Experimental analysis of one-tenth scale reinforced concrete model of a ten storey building

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ABSTRACT: Results of tests conducted on small-scale reinforced concrete models are presented in this paper. Models of vertical cantilever beams, isolated walls and a whole earthquake resistant system of an actual building that withstood the March 3, 1985 earthquake in Viña del Mar (Chile), were constructed at a scale 1:10 and tested under reverse cyclic loads.

Microconcrete was used in constructing the test specimens, whereas the model for the steel reinforcement was manufactured from steelwire.

Deformed wires were normalized by a method of heating in oven and air cooling to obtain desired stress-strain properties of sharp yielding, level yield plateau and strain hardening.

A good correlation was obtained for hysteresis cycles between full scale isolated cantilever beams and one-tenth scale models. Crack patterns were very similar.

Limit analysis was used to calculate load-displacement relationships and maximum base shear capacity of the whole building before testing. Test results were compared with associated analytical predictions. Considering the crack pattern of the actual building, a lower bound of strength was estimated from test observations and computer calculations.

1 INTRODUCTION

An earthquake of surface magnitude 7.8, which ocurred on March 3, 1985, off the coast of Central Chile, caused extensive damage and casualties in central Chile and to coastal cities, particularly due to the failure of low rise adobe and unreinforced masonry dwellings. In contrast, most of the modern engineering structures performed reasonably well considering the energy content of the input ground motion. One remarkable feature was the exceptional performance of most of the more than 400 medium to medium-high rise buildings in the coastal city of Viña del Mar, located some 110 Km from Santiago and 80 Km from the epicenter.

Results of analytical and experimental analysis of one of the buildings which withstood the above mentioned earthquake in Viña del Mar are presented in this paper.

2 BUILDING DESCRIPTION

The Villa Real complex consists of a ten story residential building, a subterranean parking structure that surrounds the building on three sides, and a small commercial center adjacent to the remaining side of the building. The building design
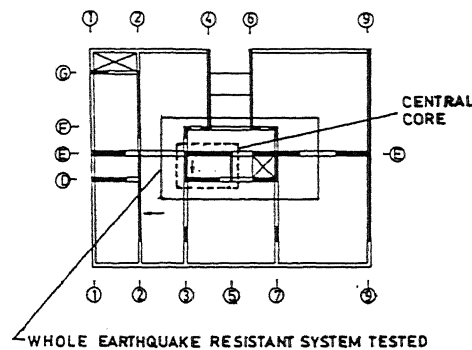

WHOLE EARTHQUAKE RESISTANT SYSTEM TESTED

Figure 1. Plan configuration of the earthquake resistant system on the Villa Real building.

was finished in 1981 and the construction was completed in 1983.

The main structure is a slab-girder wall system, founded on a 70 [cm] mat-type foundation. The layout of the earthquake resistant system of the building is shown in Figure 1, there the area studied in 1:10 scale model is included. Figure 2 shows building cross-section sets and indicates cracks observed in the actual building after the earthquake, for the N-S direction effect. Structural damage due to the earth-

quake was minimal and was concentrated near the base of the structure. Diagonal cracking was observed in the structural walls of the first, second, and third stories with most cracks measuring less than 0.2 [mm] wide. Aditional information related to the building dimensions, including the specified material properties, is listed in Table 1.

Walls thickness ranges from 20 to 30 centimeter near the base of the structure. On the other hand, most of the walls in the upper stories are 20 cm thick. Each wall contains two curtains of uniformly spaced orthogonal steel reinforcing mesh with reinforcing ratios ranging between 0.0017 to 0.0040.

3 TEST SPECIMENS AND LOADING PROGRAMS

3.1 Model materials

Microconcrete was used in constructing the tests specimens. The mix used consisted of Portland cement, graded sand aggregate and water, mixed in the proportions of 1:4:.58 by weight. The material has glass additions.

Model reinforcement was manufacturated from steel wire. This wire was normalized to lower its strength from the hard-drawn condition. The wire was then cold rolled between two grooved rolls to form protruding ribs on the bars similar to those on prototype reinforcement. The deformed wires were then normalized by heating in a 900 C oven for 30 minutes and then by air cooling to obtain the desired stress-strain properties of sharp yielding, level yield plateau, and strain hardening similar to prototype bars (Krawinkler 1985).

3.2 Small cantilever beams test

Several 1:10 scale models of cantilever beams were constructed. They were exact replicas of prototype beams tested before.

Hand operated screw jacks were used to apply horizontal loads at the end of each beam. Beam end loads were applied to reproduce similar beam end displacement histories used in previous full-scale tests, horizontal cyclic monotonic loads were applied at the top in every specimen. Typically, cycles of equal magnitude were repeated three times.

3.3 Cantilever Wall and Building tests

The central core and the whole earthquake resistant system specimens are shown in Figures 1 and 3. These specimens are 1:10 scale replicas of the Villa Real building having with properly scaled reinforcement and details.

In the whole earthquake resistance system

Tabla 1. Villa Real dimensions and specified material properties.

Total building height = 28.3 [m]
(from street level)
Foundation base level = -3.8 [m]
Building aspect ratio* = 2 to 1 E-W
                                = 1.6 to 1 N-S
Number of stories   = 10
Typical story height = 2.55 [m]
Floor slab thickness = 12 to 16 [cm]
Concrete strenght   = 300 [kgf/cm2]
(specified 28-day cubic resistant)
Rein.steel strenght = 4200 [kgf/cm2]
                           (in shear walls)
                       = 2800 [kgf/cm2]
                           (in slabs)
Wall area/floor area = 0.072 floor 1
                       = 0.059 floor 2
* Ratio of building height to plan dimension at base (Bolander 1989).

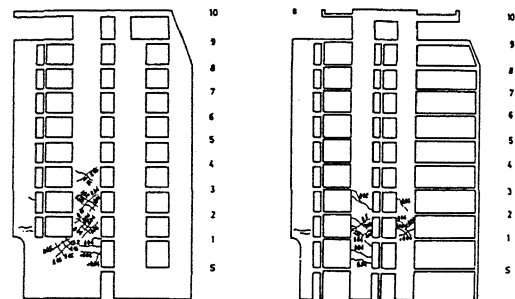
Figure 2. Villa Real cross-section sets. Cracks observed after of March 3,1985 earthquake are indicated
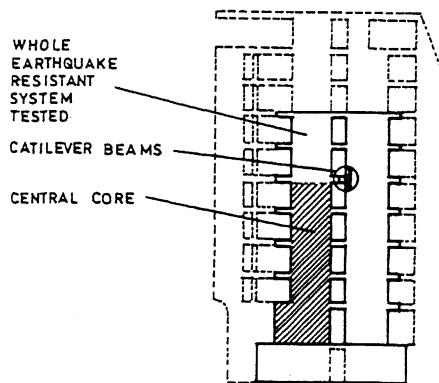

Figure 3. Central core and the whole earthquake resistant system specimens.

shown in Figure 3 one meter of the slab and transverse beams of the prototype were included on both sides of the walls.

The instrumentation system consisted of standard custom made measurement devices which were used to obtain continous records of loads and horizontal and relative vertical story displacements . Strain in reinforcing bars were not measured.

Horizontal cyclic monotonic loads were applied at the top of both specimens. Comparison between shear and bending moment distribution in height at the box seccion obtained during test, and results from analisys according to the Chilean Code NCh 433of72 are plotted in Figure 4.

Figure 4. Shear and bending moment distribution in height at the box section.

4 TEST RESULTS

4.1 Isolated Cantilever beam

The load-displacement results from tests of both the model and prototype are presented in Fig. 5. It is evident from this Figure that strenght, stiffness, and hysteresis loop properties are similar in the prototype and in the model.
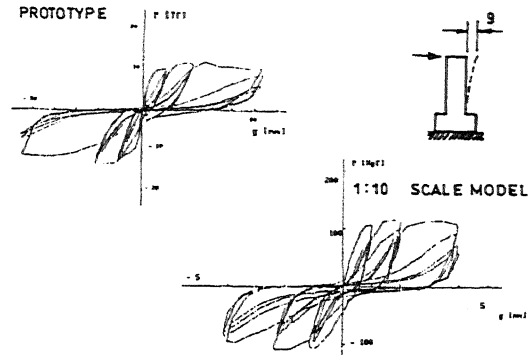
Figure 5. Load-displacement relationships in prototype and model for cantilever beams.

4.2 Central Core wall test

During the test the first crack was observed at the base of one of tranversal wings at 0.3 [mm] of lateral displacement at the top. Flexural and shear cracks initiated at 4 [mm] displacement. These cracks propagated along a diagonal path across the wall in later load cycles. These diagonal cracks spread throughout the lower wall.

Small cracks appeared at coupling beams when lateral displacement was 7 [mm]. Fracture of steel at one of the wings ocurred at 12 [mm] displacement.

The crack at the base of the wall opened much more than it did in the previous test, and horizontal sliding was not observed in this specimen as ocurred in other cantilever walls tested previously.

The fracture of the wing bars and the large opening of the horizontal crack at the base show clearly that the wall failed in a purely flexural mode.

Crushing of concrete and buckling of steel ocurred at final cycle of 20 [mm] roof displacement. Force vs roof lateral displacement relationships, and supplied bending moment and shear strenghts are shown in Figure 6.

4.3 Test of the whole earthquake resistant system of building

First cracks occurred in the flange of the box section, elevation 3, at the second and third floor when the top displacement was
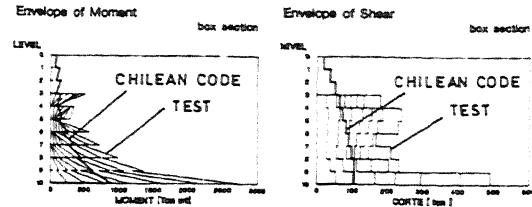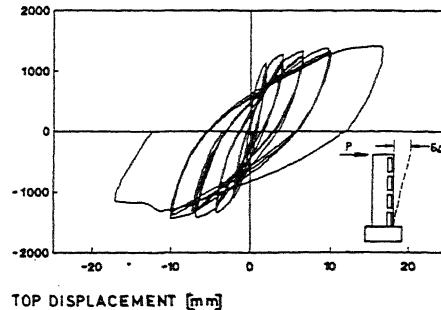

BASE SHEAR

TOP DISPLACEMENT [mm]

Figure 6. Force vs roof lateral displacement relationship, supplied bending moment and shear strenghts at the central core.

2.5 [mm]. At the same displacement,in next cycles, flexural cracks were observed across the ends of all coupling beams, except at the first floor. Slabs at fifth and sixth floors and a construction joint (axis D) in the first floor were cracked too. First shear cracks were observed in the box section in the third and fifth floor at a top displacement of 4.2 [mm]. First shear cracks in coupling beams appeared in levels third to fifth at a top displacement of 10 [mm] (.54% of height). Additional

cracks occurred in the box section similar to those observed in the central core wall test. Flexural cracks that propagated along a diagonal path were observed in the wall in axis E.

A great amount of diagonal cracks in the other wing were observed at the same time in that axis.

Flexural cracks in beams and slabs appeared in cycles corresponding to five milimeters of top displacement. Beams in the transversal direction did not show damage.

A pattern of general diagonal cracking was observed in the box section in the model, similar to the prototype building cracking after Viña del Mar earthquake. Cover loss and diagonal and longitudinal cracks in coupling beams were observed. The model presented steady hysteresis loops during these cycles.

Spread cracking with total cover loss was observed in beams between the third and sixth floors at top displacement of 25 [mm], the slabs were also cracked.

Failure of the box section, due to steel fracture in the flange (axis 3) ocurred at the third floor at a top lateral displacement of 30 [mm] (1.63 % of total height). Before fracture, a bar buckled in one corner of the box section. Final cracking in model is shown in Figure 7.

Horizontal cracks in columns in level three (intersection between axis 3 and F) suggest that the outriggering system of the transverse frame connected to the wall and the diaphragm (R/C slabs) provided significant energy dissipation after complete yielding of the wall at the base.

The crack patterns of the models were very similar to cracks observed in the actual building since concrete tensile strenght was kept low.

The model exhibited good response characteristics. Its stiffness, strength , and capacity to dissipate energy exceeded the minimum requirements of Chilean Code provisions by several times. It maintained its shear and overturning strenght at roof drift level exceeding 1.6 % . Even after the fracture of steel in the box section, the other components of the earthquake resistant system went on giving considerable strength to prevent the building colapse.

Tranverse walls inhibited the main wall to go out of plane when flexural cracks opened with fracture of steel as observed in other buildings like El Faro and Acapulco (Cruz 1990.

The response caracteristics of the structure were attained through a right selection of the layout of the structure in plan, which shows a quasi symmetric configuration. In addition, the most important walls were restrained by an outriggering system of frames in both main
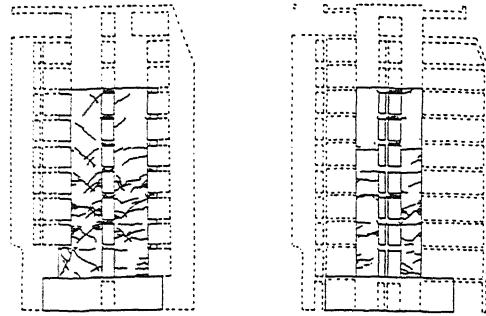

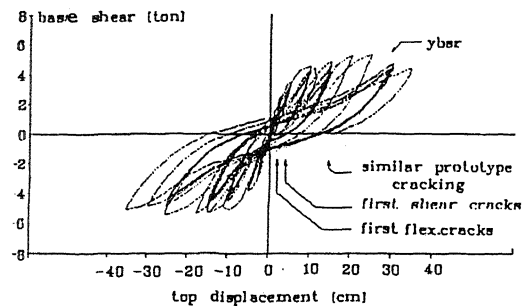Figure 7. Final cracking in model

Figure 8. Roof lateral displacement versus normal base shear
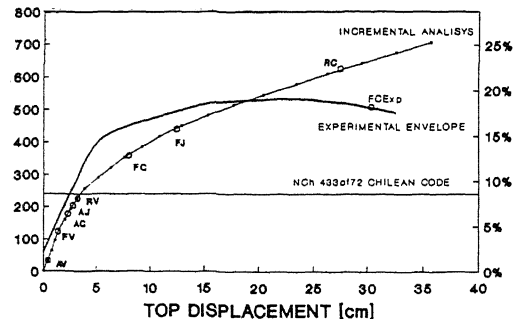
Base Shear [Ton]

Figure 9. Roof level lateral displacement of the whole earthquake resistant system specimen

directions, and they are continuous in elevation. Over level 2 the intensity of the shear and normal stresses at the critical regions, mainly at the beam end regions and at the wall in first story, were controlled by proper proportioning and reinforcement detailing.

5 INTERPRETATION OF TEST RESULTS AND CONCLUSIONS

The experimental tests discussed in this paper indicate that the overall load-deformation response of structural components can be reproduced adequately in tests of carefully designed and detailed small-scale models.

Isolated cantilever beam tests have shown that the use of properly deformed model reinforcing bars results in bond deterioration which is similar to that observed in the prototypes.

Cracks indicate that slabs contribute much more to the bending resistance of beams than is assessed in present standard design procedures.

The Villa Real building possesses overstrength against lateral loading relative to the Chilean code desing levels. Box section bending failure occurred in level three , with fracture of steel roughly at a base shear value of three times the NCh 433of72 Chilean Code specified seismic base shear. Damage occurred in the building model at the Code base shear level was limited to slight cracks in the coupling beams and minor flexural cracking in the wing of one of the walls forming the box section. (cycles corresponding to 2,5 [mm] of displacement).

The building appears to be quite stiff with respect to lateral load resistance. The maximum interstory drift index resulting at the base shear level given by Chilean Code was roughly an order of magnitude less than building code allowable drift limits and helps substantiate the short building fundamental period values, T = 0.5 [seg] , obtained in preliminary analysis.

The ultimate failure mechanism of the structure proved to be the progressive flexural cracking in walls. The steel in bars in the box section which had had yield internation during previous cycles finally fractured.

In Figure 8 , roof level lateral displacement versus normal base shear is plotted, indicating the load level at which cracks appears. YBSR indicates yielding of both the flexural and mesh reinforcement in the extreme tensile regions of the walls.

Initial cracking in all coupling beams started at a roof displacement of 2.5 mm [0.14% of height]. Figure 8 shows analytically predicted using Drain 2D [Kanaan & Powell (1973)] and test results values of floor level lateral displacements for the whole earthquake resistant system specimen.

In general , all the walls exhibited ductile flexural behavior. There was significant flexural yielding at levels 1,2,3,4 and a development of an intersecting pattern of flexural-shear cracks, but no sign of impending failure.

Structural walls similar to walls used in the Villa Real building can experience average story drift of at least 0.5 percent without major damage. Drift of 0.1 to 0.16 percent are possible with moderately confined boundary elements given by perpendicular walls (wings)(Wallace and Moehle 1989).

Fracture of steel ocurred at a top displacement of 30 mm, corresponding to a 0.16% of height, under reversal cycles.

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