EARTHQUAKE-RESISTANT DESIGN OF REINFORCED MASONRY BUILDINGS

P.A. Hidalgo (I)
C. Lüders (I)
Presenting Author : P.A. Hidalgo

SUMMARY

This paper summarizes the results of research programs that have been carried out in order to adapt the Uniform Building Code provisions to the design of reinforced masonry buildings in Chile. These programs include the cyclic shear tests of cantilever piers, the compressive strength of prisms with different height-to-width ratios, the prediction of the strength associated with the shear mode of failure of the piers, and an experimental study on the masonry stress-strain relationship. The results obtained are used in an analytical study to evaluate the seismic design provisions of the proposed chilean specifications for the design of masonry buildings.

INTRODUCTION

The use of reinforced masonry in chilean building construction is relatively recent. Despite isolated efforts made during the past twenty years, it was not until six or seven years ago that a number of buildings up to four stories high began to be erected in the residential areas of our cities. The increasing volume of this type of construction forced the Ministry of Housing and Urban Development to ask for a set of specifications for the design and construction of reinforced masonry buildings. The Technical Specification 20/81 (Ref. 1) appeared in late 1981 and has become the draft of the chilean code that is presently under study. The provisions contained in this specification were written on the basis of Chapter 24 of the Uniform Building Code (Ref. 2). Nevertheless, masonry units, mortar, grout and workmanship used in Chile may vary significantly from those used in the United States. The research work presented in this paper and that developed by others in Chile intends to validate some of the research that constitutes the background of the UBC provisions and develop an improved set of recommendations to protect masonry structures against severe damage or collapse during an earthquake.

CYCLIC TESTS OF CANTILEVER PIERS

Previous research (Ref. 3) has shown that the pier is one of the main structural components present in shear wall panels. Twelve cantilever piers constructed from clay brick units type A (Fig. 1), with overall dimensions 1.02 m (40.2 in) high by 1.04 m (40.9 in) wide, were tested under cyclic, in-plane shear loads using the test setup shown in Fig. 2. The parameters of these tests included the amount of vertical reinforcement and the type of grouting; all the specimens had no horizontal reinforcement and had no vertical load acting on the piers. The amount of reinforcement was designed to control the mode of failure; the grouting of the cells was done with the same mortar used in the joints at the same time the units were being laid; full shoved mortar

(I) Professor of Civil Engineering, Universidad Católica de Chile, Santiago, CHILE.

joints, except for the cells, was used; partially grouted piers had only the cores containing reinforcement filled, while fully grouted piers had all the cells filled with mortar. The lateral loading sequence for each test consisted of sets of two displacement cycles applied at a specified actuator displacement amplitude; this amplitude was gradually increased until failure was obtained. The basic product obtained from the tests was the hysteresis loops diagram, (Fig. 3). The vertical reinforcement used, the strength and deformation properties of the piers and the compressive strength of the prisms associated with each pier are summarized in Table 1.

The results of this experimental program confirm the main findings obtained by previous research (Ref. 3). Typical force-displacement curves obtained during the tests are shown in Figs. 3a and 3b for the flexural and the shear modes of failure, respectively. The concentration of the flexural cracks at some particular joints showed that poor bond was developed between reinforcing bars, grout and masonry units; this effect is due to the material used as grout (mortar), and the technique used to fill the cells. This loss of bond was also responsible for experimental lateral displacements larger than the theoretically calculated values. It is also interesting to observe that the presence of the top reinforced concrete beam to anchor the vertical reinforcement, influenced the crack pattern of the piers. A new test setup that will eliminate the beam at the load level is presently being used in the continuing pier test program.

PREDICTION OF SHEAR STRENGTH

One of the parameters that influences the ultimate strength of the piers associated with the shear mode of failure is the quality of masonry, represented by the prism compressive strength f'_m. The UBC specifies a prism with height-to-thickness (or minimum prism dimension) of 2 to determine the value of f'_m and gives correction factors for prisms with other height-to-thickness ratios (Ref. 2). An experimental program of prism tests including all the masonry units shown in Fig. 1 and height-to-thickness ratios varying from 1 to 6 was carried out, considering both grouted and non-grouted specimens. In the case of the clay brick prisms, grouting was done with the same mortar used for the joints at the same time the units were being laid; for the grouted concrete block prisms, the cores were filled with pea-gravel grout when the specimen had its full height. Full shoved mortar joints and face-shell mortar was used for the clay brick and concrete block prisms, respectively. The result of the compressive strength f'_m has been plotted in Figs. 4a and 4b, where each point represents the average strength of three specimens. It can be observed they do not agree with the UBC correction factors, since these results indicate the prism strength is relatively constant for height-to-thickness ratios between 2 and 5. In the present chilean code under study it has been suggested the use of a prism with a height-to-thickness ratio of 3 to obtain the value of f'_m. Another conclusion of these tests is the difference in improved strength due to grouting between clay brick and concrete block prisms; the smaller improvement for the clay brick prisms is due to the core size and the use of simple mortar as grouting material, (Ref. 4).

The experimental results of cantilever piers that failed in the shear mode of failure are shown in Table 1. Figure 5 shows a comparison between these results and those obtained at the University of California, Berkeley, (Ref. 3), using fixed ended piers without shear reinforcement; masonry units, grout and workmanship used in both programs were quite different, as well as test specimen and the device used to anchor the vertical reinforcement. Figure 5 shows consistency between both sets of results; a continuing pier test program now in progress will provide more information on this subject and will eventually confirm the decision of adopting the UBC provisions as the basis for the chilean code on reinforced masonry.

STRESS-STRAIN RELATIONSHIP

In spite of the fact that most masonry codes are presently based on the linear elastic assumption of the material behavior, the tendency is to develop design provisions based on the ultimate strength of structural elements. This will require a reliable analytical model of the stress-strain relationship for masonry, specially when the ultimate flexural strength of beams or shear walls needs to be evaluated. The results of two research programs to study the masonry stress-strain relationship are presented.

The first program worked with 52 stress-strain experimental curves obtained under axial compression from prism tests included in the study reported on Ref. 5. All the prisms had a height-to-thickness ratio of about 5, included all the masonry units of Fig. 1 plus a concrete block similar to type D but 0.20 m (8 in) high, and considered non-grouted prisms for all the masonry units and grouted specimens for the concrete block prisms. The experimental stress-strain curves indicate that the secant modulus of elasticity at 0.45 f'_m may be approximated by 700 f'_m for clay brick and non-grouted concrete block masonry, and by 800 f'_m for grouted concrete block masonry, (Ref. 5).

Besides, the curves show it is unsafe to rely on masonry strength for compressive strains ε larger than ε_0, where ε_0 is the strain at the ultimate compression capacity (f'_m), that varied between 0.0015 and 0.0035. Then, a study to obtain the analytical model that best fits the experimental data was undertaken, using the least squares method. Several models were tested and the best result was obtained with the second degree parabola that has zero slope at $f = f'_m$, $\varepsilon = \varepsilon_0$.

The second program also measured experimentally the stress-strain relationship, but used a compressive stress block with linearly varying strains from zero up to a maximum value; in order to do this, nine prisms built with clay brick units type A (Fig. 1) and mortar grouting, were tested under eccentric compression using the test setup shown in Fig. 6. At the same time the load R was being applied, the spring force Q was manually adjusted so that the strain at the interior face of the prism was kept at zero value; the reading of the zero strain was obtained from a strain-gage, while the maximum strain at the opposite face was recorded using a LVDT; the values of R and Q were obtained continuously from force transducers, and permitted to determine the position and the magnitude of the stress resultant over the prism cross section. The least squares method was used to determine the parameters of a Ramberg-Osgood type stress-strain relationship that best fits the experimental values of the magnitude and the position of the stress resultant for every value of the maximum compressive strain of the cross section of the prism. The result of this adjustment is shown in Fig. 7 and compared with the stress-strain curve obtained from similar axial compression tests. The curves are quite similar, showing that the distribution of cells of this type of masonry unit prevents the contribution of the less stressed fibers to the deformation capacity of the extreme fibers of the section, (Ref. 6).

EVALUATION OF CHILEAN SEISMIC DESIGN PROVISIONS

The method used to evaluate the provisions contained in the draft of the chilean code (Ref. 1) for the design of masonry structures is based on the method developed by Mayes et. al. at the University of California, Berkeley, (Ref. 7). This method compares the area of masonry required at a particular element as given by a particular code with that required to survive a realistic earthquake. The determination of the latter area is the most difficult aspect of a study of this kind, since it requires to define the design spectrum for a severe, once in a lifetime ground motion that may affect the region where the code applies; moreover, the ultimate capacities of the masonry elements, both strength and deformation, need to be defined.

This study made the comparison mentioned above using the area of the masonry walls required to resist the shear earthquake forces, because this is the structural element that seems to control the seismic behavior of masonry buildings. The realistic earthquake excitation was assumed as the elastic design spectrum for a ground motion approximately 25% more intense than that recorded in Santiago on July 8, 1971; this earthquake had an intensity of VI (Modified Mercalli Scale) in Santiago and X near the epicenter, which was located off the coast 120 km. away from Santiago. The elastic design spectrum was constructed from the ground motion spectrum using the coefficients of Ref. 8. The ultimate shear strength of masonry walls was based on the experimental results obtained at the University of California, Berkeley, (Ref. 3), and the pier test results reported above, and was taken as

$$v_u = 2v_{nr} \left(1 + \frac{\rho}{0.006}\right), \qquad \rho \leqslant 0.006$$

where ρ is the horizontal reinforcement ratio and v_{nr} is defined in Fig. 5. The ductility factor associated with the structure seismic behavior was assumed to be 1.0, (Ref. 9).

The comparison of the masonry wall areas was made on a wall by wall basis of a four story clay brick masonry building, which is typical of the chilean residential building construction. The results of this comparison for the bottom story walls indicate the allowable shear stresses of Ref. 1 are non-conservative; these allowable stresses are the same included in the UBC (Ref. 2). It should be noted the draft of the chilean masonry code uses a factor of 2.0 to increase the shear stresses obtained from the earthquake loads provided by the chilean earthquake resistant design code. The scope of this study is presently being expanded to include buildings with a larger number of stories and different floor plans, other types of masonry construction, realistic earthquakes of different intensities and the consideration of moderate ductility factors in the structural response, before a final decision is made concerning the allowable shear stresses of the chilean code.

CONCLUSIONS

1. The prism strength is relatively constant for prism height-to-thickness ratios between 2 and 5.

2. The prism strength f'_m is a good indicator of the influence of masonry qual

ity in the shear strength of reinforced masonry piers.

3. The stress-strain relationship for masonry under axial compression is adequately represented by a second degree parabola with zero slope at $f = f'_m$, $\varepsilon = \varepsilon_0$. The ultimate strain is not larger than ε_0.

4. For the clay brick units used in Chile, the stress-strain relationship under linearly varying strains is similar to that obtained under axial compression.

5. The UBC based allowable shear stresses for shear walls are non-conservative when used in conjunction with the lateral loads provided by the chilean earthquake resistant design code, with a 100% amplification.

ACKNOWLEDGEMENTS

The research reported in this paper has been funded by the Research Division of the Universidad Católica de Chile with the collaboration of the companies PRINCESA and INEIN Soc. Ltda. of the masonry industry. The sponsorship of these institutions is gratefully acknowledged. Thanks are also due to Prof. L. Cifuentes and Messrs. W. Roldán and C. Gárate who participated in some of the studies described above.

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TABLE 1. TEST RESULTS FOR CANTILEVER PIERS (A_g = 1456 cm^2 = 225.7 in^2)

SPECIMEN	GROUTING	VERTICAL REINFORCEMENT			MAXIMUM SHEAR STRESS $v_{max} = \frac{V_{max}}{A_g}$ (MPa)		LATERAL DISPLACEMENT AT MAXIMUM SHEAR δ_{max} (mm)		PRISM COMPRESSIVE STRENGTH (MPa)
		N° Bars	Yield Strength (MPa)	$\rho_v = \frac{A_{vs}}{A_g}$	POS.	NEG.	POS.	NEG.	
TC-1-01	Full	2-12 mm	420	0.00155	0.34 (1)	0.34 (1)	6.5	9.4	6.76
TC-1-02	Full	2-12 mm	420	0.00155	0.35 (1)	0.38 (2)	4.5	>20.0	6.76
TC-1-03	Full	2 - 8 mm	280	0.00069	0.13 (2)	0.13 (2)	7.5	9.6	6.76
TL-1-04	Partial	2-12 mm	420	0.00155	0.29 (3)	0.24 (1)	12.2	6.2	4.08
TL-1-05	Partial	2-12 mm	420	0.00155	0.22 (1)	0.27 (1)	7.0	7.2	4.08
TL-1-06	Partial	2 - 8 mm	280	0.00069	0.11 (2)	0.12 (2)	6.0	9.0	4.08
TL-1-07	Full	2-12 mm	420	0.00155	0.33 (1)	0.27 (1)	5.6	3.6	7.04
TL-1-08	Full	2-12 mm	420	0.00155	0.25 (1)	0.24 (1)	3.5	2.1	7.04
TL-1-09	Full	2 - 8 mm	280	0.00069	0.14 (2)	0.13 (2)	10.0	10.0	7.04
TL-1-10	Full	2-10 mm	280	0.00110	0.20 (2)	0.20 (2)	>17.0	>17.0	7.04
TL-1-11	Full	2-12 mm	420	0.00155	0.29 (1)	0.27 (1)	6.3	5.7	9.46
TL-1-12	Full	2-12 mm	420	0.00155	0.34 (2)	0.30 (1)	>17.0	5.2	9.46

Mode of failure: (1) Shear, (2) Flexure, (3) Sliding

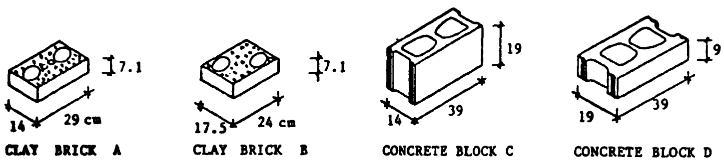


FIG. 1. MASONRY UNITS

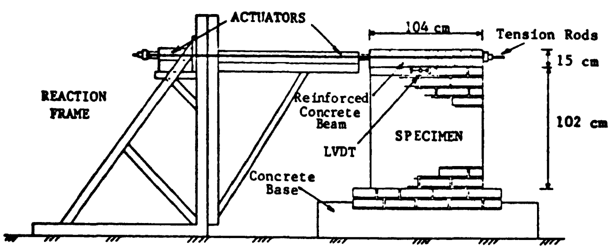


FIG. 2. TESTS SETUP FOR CANTILEVER PIERS

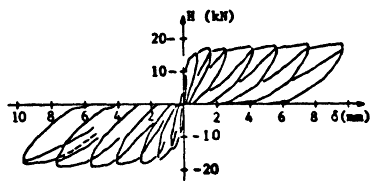


FIG.3(a) FORCE - DISPLACEMENT HYSTERETIC CURVE FOR SPECIMEN TL-1-03

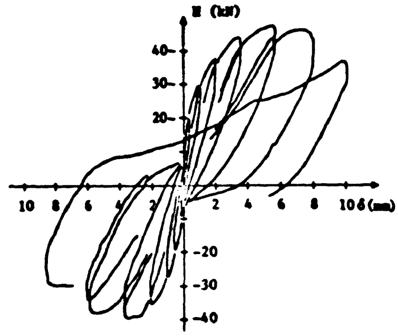


FIG.3(b) FORCE - DISPLACEMENT HYSTERETIC CURVE FOR SPECIMEN TL-1-07

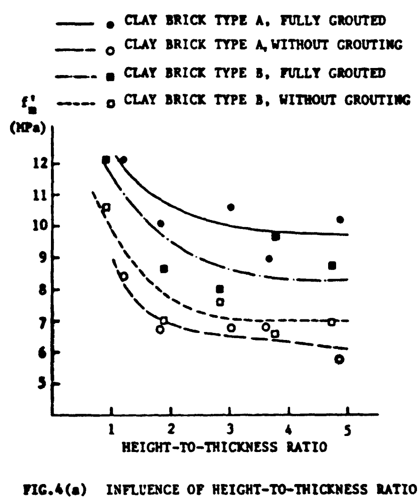


FIG.4(a) INFLUENCE OF HEIGHT-TO-THICKNESS RATIO ON CLAY BRICK PRISM COMPRESSIVE STRENGTH

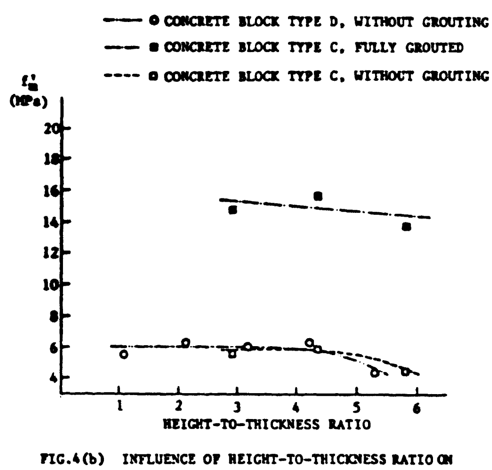


FIG.4(b) INFLUENCE OF HEIGHT-TO-THICKNESS RATIO ON CONCRETE BLOCK PRISM COMPRESSIVE STRENGTH

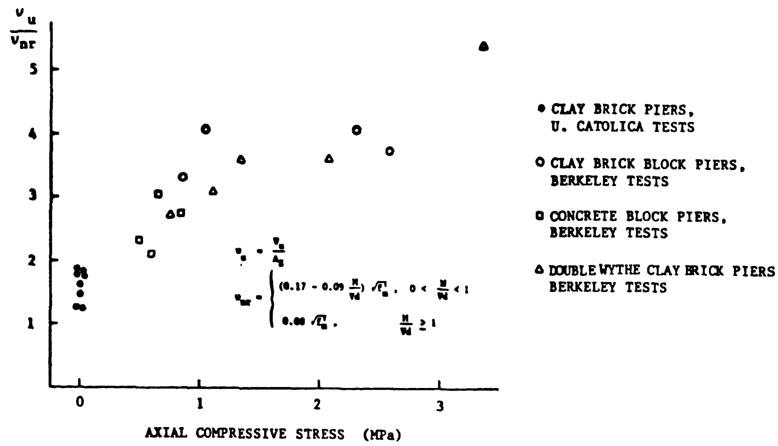


FIG. 5. SHEAR STRENGTH OF PIERS WITHOUT HORIZONTAL REINFORCEMENT

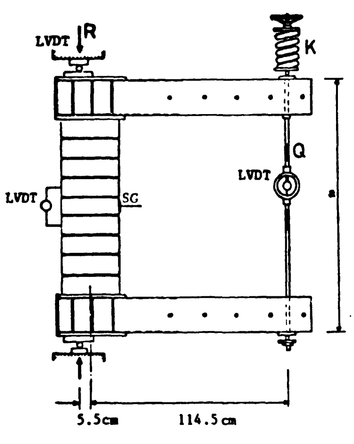
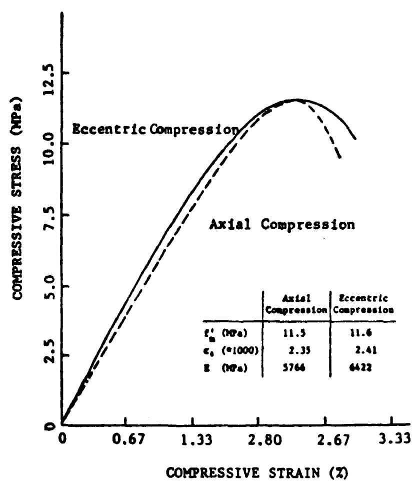


FIG. 6. TEST SETUP FOR PRISM ECCENTRIC COMPRESSION TESTS



	Axial Compression	Eccentric Compression
f'_m (MPa)	11.5	11.6
ϵ_o (*1000)	2.35	2.41
E (MPa)	5766	6422

FIG. 7. COMPARISON OF STRESS-STRAIN RELATIONSHIP FOR AXIAL AND ECCENTRIC COMPRESSION TESTS OF CLAY BRICK MASONRY