damage in each of the cracked walls. The same procedure allowed to establish an upper bound for the amplification factor of the walls that exhibited no damage.

The three buildings considered in this study were thoroughly inspected after the earthquake and cracking patterns for all the walls were recorded. The comparison mentioned above was restricted to those walls that exhibited the same cracking pattern that was observed during the experimental research programs, a diagonal crack that occurs by itself or as a continuation of an horizontal flexural crack. Walls that showed horizontal cracks due to flexure or due to sliding shear, or diagonal cracks starting at a window corner, were not considered in the determination of the overall amplification factor of the building.

This factor was defined for each wall as the ratio between the shear cracking capacity available to resist earthquake forces and the shear value induced by the earthquake action V_E. The available shear cracking capacity is equal to the shear cracking capacity of the wall as given by Eq. (5) minus the shear force induced by the live loads. The shear force due to dead loads was not considered to reduce the available shear cracking capacity. Since the walls undergo small settlements and deformations as the structure is being erected, to accomodate the differences in vertical displacements due to non-uniform vertical loading, no significant shear forces are present in the walls due to dead loads alone. Therefore, the amplification factor for earthquake loading was defined as

$$F = \frac{v_{cr} \cdot d \cdot b - |V_L|}{|V_E|} \tag{6}$$

where v_{cr} is obtained from Eqs. (1), (2), and (5), d is the wall length, b is the wall thickness, and V_L and V_E are the shear forces due to live loads and earthquake action, respectively.

RESULTS OF THE STUDY

The values of the F factor for the individual walls are given in Fig. 3 for the Duble Almeyda building; the results for each of the walls are identified by wall number (defined in the floor plan of the building shown in Fig. 4), the story number (increasing from bottom), and the direction of the earthquake action considered. All the buildings were analyzed considering the earthquake acting either in the X or in the Y directions and the largest value of the shear force was taken to compute the F factor from Eq. (6). In all cases, the largest value of V_E was obtained when the earthquake was considered acting along the same direction as the length of the wall, reflecting that all three buildings had a reasonable plan distribution of resisting elements, such that no significant horizontal torsional effects developed. The fourth column of Fig. 3 shows a sketch of the degree of cracking of the wall after the earthquake, and the fifth column indicates the F factor for each of the walls.

ELEM. (AXIS)	STORY	EQ DIR	TYPE OF CRACK	FACTOR F
7(B)	1	Same	Undamaged	6.60
7(B)	2	Same	Undamaged	6.59
12(C)	1	Same	Undamaged	5.78
12(C)	2	Same	Undamaged	6.96
14(C)	1	Same	Undamaged	6.24
14(C)	2	Same	Undamaged	9.17
23(1)	1	Same	Undamaged	5.36
23(1)	2	Same	Undamaged	5.90
37(3')	1	Same	Undamaged	6.81
37(3')	2	Same	Undamaged	9.65
39(3')	1	Same	Undamaged	7.42
39(3')	2	Same	Undamaged	10.70
40(3')	1	Same	Undamaged	7.44
40(3')	2	Same	Undamaged	10.71
42(3')	2	Same	Undamaged	9.61
49(1')	1	Same		5.30
49(1')	2	Same	Undamaged	5.86

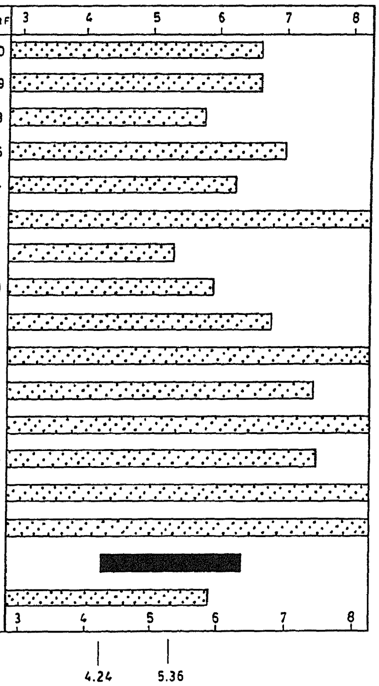

FIG. 3 DERIVATION OF AMPLIFICATION FACTOR F FOR THE DUBLE ALMEYDA BUILDING

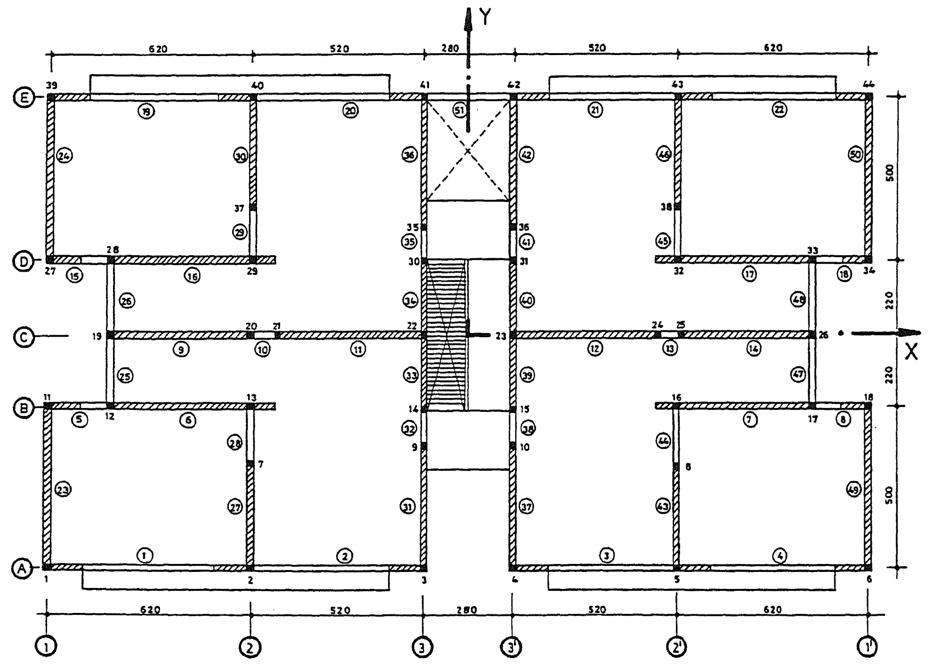

FIG. 4 TYPICAL FLOOR AND ELEMENT IDENTIFICATION FOR THE DUBLE ALMEYDA BUILDING

The determination of the F factor for each of the buildings was performed on the basis of the work done by Cruz and Luders (Ref. 5). The idea is to obtain an F factor for a building that is consistent with the factors obtained for each of the walls of this building. Two other concepts were introduced at this point, as follows:

a) An interval of confidence for the estimation of the shear cracking stress given by Eq. (5). It was assumed that this value could have a variation of $\pm 20\%$ with respect to the value indicated above, and therefore, the F factor was computed from Eq. (6) using 0.8 v_{cr} and 1.2 v_{cr}. For this reason, the wall elements that exhibited diagonal cracking show an interval of possible values shown in black in Fig. 3.

b) A degree of uncertainty in the values of V_E obtained from the dynamic analysis due to the extent of damage exhibited by the buildings. The analysis assumes that all elements maintain their original stiffness to evaluate the internal forces, but, as the walls get cracked, their stiffnesses decrease and other elements must take a larger share of the story shear; therefore, the values of V_E for the uncracked walls may have been larger than those obtained from the analysis, with a consequent reduction in the values of the F factors. An "analysis uncertainty factor" was introduced for the buildings that presented more damage than just an initial crack, Los Alamos and Emilia Tellez. This factor was set to -10% and to -30% for these buildings, respectively.

Fig. 3 shows de F factors for the most important walls of the Duble Almeyda building. It should be noted that an undamaged wall element may justify F factors up to the value corresponding to initial diagonal cracking, which are represented by the dotted field. The same analysis was performed for the other two buildings, and the following ranges for the F factors were obtained: Duble Almeyda between 4.24 and 5.36, Los Alamos between 4.21 and 4.75, and Emilia Tellez between 5.95 and 6.65. Based on the reliability of the information used to derive the F factors for the different walls, overall factors of 5.3, 4.7, and 6.6 are suggested for these buildings, respectively. Given the degree of damage and the fundamental periods of the buildings, and the Chilean design code spectrum shape, these factors correspond to elastic response spectrum base shear coefficients of 0.53, 0.47, and 0.66, respectively, associated to the damping coefficient typical of Chilean reinforced masonry buildings.

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