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BEHAVIOR OF REINFORCED MASONRY CONSTRUCTION IN THE CHILEAN EARTHQUAKE OF MARCH 3, 1985

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SUMMARY

The strong earthquake which shook Chile on March 3, 1985 severely damaged some structures. However, many structures suffered only minor damage, and some of these performed very well. One such structure, a 4-story apartment building of grouted concrete block masonry, was designed consistently with the 1985 Uniform Building Code. It is believed to have resisted spectral accelerations of at least 0.45 g, and to have remained essentially elastic. In this paper, the computed response of the building is discussed and compared against the observed behavior, and the factors contributing to its good performance are noted. Based on this analysis, it is suggested that for buildings of this type, full advantage should be taken of the following structural characteristics: a wall area ratio (in plan) of at least 2.5% in each principal plan direction; multiple walls in each direction; and the use of 0.07% horizontal reinforcement, and of about 0.20% vertical reinforcement at wall ends.

BUILDING CHARACTERISTICS AND OBSERVED DAMAGE

The Duble Almeyda building, located in the Chilean capital of Santiago, is a 4-story apartment building of fully grouted, 20-cm concrete block walls. Its plan symmetry is evident in Fig. 1. Its wall area ratio is about 6%, evenly divided in each direction. Its floor system consists of a 13-cm, cast-in-place concrete slab. The steel used has a yield strength of 2800 kgf/cm^2, and the masonry compressive strength is estimated at 120 kgf/cm^2. The building was designed consistently with the 1985 Uniform Building Code (UBC) (Ref. 1). The horizontal reinforcement percentage for the walls is typically 0.13%, more than the minimum 1985 UBC requirement of 0.07%. The vertical reinforcement percentage is typically 0.33%. About one-third of the vertical steel is placed at each end of the wall, and the rest is distributed uniformly. Total horizontal and vertical steel percentages are 0.46%, considerably higher than the UBC minimum of 0.20%. This building resisted the earthquake of March 3, 1985 with only one minor crack. This implies essentially elastic performance, surprising given the severity of the earthquake.

PROCEDURE FOR COMPARING PREDICTED WITH OBSERVED DAMAGE

The objective of this study was to correlate analytically predicted damage with that observed, in order to assess the accuracy of various analytical models, and to validate the predicted response. Analysis required establishment of the following: the seismic input; an analytical model of the building; and estimates of wall capacity as governed by shear and flexure.

Seismic Input: The building is located at an epicentral distance of about 150 km (Ref. 2). Even though many ground acceleration records were obtained elsewhere, none were available for Santiago. Strong-motion records were obtained from four nearby sites with similar observed intensities and local geologic conditions: Peldehue, San Fernando, Llay llay and San Felipe. These records were used to obtain response spectra representing the average, and the average plus or minus one standard deviation, as shown in Fig. 2. The average minus one standard deviation was taken as a lower bound to the probable spectral acceleration in Santiago. For the period range of 0.1 to 0.4 seconds and 5% damping, this lower bound corresponds to a flat spectrum with an ordinate of 0.5 g (Ref. 3).

Building Modeling: Different linear elastic models were used to predict the actual behavior of the building. All modeled the masonry walls using finite elements. The simplest model had a rigid base and floor diaphragms assumed rigid in their own planes. More complex models included soil-foundation flexibility, in-plane flexibility of floor diaphragms, and reduced flexural stiffness of walls caused by flexural cracking. Forces were estimated by spectral analysis.

Shear Capacity of Masonry Walls: Shear capacity of masonry walls was estimated as proposed by Hidalgo et al (Ref. 4):

$$v_{cr\ o} = (1.335 + 67.1\rho_h)v_m, \quad 0.0006 < \rho_h < 0.006 \tag{1}$$

where v_m, the allowable nominal shear stress of the 1985 UBC, is defined in the Nomenclature section of this paper. Adopting a principal stress criterion and including the effects of axial load (Ref. 4),

$$v_{cr}^2 = (v_{cr\ o})^2 + (v_{cr\ o})f_a/1.5 \tag{2}$$

The shear capacity of a wall with slight axial tension was considered equal to that of a wall with zero axial load. A lower bound to the ultimate shear strength was taken (Ref. 4) as

$$v_u = 2.5(1 + \rho_h/0.006)v_m \tag{3}$$

Flexural Capacity of Masonry Walls: Cracking moments and flexural capacities of the walls were calculated using beam theory. Flexural cracking was accounted for in the analytical models using a reduced flexural inertia equal to 20% of the gross inertia.

Hypothesized Damage Sequence for the Building: Using models in which flexural cracking was neglected, shear cracking was predicted at several locations. When flexural cracking was included, all the bottom-story walls were predicted to crack in flexure, and shear cracking was predicted only at the location at which it actually occurred. Based on this model, the most probable damage sequence was the following: 1) linear elastic, uncracked behavior; 2) linear elastic behavior with flexural cracking throughout the bottom story walls; and 3) formation of a diagonal tension crack in the critical wall of the first floor, corresponding to a spectral acceleration of 0.45 g.

PROBABLE RESPONSE TO A STRONG EARTHQUAKE

The Duble Almeyda building behaved very satisfactorily for this moderate earthquake. Shear cracking was minor and limited to only one wall. Judging by the absence of permanent flexural cracks, flexural yielding was avoided. The building is believed to have sufficient reserve capacity to resist a strong earthquake. As observed, and also as predicted using a 0.5-g spectral acceleration, only one wall cracked in shear. Calculated wall moments did not exceed 50% of yield. On this basis, the building would have had sufficient flexural capacity to resist twice as much spectral acceleration–that is, up to about 1.0 g. In the remainder of this section, the capacity of the building is estimated in terms of cracking and ultimate shear. While sliding shear would typically not govern and is not addressed further here, it should be checked.

Cracking Base Shear: Due to a series of conservative assumptions embodied in the 1985 UBC, the estimated base shear required to cause diagonal cracking in the walls of the Duble Almeyda building far exceeds the UBC design shear. Using the nomenclature of the 1985 UBC, design shears for the walls of the Duble Almeyda building would be obtained using a soil-modified seismic coefficient CS of 0.19, and would be amplified by a factor of 1.5. Using Equation (1), the shear required to cause first cracking exceeds the UBC allowable by a factor of (1.434 v_m / 1.333 v_m), or 1.08. In a building with a 6% wall area ratio in plan, the shear stress in the critical wall rarely approaches the Code limit. In the Duble Almeyda building, for example, this architectural safety factor is about 1.15. As a net result, the cracking base shear for this building should equal its weight multiplied by (0.19 x 1.5 x 1.08 x 1.15), or 0.35 times its weight. Considering that the total weight of this building exceeds its effective weight in the fundamental mode by a factor of about 1.2, the Duble Almeyda building would be expected to crack in shear at a spectral acceleration of about (0.35 g x 1.2), or 0.43 g.

Ultimate Base Shear: If a building's wall layout can adequately redistribute lateral forces after cracking, and if no intervening brittle failure modes occur, the building can resist higher lateral forces than those required to cause shear cracking. Using the Equation (3), it can be seen that the shear capacity of a typical Duble Almeyda wall exceeds the UBC design shear by a factor of (2.5 x 1.22 v_m / 1.33 v_m), or 2.29. Using the same architectural safety factor as before, the ultimate base shear for this building should equal its weight multiplied by (0.19 x 1.5 x 2.29 x 1.15), or 0.75 times its weight. The corresponding ultimate shear capacity would be reached at a spectral acceleration of about (0.75 x 1.2) or 0.90 g. This estimate is conservative (low) because it assumes that a building's ultimate shear capacity is limited by the shear capacity of its critical wall. This is not true for buildings with multiple walls, adequately arranged.

INFLUENCE OF WALL REINFORCEMENT

The critical wall of the Duble Almeyda building has a horizontal reinforcement percentage of 0.13%, about twice the 1985 UBC minimum of 0.07%. Using the formulas of Ref. 4 to predict cracking shear and ultimate shear capacity, it is evident that cracking shear is not sensitive to the amount of horizontal steel. If the 1985 UBC minimum had been used in this case, the cracking shear would have decreased by only about 4%. Nevertheless, minimum horizontal steel is necessary to control the width of diagonal cracks, and can approximately double the shear capacity of a masonry wall.

A moment-curvature analysis, conducted on the critical wall of the Duble Almeyda building (using actual steel properties), reveals the importance of vertical reinforcement. This wall had a vertical reinforcement percentage of 0.33%, about one-third of which was placed at each end of the wall. In this moderate earthquake, this amount and distribution of vertical steel limited the stresses in steel and masonry to the elastic range, permitting reversed cyclic response without strength or stiffness degradation. In contrast, if a 1985 UBC minimum steel percentage of 0.14% had been used, distributed uniformly, moment-curvature analysis indicates that the critical wall would have experienced yielding of vertical steel, permanent visible flexural cracks, and compressive failure of the wall toe.

CONCLUSIONS

The satisfactory behavior of the Duble Almeyda building in the March 3, 1985 Chilean earthquake points out the importance of the following aspects of seismic-resistant design for masonry bearing wall buildings:

1) for moderate earthquakes, satisfactory and essentially elastic response is possible if the total wall area ratio is kept above 5%.

2) The walls should be arranged so as to inhibit plan torsion, and the floor diaphragms should be sufficiently rigid in their own planes to produce comparable shears in walls of similar lateral stiffness.

3) Elastic behavior of walls can be promoted by the use of 0.07% horizontal reinforcement to control the growth of diagonal cracks, and by the use of about 0.20% vertical reinforcement at wall ends to control flexural cracking, avoid flexural yielding and fracture, and prevent compression toe failures.

If these recommendations are met, and if secondary failures (such as sliding shear at construction joints) are avoided, buildings similar to the Duble Almeyda building should have enough inherent redundancy to resist even larger earthquakes. In this particular case, the Duble Almeyda building is believed able to resist a spectral acceleration of 0.45 g before cracking in shear, and a spectral acceleration of at least 0.90 g before reaching its ultimate shear capacity. Predicted high strengths such as these are consistent with the observed satisfactory performance of many engineered masonry buildings in moderate and large earthquakes, and suggest that low-rise masonry wall buildings are practical and safe, even in highly seismic regions.

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NOMENCLATURE

$v_{cr\ o}$ shear stress corresponding to first diagonal cracking of a masonry wall, no axial load (Ref. 4)

ρ_h horizontal steel ratio

v_m allowable shear stress assuming masonry takes all shear, Formulas 6-10 and 6-11, 1985 Uniform Building Code (Ref. 1)

v_{cr} shear stress corresponding to first diagonal crack, axial load included (Ref. 4)

f_a average wall compressive stress

v_u ultimate shear capacity of a masonry wall (Ref. 4)

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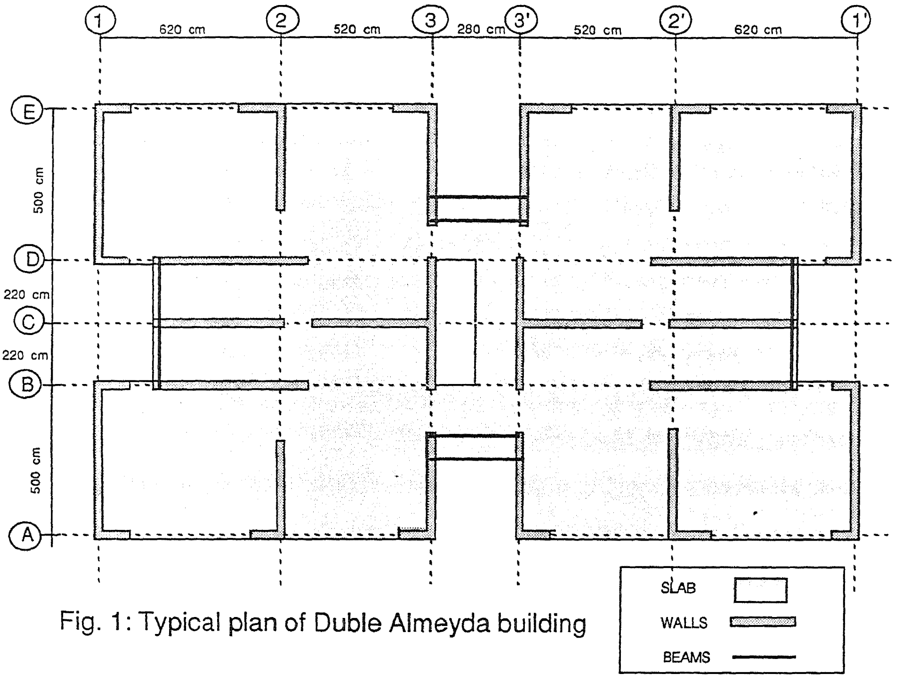

Fig. 1: Typical plan of Duble Almeyda building

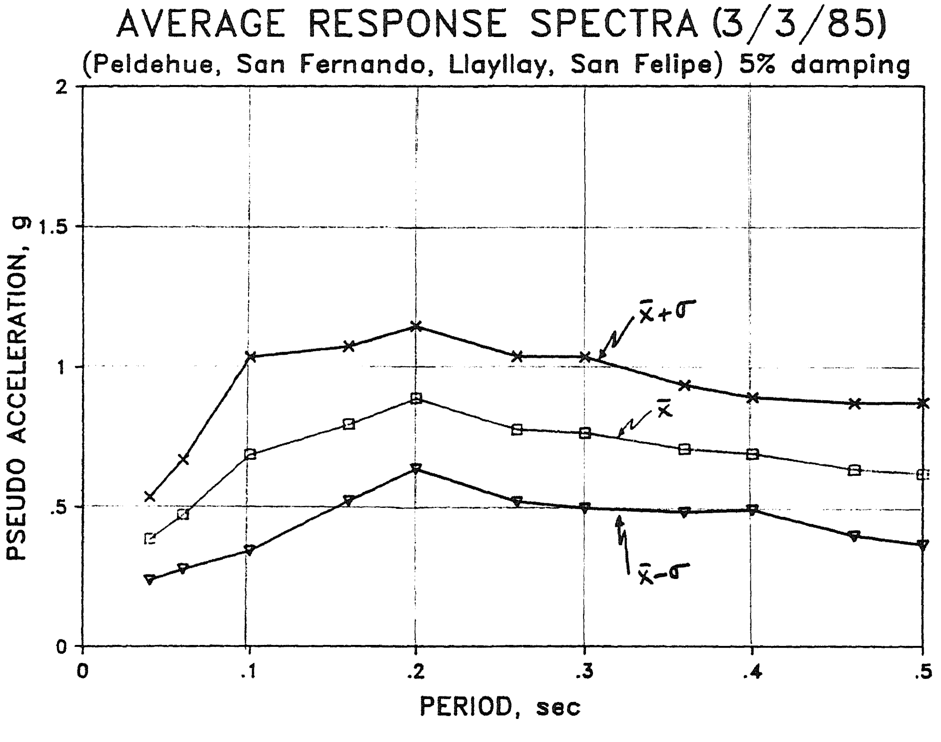

Fig. 2: Average spectral estimates for Santiago (March 3, 1985)