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Behaviour of RC frame models strengthened with walls

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ABSTRACT: A review of work done at FIUAEM related with evaluation and strengthening of damaged structures is presented. Experimental results of three programs are introduced and discussed: a) Direct shear tests on models representing the interface beam-wall of three constructional schemes used to strengthen structures in Mexico, b) Frame-wall bidimensional model representing a subassemblage of a strong beam weak column frame structure strengthened with walls, and c) Tridimensional model of a floor slab or waffle slab structure. Practical implications of the findings are presented.

1. INTRODUCTION

After the 1985 Mexico earthquake a research team was formed at FIUAEM with continuous work up to date. A 145 page report on the state of the knowledge on evaluation and strengthening of R/C structures was done, as a result, two methods for evaluation of damaged structures were developed: one for preliminary evaluation applicable to 16 structural types common in mexican practice, and the other for masonry buildings up to 6 stories.

The inspection of 110 structures in the repair process, shows that 49 of the cases (45%) are based on the addition of walls constructed as infills, this proportion is also reported in Japan (Sugano 1987). Three basic construction procedures, schemes, for infilling existing frames were detected, see figure 1:

A. Reinforced concrete walls with continuous vertical reinforcement,

B. Reinforced concrete walls with its reinforcement hooked to the reinforcement of the existing frame, and

C. Masonry concrete infill walls reinforced with a wire fabric and a concrete cover.

One of the main problems detected was the lack of data related with the structural behaviour of the schemes, because of this, two experimental programs were conducted and are related in parts 2 and 3 of this paper.

On the other hand because buildings with floor slab system had structural problems during past earthquakes, it was decided to study the structural behaviour of this system using tridimensional model. This is related in part 4 of this paper.

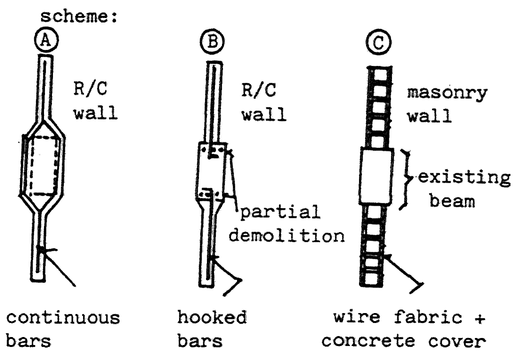

Figure 1. Constructional schemes, cross sections.

2. DIRECT SHEAR TESTS

This experimental program was carried out on 2/3 scaled models representing the interface beam-wall constructed according with the schemes previously described. Three models for each scheme (A, B, and C) were tested, geometry of the models was kept constant, varing the reinforcement quantity. The models were tested with increasingly shear force reversals, at each level of load three cycles were applied. Load and slippage of the interface were monitored, figure 2 shows the envelopes of load and slippage (push, first cycles) for the tested specimens.

Main observations from the data obtained

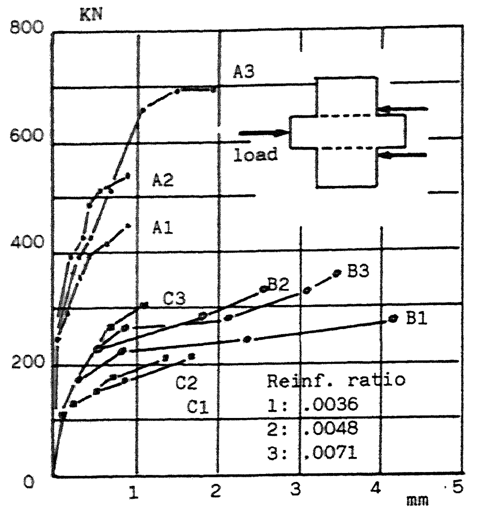

Figure 2. Envelopes load to slippage, direct shear tests.

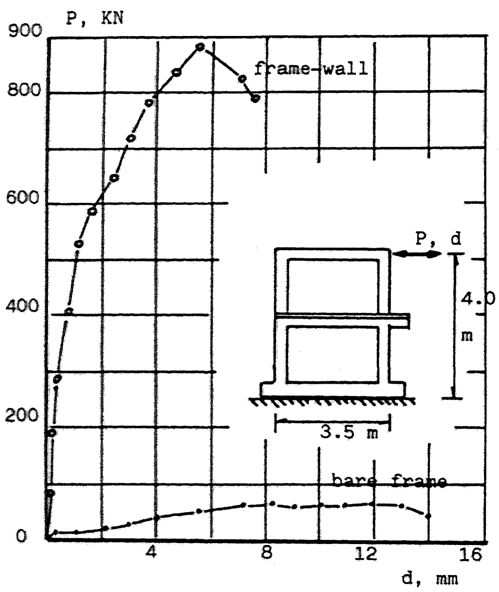

Figure 3. Envelopes load to displacement.

are as follows:

1. More quantity of reinforcement normal to the interface imply a increase in strength and a decrease in deformation capacity.
2. Strength was higher in scheme A but deformation capacity was low.
3. Scheme B showed more deformation capacity, and about half the strength of the corresponding scheme A.
4. With scheme C it can be reached a strength of the same order of that of scheme B, but deformation capacity reduces to about half.
5. Shear strength deterioration started at a slippage of the interface near to 1 mm, this imply that structures strengthened with walls nead to be designed with a drift limitation, it is proposed a limit of 0.2 %.
6. In intermediate and low rise buildings schemes B and C are recommended because lateral stiffness is increased and lateral forces due to earthquakes can be resisted within the elastic range.

3. FRAME-WALL TEST

With a grant of SEP-CONACYT-UAEM it was possible to develop a experimental facility capable for the testing of large structural models. A model representing a subassemblage of a building of the type strong beam weak column was constructed and tested. The model was sujected to increasingly lateral load reversals, at each level of load three cycles were applied. The purpose of this test was to generate damage in the model, a maximum drift of 4 % was reached. Maximum load was 61 KN at a drift of 2 %, this level of load was sustained until a total drift of 3.6 %. The ratio of maximum lateral displacement to that of first yielding of the reinforcement was 2.24.

After the test of the bare frame, the model was strengthened with reinforced concrete walls constructed as infills of the original frame according with scheme B. Columns were encased, a wall thickness of 0.10 m was used, reinforcement ratio in the wall was 0.004, no transversal reinforcement was used in the original beam to column joint which was damaged during the previous test.

The frame-wall model was subjected to increasingly lateral loads reversals until failure occurred, at each level of load three cycles were applied. Figure 3 shows lateral loads to displacements envelopes (push, first cycles) for the bare frame and the frame-wall models. The structural behaviour was dictated by the shear strength of the interface beam-wall of the first level. The failure of the interface occurred after the formation of diagonal craks in the wall, cracks formed in both directions with uniform distribution and at a close spacing, this is an indication of the important contribution of the walls to lateral strength prior to interface failure. The model exhibited low deformation capacity, maximum load was 889 KN at a total drift of 1.3 %.

Main obsevations based on the data obtained are as follows:

1. Important increments in lateral strength and lateral stiffness were observed. Ratio of maximum load of the frame-wall to maximum load of the bare frame was 13.6,

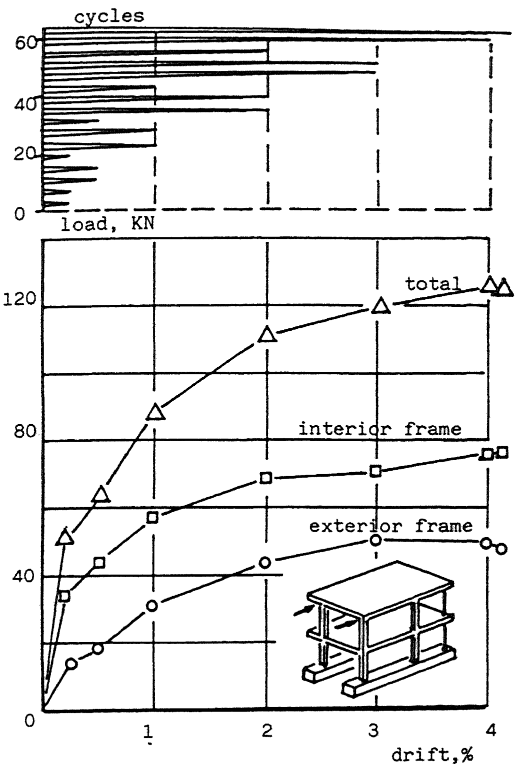

Figure 4. Envelopes load to drift, tridimensional model.

stiffness ratio (initial elastic stage) was 15.3.

2. The strength of the frame-wall was 85% of the computed strength assuming monolithic wall with ACI approach.

3. The strength of the frame-wall can be estimated with a shear strength given by $0.66\sqrt{f'_c\ (\mathrm{MPa})}$, considering total area of the wall without the columns.

4. The assumtion of monolithic wall when scheme B is used is not valid, however it is possible to increase the strength and the deformation capacity with the proper use of dowels and reinforcement details (Jirsa 1986)

4. FLOOR SLAB TRIDIMENSIONAL MODEL

The construction of buildings up to 15 stories with floor slab system (waffle slab, also called: equivalent frame system)become very popular in Mexico because of construction and econimic advantages. However, under lateral loads this system exhibit low stiffness and the structural behaviour is dictated by the shear strength of the slab to column connections which may be the cause of progressive collapse under seimic loads (Del Valle 1986).

After the September 1985 Mexico earthquake it was estimated that 3 % of the frame structures and 6 % of the floor slab structures in Mexico City collapsed or had severe damage, and that 15 % of the frame structures and 22 % of the floor slab structures suffered some degree of structural damage in such a way that strengthening was reccomended (Iglesias 1986).

It was decided to study the structural behaviour of this system and search for practical methods to increase lateral strength and stiffness. With a grant of CONACYT-UAEM it was possible to initiate in November 1989 an experimental program based on tridimensional models escaled to 1/2. To date one model had been constructed and tested in its original form, the upgrading and instrumentation of the damaged model is in progress. Test results on the original model are only shown.

The model consist of two stories, one and a half spans in the direction of loading and one and a quarter spans in the orthogonal direction. Free interstory dimension is 1.20 m, slab thickness is 0.16 m, the voids to lighten the slab are 0.30, 0.30 and 0.13 m, principal ribs are 0.20 m width, ribs next to principal 0.10 m, and the rest 0.05 m. Column section is square 0.20 m side. To simulate continuous internal spans in the direction of loading steel props with hinge ends were introduced in both stories. The model has typical dimensions and reinforcement, quantities and details, of buildings constructed before 1985, however the model does not represent a particular prototype because it was not possible to apply vertical loads.

Lateral loads were applied at the top of the model with two dobleaction actuators, the test was deformation controlled in such a way to produce uniform displacements in the direction of loading. Cycles of load reversals were applied, the history of load is represented at the top of figure 4, one half is shown.

It was convenient for data reduction to define an interior frame and an exterior frame corresponding to each of the column lines in the direction of the loading. See figure 4. The envelopes of load to drift (push, first cycles) for the exterior frame, interior frame and the complete model are shown in figure 4.

On the basis of the results the next observations are made:

1. The contribution to strength of the interior frame was greater than the contribution of the exterior frame, the difference decreases as more deformation was produced. At the beginning of the test the contribution of the exterior frame was 37 % of the corresponding to interior frame, at the end of the test this value was 65 %. This is attributed

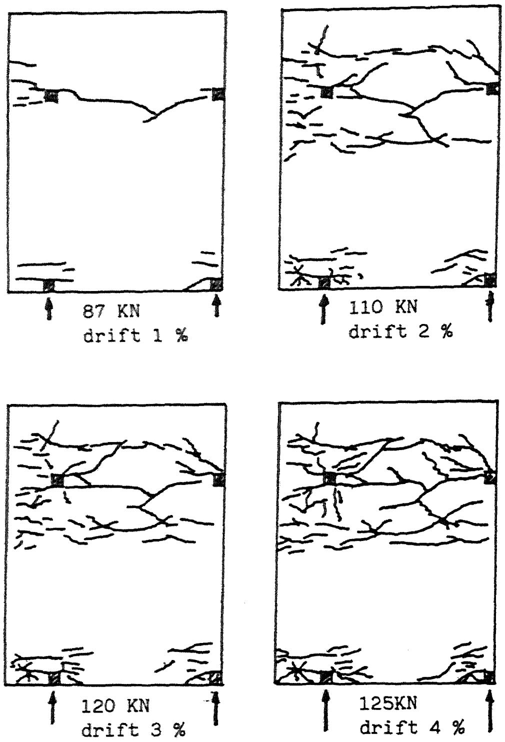

Figure 5. Crack patterns, first story slab (plan view)

to inelastic behaviour and stress redistribution.

2. Stiffness degradation was more noticeable in the interior frame. The model exhibit a drastic stiffness reduction, at the end of the test total stiffness was only 14 % of the original. In real structures this loss of stiffness may be the cause of an increased dynamic response and of a moment magnifying at the columns, which may produce the collapse.

3. Cracks in the slab formed near the column lines with paths perpendicular to load direction, figure 5 shows in plan view first story slab crack patterns as drift was increased. The slab to column connection of the exterior frame exhibited considerable damage but the rest of the connections had moderate damage. Flexural cracking was observed in the columns, mainly in the lower 2/3 of first story.

At present the model is under the strengthening prosses, columns will be encased and wing walls added; in both sides of interior columns and in one side of exterior columns. Section of the encased columns will be 0.30, 0.30 m and wing walls will be 0.35 m width and 0.10 m thick. Main longitudinal reinforcement will be continuous and special considerations will be given to connection details.

5. CONCLUSIONS

A frame strengthened with walls constructed as infills in existing structures, may or may not reach the strength of a corresponding monolithic wall depending on the constructional details used in the interface and in the existing connections.

An existing damaged frame strengthened with walls in which strength demand is high, proper design of dowels and strengthening of the existing connections is required. When only a stiffness increase is required schemes B and C, as described in this paper, may be suitable. This is the case of moderate and low rise buildings.

Under seimic movements floor slab buildings may be in danger of progressive collapse as a consequence of lateral stiffness degradation.

Investigation of proper retrofitting methods for structures with floor slab systems is of main concern.

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