Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.V)

7-6-18

INELASTIC SEISMIC ANALYSIS OF REINFORCED MASONRY BUILDINGS

Juan CASSIS[1], María O. MORONI[1] and Marcello MIGONE[1]

[1]Department of Civil Engineering, University of Chile,
Santiago, Chile.

SUMMARY

Based on the observed damage on reinforced masonry buildings after the Chile earthquake of March 3, 1985, characterized by shear cracks on walls, a model to predict the inelastic response of those buildings is developed.

In this approach, a model of a reinforced masonry wall that behaves elastically in bending and inelastically in shear is proposed with the aim of having shear failure under a severe earthquake. The parameters of the model have been determined by experimental tests. The model is used in the inelastic dynamic analysis of a tipical three story reinforced masonry building subjected to the March 3, 1985 Chile earthquake (Melipilla N-S component).

INTRODUCTION

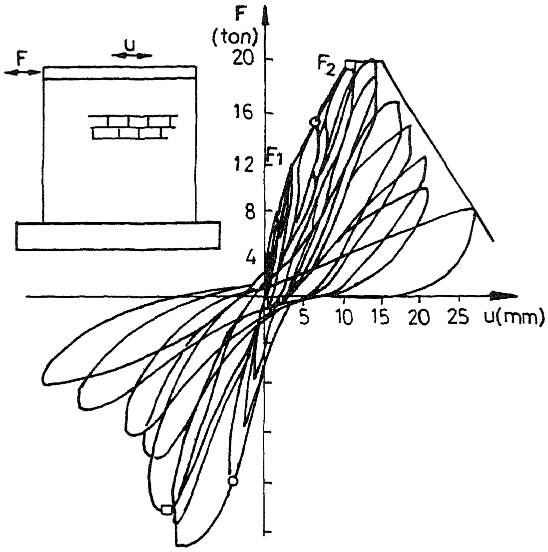
The observed behavior of some reinforced masonry buildings during the Chile earthquake of March 3, 1985, showed the necessity of improving the procedures for the analysis and design of this kind of structures. On the other hand, damage was mostly due to shear cracking. These considerations led to try to develop a model of analysis capable of predicting inelastic behavior and level of damage due to severe earthquakes in reinforced masonry buildings. It should also provide relevant data to revise the design codes.

As a basis for the present study the models and propositions given by Refs. 1,2,3,4 have been considered.

THEORETICAL MODEL

The parameters that characterize the theoretical model for a reinforced masonry wall have been mainly obtained from tests performed in Chile (Refs. 5,6). Fig. 1 shows the load-displacement curve corresponding to a test reported in Ref.5. The load is lateral and applied cyclically. From this plot, an envelope curve of the load cycles can be obtained.

The envelope can be represented by the composite curve shown in Fig.2, which depends on the parameters F_1, F_2, u_1, u_2, u_3 and α.



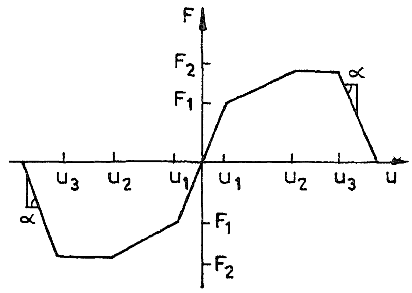


Fig.1. Load-displacement Curve for a Wall

Fig.2. Envelope of F-u Cycles

These parameters essentially depend on the masonry quality, the wall dimensions, the amount and location of the steel reinforcement, the presence of vertical load. From experimental tests (Refs. 5,6), they have the following expressions :

$$F_1 = (0.37 \frac{M}{Vd} + 0.1)\ F_2 \qquad (1)$$

$$\frac{F_2}{A} = \begin{cases} 2(1 + \frac{\rho}{0.006})\ (0.53 - 0.29\frac{M}{Vd})\ \sqrt{f'm} & \text{if } 0 < \frac{M}{Vd} < 1 \\ 0.48\ (1 + \frac{\rho}{0.006})\ \sqrt{f'm} & \text{if } \frac{M}{Vd} \geqslant 1 \end{cases} \qquad (2)$$

where M is the maximum bending moment due to the lateral load in the wall, V is the shear force, d is the height of the wall cross section, A is the cross sectional area, ρ is the horizontal reinforcement ratio, and f'm is the prismatic compressive strength of masonry in kgf/cm². F_2/A is expressed in kgf/cm².

The maximum strength F_2 increases with axial load N according to the formula (Ref. 3) :

$$F_{2N} = F_2 + 0.4\ N \qquad (3)$$

The displacement parameters have the following expressions :

$$u_1 = \frac{F_1}{K} \qquad (4)$$

where K = 1.2 h/AG is the initial stiffness of the wall, h is the height of the wall and G is the shear modulus.

If there exists horizontal reinforcement

$$u_2 = \left(-\,4.51\,\frac{M}{Vd} + 10.81\right) u_1 \qquad (5)$$

if not

$$u_2 = \left(-\,6.17\,\frac{M}{Vd} + 14.52\right) u_1 \qquad (6)$$

$$u_3 = u_2 \qquad \text{if } \frac{M}{Vd} > 2 \qquad (7)$$

$$u_3 = \left(-\,0.38\,\frac{M}{Vd} + 1.76\right) u_2 \qquad \text{if } \frac{M}{Vd} \leqslant 2 \qquad (8)$$

The value of α was estimated as

$$tg\alpha = -\,\frac{F_1}{u_1} \qquad (9)$$

The wall model is an element having a shear spring K_S that behaves as shown in Fig. 2; this spring is coupled with a flexible bar with linear elastic flexural characteristics E and I. It also has rigid ends to represent joints with large dimensions. A similar element, applied to reinforced concrete members, was introduced in Ref. 1. Fig.3 shows the element and its deformed position.

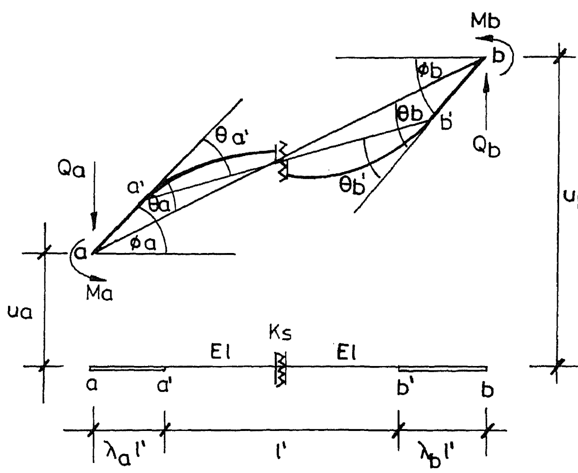


Fig.3. Finite Element of the Structural Model

The stiffness matrix, referred to the nodal displacements $\varphi_a, \varphi_b, u_a, u_b$ (see Fig. 3), is :

$$\begin{bmatrix} r(3\lambda_a^2 + 3\,(1+\lambda_a)^2 - q) & & \text{Sim} & \\ r(3(\lambda_a+\lambda_b+2\lambda_a\lambda_b) + q) & r(3(1+2\lambda_b+2\lambda_b^2) - q) & & \\ r\,\dfrac{3+6\lambda a}{\ell'} & r\dfrac{3+6\lambda b}{\ell'} & \dfrac{6r}{\ell'^2} & \\ -r\,\dfrac{3+6\lambda a}{\ell'} & -r\dfrac{3+6\lambda b}{\ell'} & -\,\dfrac{6r}{\ell'^2} & \dfrac{6r}{\ell'^2} \end{bmatrix}$$

where $r = \dfrac{6EI}{\ell'}\;\dfrac{1}{9-6q}$, $q = 1-p$, $p = \dfrac{6\,E\,I}{K_S\,\ell'^2}$

SEISMIC ANALYSIS OF A 3 STORY RM BUILDING

A 3 story reinforced masonry building that typifies existing buildings is analyzed using the element of Fig. 3. A typical plan and a couple of elevations are shown in Fig. 4.

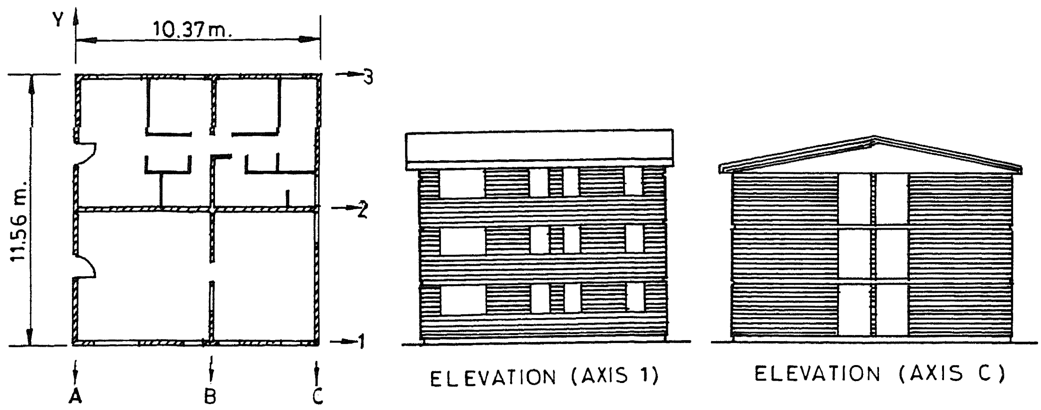


Fig.4. Plan and Elevations of the 3 Story RM Building

All the floors are of reinforced concrete with 0.12 m thickness. All the walls have 0. 14 m thickness. The story height is 2.47 m. An accelerogram from the March 3, 1985 Chile earthquake (Melipilla N-S component) was used as seismic load (representative of a severe earthquake). A step by step dynamic analysis using the computational program DRAIN-TABS (Ref.7) was performed in each direction x and y. The finite element developed in this work was incorporated in that program. The masonry characteristics are:$f'm$=13.76 MPa, E=4000 MPa, G=1200 MPa.

From the analysis, the building cracking history was obtained. Although it gave a generalized cracking (F_1 was exceeded), maximum strength F_2 was never reached. In fact,some walls absorbed up to 80% of F_2. The greatest damage was produced at base level, which agrees with what was observed after the earthquake.

To assess the adequacy of the force amplification factors for working stresses prescribed in the Reinforced Masonry Design Chilean Code (Ref.8), a linear-elastic dynamic analysis of the building was carried out. If gave wall cross sectional areas smaller than the minimum established by architecture : 0.14 thickness. With these areas the force F_2 would have been reached.

CONCLUSIONS

Supported by experimental evidence, a useful analytical model to predict the inelastic seismic behavior of reinforced masonry buildings has been introduced. It can be used to compute levels of damage produced by severe earthquakes and compare them with observed damage. In addition, it may provide relevant data for design.

ACKNOWLEDGMENTS

The authors thank the Departamento Técnico de Investigación (Research Tech nical Department), Univ. of Chile, Project I 2462-8722, and FONDECYT, Project 1195/88, for the financing of this work.

REFERENCES

1. Shiga,T., Shibata, A.,Shibuya, J., Takahashi, J., and Sakate, N, "FRAMED-D a Computer Program to Analyze Inelastic Earthquake Response of R/C Frames", Laboratory of Structural Engineering, Dept. of Architecture, Fac. of Engineering, Tohoku University, Japan, (1985).
2. Meli, R., "Comportamiento Sísmico de Muros de Mampostería", Instituto de Ingeniería, UNAM, México, (1975).
3. Meli, R. and Bazan, T., "Muros de Mampostería ante Cargas Laterales", Estudios Analíticos. Ciudad Universitaria, México, (1980).
4. Caprari, A. and Decanini, L., "Evaluación de la Resistencia Sísmica de Estructuras de Mampostería. Aplicación del Programa POR y Comparación con Criterios Normativos", 2as. Jornadas Argentinas de Ingeniería Estructural, Argentina, 35 p., (1982).
5. Diez, J., "Estudio Experimental de Muros de Albañilería Sometidos a Carga Lateral Alternada", Civil Eng. Thesis, Univ. of Chile, (1987).
6. Hidalgo, P., Lüders, C. and Jordán, R., "Diseño Sísmico de Edificios de Albañilería Armada en Chile", 4as. Jornadas Chilenas de Sismología e Ingeniería Antisísmica, Viña del Mar, Chile, I 123 - I 140, (1986).
7. Guendelman, R. and Powell, G.H., "DRAIN-TABS a Computer Program for Inelastic Earthquake Response of Three Dimensional Buildings", Earthquake Engineering Research Center, Report EERC 77/8, University of California, Berkeley, (1977).
8. Instituto Nacional de Normalización, "Albañilería Armada. Requisitos para el Diseño y Cálculo", Norma NCh 1928, of.1986, Santiago, Chile, (1986).