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Evaluation of the failure of the Hotel Las Olas building as a consequence of the 22nd of April earthquake in Costa Rica

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ABSTRACT: A study of the partial colapse of the Hotel Las Olas during the 22 of April earthquake in Costa Rica both in the elastic and inelastic range is presented. From the analysis, the failure of two of the ground floor columns is concluded to be due to the presence of a masonry wall which reduced the clear span of these columns, making them behave as short columns that attracted a very large shear force and caused them to fail in a brittle manner, resulting in the partial colapse of the building.

1 GENERAL CONSIDERATIONS

The 22nd of April of 1991 there was an earthquake on the Atlantic coast of Costa Rica that produced considerable damage in Puerto Limon. Apart from others there where two hotels with R.C. structure which colapsed.

The Hotel Las Olas is situated on the outskirts of Puerto Limon, on the way to the Moin wharf (Figures 2,3). It was founded on a coral formation with the bases under sea water.

The building is constituted by a main block with 5 levels, with a very long rectangular plan form (94900x 6700 mm^2) and no construction joints. Additionally there is a smaller block with 2 levels of rectangular plan form (12200x9000 mm^2). At the moment when the earthquake ocurred the hotel was empty due to a total paint job being undertaken, therefore there were not many people in the hotel and no casualties were reported.

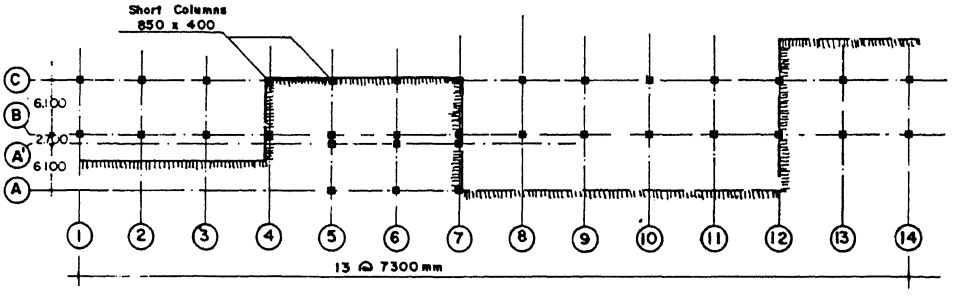


FIGURE 1.a PLAN VIEW

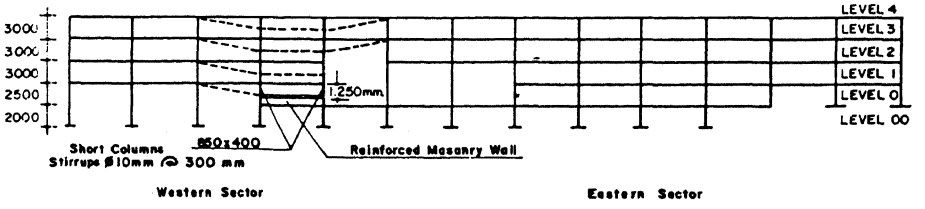


FIGURE 1.b.- FRAME C

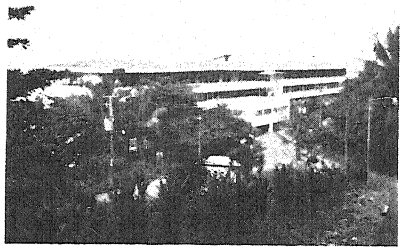


Figure 2. Frontal view of Las Olas Hotel

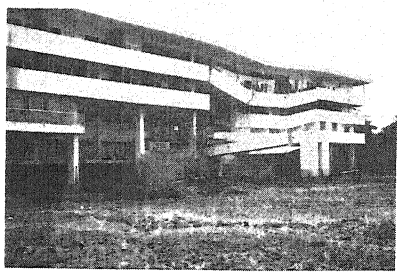


Figure 3. Rear view of Las Olas Hotel

The building colapsed partially during the earthquake in three bays due to the failure of two base columns (see Figure 1). Another column in the smaller block also failed.

2 STUDY OF THE MAIN BLOCK

The structure is formed by R.C. beams and columns, and prefabricated R.C. slabs with a cast-in-situ concrete topping. In the longitudinal direction (B and C axes) there are two very long frames of 94900 mm length that have 13 bays of 7300 mm each. Frame C is very irregular as can be seen in Figure 1. Practically there are two structures connected at level 3. Also at the end of the eastern sector there was a reinforced masonry wall of 2100 mm which carried a set of stairs.

In the transverse direction there are 14 frames (axes 1 to 14), of on bay of 6100 mm and two cantilevers of 2700 mm each. As may be seen the building is very flexible in the transverse direction.

The R.C. slabs of the hotel rooms were reinforced in the transverse direction (North-South) while the slabs of the balconies were reinforced in the londitudinal direction.

The dimensions of the columns vary from 850x400 mm at the two bottom levels to 350x350 mm at the top level and the beams vary from 350x600 mm to 250x500 mm. Initially all the columns between axes 7 to 12 were 4500 mm high. Later on a dining room was built between these axes which made necessary an additional structure next to the original one. So that the sea water would not enter certain parts of the hotel, as the lobby, a continuous reinforced masonry wall was built as indicated in the plan of Figure 1, which stiffens considerably the bottom level between axes 4 and 7. Apart from this wall the bottom level is almost free to permit the entry of the sea under the building, which results in great irregularities in the stiffnesses of the structure.

The foundations are square footings founded directly on coral rock. After the earthquake and due to the uplifting of the earth's crust, the shoreline retreated over 80 m., which allowed a direct inspection of the foundations which were previously under water. From the visual inspection done it was found that there were no foundation beams or pedestals. Also the concrete covers were observed to be small for the columns which were submerged in sea water which would worsen the corrosion process.

Between the axes 4-5 and B-C, at the top of the retaining wall a reinforced masonry wall was built which braced columns B4, C4 and C5 making them to behave as short columns of 1250 mm in height. As a consequence a great amount of the seismic shear was taken by them. Column B4 in the longitudinal direction had a light and flexible partition wall on one face and a masonry railing on the other. In the transverse direction there was a masonry wall which confined column B4 but not enough to cause a short column behavior. However, columns C4 and C5 were confined by a masonry wall on two faces. This, together with the great flexibility of the slender columns of the western sector produced a great vertical irregularity which eventually caused the colapse of three bays of the building, with a total destruction of the short columns which were totally crushed during the earthquake. In Figure 4 column C4 can be seen to have dissappeared completely and the floor slab lying directly on the masonry wall.

From the visual inspection it was observed that the detailing and concrete covers were insufficient, the stirrups at the ends of the columns are separated 300 mm and both these and the longitudinal steel were very corroded. It is assumed that the co-

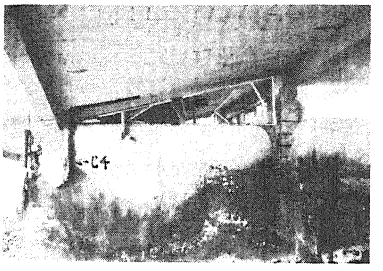


Figure 4. Detail of colapsed column C4

rrosion ocurred before the earthquake.

3 EVALUATION OF THE STRUCTURAL FAILURE

3.1 Static Elastic Analysis

A linear behavior of the structure was assumed with a maximum ground acceleration of 0.47g, value which was estimated for Puerto Limon by Climent (1991). For the determination of the lateral seismic forces a ductility factor of 2 was used. This value was obtained using a computer program developed at the University of the Andes by Ramirez (1989) and DeBarcia (1991), based on the incremental colapse of the structure (Figures 5,6).

The answers obtained from these analyses are presented in Tables 1.a, 1.b and 1.c for levels 00, 0 and 1 (see Figure 1). The ratios between capacities and demands for the columns in bending and shear are shown there. The capacities in shear were calculated using the ACI committee 318 (1983) formulas corrected by Mattock and Wang (1984). Reports by Umehara and Jirsa (1984) and Watanabe (1984) were also consulted but not applicable to the case being studied.

Table 1a. Capacity/demand ratios for columns of level 00

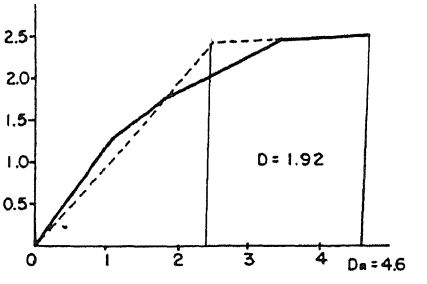
Column	Mu(KNm)	Mn/Mu	Vu(KN)	Vn/Vu
C1	78.6	2.93	22.6	12.82
C2	82.0	3.90	19.9	16.61
C3	60.3	5.31	8.0	32.15
C8	172.8	1.79	68.0	4.67
C9	151.8	2.11	34.8	9.36
C10	136.0	2.35	23.9	13.02
C11	168.9	1.90	65.9	4.91

Table 1b. Capacity/demand ratios for columns of level 0

Column	Mu(KNm)	Mn/Mu	Vu(KN)	Vn/Vu
C4	683.7	0.47	725.6	0.45
C5	479.2	0.54	448.4	0.70
C6	313.2	0.83	103.6	2.34
C7	334.7	0.84	114.4	2.12
C8	172.8	1.77	105.4	2.97
C9	151.8	2.11	112.0	2.88
C10	136.0	2.35	95.4	3.37
C11	168.9	1.89	114.8	2.79
C12	371.4	0.86	221.2	1.34
C13	358.2	0.89	214.9	1.44
C14	268.3	0.82	167.7	1.64

Table 1c. Capacity/demand ratios for columns of level 1

Column	Mu(KNm)	Mn/Mu	Vu(KN)	Vn/Vu
C1	284.4	0.60	180.8	0.87
C2	331.3	0.57	217.0	0.74
C3	351.5	0.54	226.7	0.70
C4	340.2	0.56	212.5	0.75
C8	229.5	0.83	146.3	1.11
C9	251.5	0.76	164.8	0.98
C10	252.5	0.75	166.4	0.97
C11	259.4	0.73	170.5	0.95
C12	249.1	0.76	163.8	0.99
C13	270.3	0.70	180.2	0.90
C14	215.7	0.79	137.4	1.16



TOP STORY HORIZONTAL DISPLACEMENT

FIGURE 5a - DUCTILITY FACTOR FOR FRAME 4

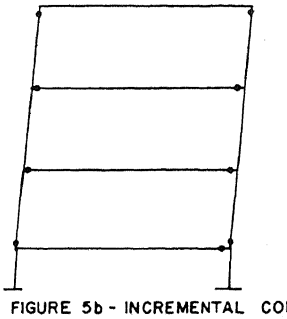
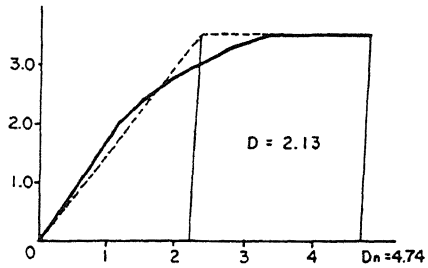


FIGURE 5b - INCREMENTAL COLAPSE FOR FRAME 4



TOP STORY HORIZONTAL DISPLACEMENT
FIGURE 6a-DUCTILITY FACTOR FOR FRAME 5

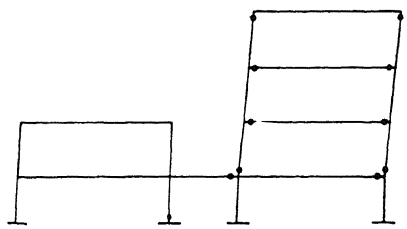


FIGURE 6b - INCREMENTAL COLAPSE FOR FRAME 5

In Tables 1a to 1c, Mu and Mn are respectively the acting and capacity moments; Vu and Vn are respectively the acting and capacity shear forces.

All the results for level 00 are greater than 1 which shows that the capacity is greater than the demand both in bending and shear and that this level had no problems.

At level 0 columns C4 to C7 and C12 to C14 show ratios in bending that are less than 1. In shear columns C4 and C5 have ratios which are very low, especially column C4 with a ratio of 0.45 which indicates that the demand in shear was over twice the capacity, and suggests that the failure which ocurred in these short columns during the earthquake was due to shear.

3.2 Dynamic elasto-plastic analysis

For these analyses the computer program DRAIN-2D was used with bilinear hysteretic loops and 2% strain hardening with no degradation in strength or stiffness.

Before these analyses a detailed inspection of the building was done, having at our disposal the structural drawings. From this inspection several differences were found between the existing structure and the drawings. Originally the building was designed to have 3 levels but during construction another level was added on. At a later date the bridge between axes 7 and 12 at level 0 was widened to give space for a dining room. This served as a middle support for columns C7 to C12 which caused a change in the structural behavior of these columns. The columns of level 0 in the drawings were of 700x400 mm with eight 22 mm steel reinforcing bars (1.1%). In the construction they were changed to 850x400 mm with eight 19 mm bars and two 10 mm bars (0.7%). With this change the reinforcement ratio was reduced to a value below the minimum and 10 mm bars were used in one of the faces that should not be used for longitudinal reinforcement. The slabs in the balconies were reinforced in the longitudinal direction and the slabs in the hotel rooms were reinforced in the transverse direction which causes an irregular distribution of vertical loads and a deficient distribution of reinforcement in beams and slabs.

The frames of the structure were analyzed under the action of acceleration records of El Centro 1940 and Siquirres 1991. This last record was digitized by the authors from the original accelerogram reported by Climent (1991). The accelerations of this record were scaled down to produce a peak acceleration of 0.47g as estimated for Puerto Limon by the same author.

From the visual inspection the main damages were observed to have ocurred in the longitudinal direction (Frames B and C) mainly at the axes 4 and 5 of Frame C (columns C4, C5, Figure 1) which were laterally supported by a reinforced masonry wall leaving a clear span of 1250 mm. On the other hand the column at axis 4 of Frame B (column B4, Figure 1) was able to free itself from the masonry which gave it lateral support and was able to behave as a columns of 2500 mm in height, therefore showing important damage but without colapsing. The column of axis 5 of this frame (column B5) was not laterally supported by masonry and therefore did not suffer important damage. It was estimated that columns C4 and C5 did not suffer important rotations at the height of the reinforced masonry wall, thus, for the seismic analyses they were considered to be fixed at the top of the wall.

A complete study of the formation of plastic hinges in bending at both ends of beams and columns was done for both earthquakes mentioned (see Figures 7,8). From the same it can be seen that there was no possibility of the formation of a colapse mechanism in bending. The presence of a level more flexible than the rest almost caused the formation of a lateral colapse mechanism as shown in Figures 7a and 7b. This was confirmed by the

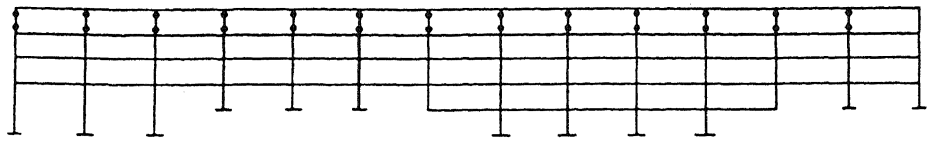


FIGURE 7a.- PLASTIC HINGES IN FRAME B FOR EL CENTRO EARTHQUAKE T = 1.20 secs

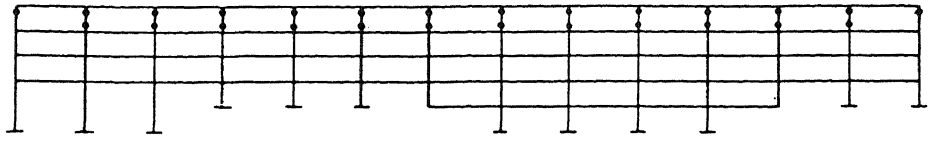


FIGURE 7 b.- PLASTIC HINGES IN FRAME B FOR SIQUIRRES EARTHQUAKE T = 0.75 secs

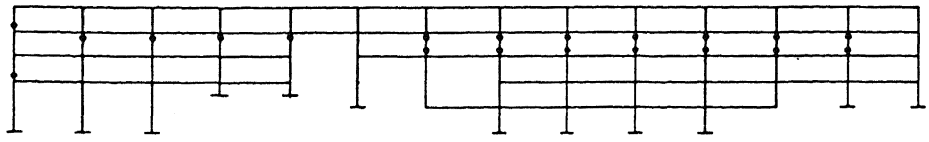


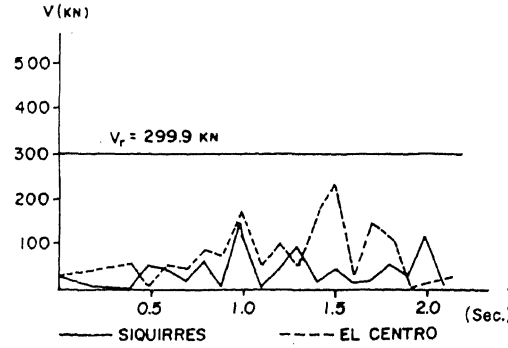
FIGURE 8.- PLASTIC HINGES IN FRAME C FOR EL CENTRO EARTHQUAKE T = 1.40 secs

fact that most of the top story masonry partitions failed laterally in the longitudinal direction.

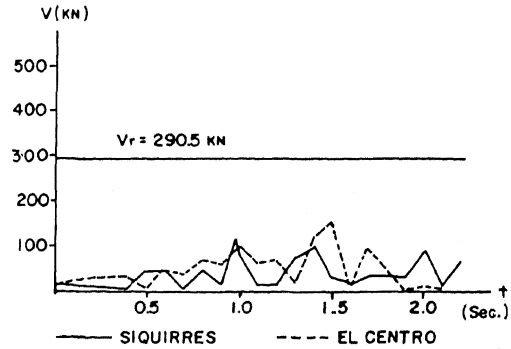
In Figures 9 and 10 the demands and capacities in shear are graphed versus time. In Figure 9, which corresponds to Frame B, the demand never exceeds the capacity for the earthquakes analyzed. In Figure 10a it can be seen that in column C4 with the Siquirres earthquake the shear capacity was exceeded by up to 35% from 1.12 to 1.15 secs and with the El Centro earthquake up to 80% from 0.98 to 1.21 secs, from 1.38 to 1.54 secs and from 2.11 to 2.25 secs. In Figure 10b it can be seen that in column C5 with the Siquirres earthquake the shear capacity was exceeded mainly from 1.11 to 1.13 secs and with the El Centro earthquake from 1.40 to 1.50 secs. The axial force remained near constant during the periods of time considered.

4 CONCLUSIONS

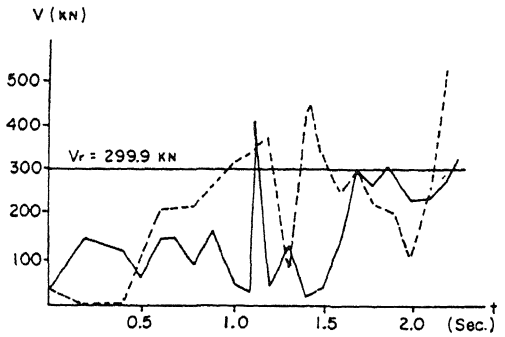
From the results obtained and presented herein it was found that both in elastic and inelastic behavior there was no possibility for the formation of a colapse mechanism in bending. On the other hand, in columns C4 and C5 the shear demands exceeded greatly the capacities, thereby concluding that the failure observed was due to shear in the mentioned columns producing a brittle type failure.



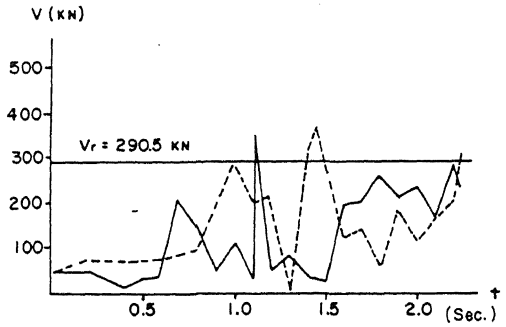
FRAME B - COLUMN B4
FIGURE 9a - SHEAR FORCE VS. TIME



FRAME B - COLUMN B5
FIGURE 9 b - SHEAR FORCE VS. TIME



FRAME C - COLUMN C4
FIGURE 10a SHEAR FORCE VS. TIME



FRAME C - COLUMN C5
FIGURE 10b - SHEAR FORCE VS. TIME

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