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 steel wire. 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 occurred 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 30 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 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-

Figure 1. Plan configuration of the earthquake resistant system on the Villa Real building.
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. Additional 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 0.040.

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 1:4:56 by weight. The material has glass additions.

Model reinforcement was manufactured 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

<table>
<thead>
<tr>
<th>Table 1. Villa Real dimensions and specified material properties.</th>
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<tbody>
<tr>
<td>Total building height = 28.3 [m]</td>
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<tr>
<td>( from street level )</td>
</tr>
<tr>
<td>Foundation base level = -3.8 [m]</td>
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<tr>
<td>Building aspect ratio* = 2 to 1 E-W</td>
</tr>
<tr>
<td>N = 1.6 1 N = S</td>
</tr>
<tr>
<td>Number of stories = 10</td>
</tr>
<tr>
<td>Typical story height = 12 to 16 [cm]</td>
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<tr>
<td>Concrete strength = 300 [kgf/cm²]</td>
</tr>
<tr>
<td>( specified 28-day cubic resistant )</td>
</tr>
<tr>
<td>Rein. steel strength = 4200 [kgf/cm²]</td>
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<tr>
<td>( in shear walls )</td>
</tr>
<tr>
<td>2800 [kgf/cm²]</td>
</tr>
<tr>
<td>( in slabs )</td>
</tr>
<tr>
<td>Wall area/floor area = 0.072 floor 1</td>
</tr>
<tr>
<td>= 0.059 floor 2</td>
</tr>
<tr>
<td>* Ratio of building height to plan dimension at base (Bolander 1989)</td>
</tr>
</tbody>
</table>

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 continuous 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 section obtained during test, and results from analyses according to the Chilean Code NCh 4330f72 are plotted in Figure 4.

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 strength, stiffness, and hysteresis loop properties are similar in the prototype and in the model.

4.2 Central Core wall test

During the test the first crack was observed at the base of one of transversal 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 occurred 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 occurred 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 occurred at final cycle of 20 [mm] roof displacement. Force vs roof lateral displacement relationships, and supplied bending moment and shear strengths 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 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
diagonal paths 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
millimeters 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
displacement of 30 (mm) (1.63 % of total
height). Before fracture, a bar buckled in
one corner of the box section. Final
Figure 7. Final cracking in model
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 strength was
kept low.
The model exhibited good response
characteristics. Its stiffness, strength,
capacity to dissipate energy exceeded
the minimum requirements of Chilean Code
provisions by several times. It maintained
its shear and overturning strength 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 collapse.
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
The response characteristics 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
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 design levels. Box section bending failure occurred in level three, with fracture of steel in a base shear value of three times the NCh 43JoF72 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, $T = 0.3$ [sec], 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 interation 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, 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 occurred at a top displacement of 30 mm, corresponding to a 0.16% of height, under reversal cycles.

ACKNOWLEDGEMENTS

This research was supported by Committee of Science and Technological Research (Conicyt) under Fondecy Grant 585-88 and was conducted using the facilities of the Universidad Tecnica Federico Santa Maria, Valparaíso, Chile. The skilful assistance of students in the fabrication of the test specimens is gratefully acknowledged. Ernesto Villalobos worked in his thesis helping with calculations and discussions. The authors have benefited from conversations and works of Professor Moehle (University of California, Berkeley), Professor Sozen (University of Illinois, Urbana) and specially of Helmut Krawinkler (Stanford University, California).

REFERENCES


