Seismic Retrofitting of Reinforced Concrete Building Damaged by Tucacas Earthquake, Venezuela

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SUMMARY
On September 12th, 2009, a magnitude 6.4 earthquake took place with epicenter located about 50 km northwest of the town of Tucacas, Venezuela. The earthquake did not produce victims or injuries, although it did cause material damages to few houses and 23 buildings, including the Miramar building.
The damage caused by the earthquake in this building was basically located in the ground floor columns, with minor damages in the upper floors; the carried out structural reinforcement included: (i) placing of inverted V bracings in selected frames; and (ii) strengthening of columns of several frames. It was concluded that: (i) the poor performance of the building was due to concurrent deficiencies in concept, design and construction; and (ii) the revealed strength and stiffness deficiencies can be corrected with the prescribed reinforcement.

Key words: earthquake, damage, retrofitting, steel, reinforced.

1. INTRODUCTION

This paper presents the seismic adequacy of Miramar building damaged after the earthquake of magnitude 6.4 that occurred on September 12, 2009 in the Caribbean Sea north of the Venezuelan coast (Bolivar, et al, 2010). The quake did not cause deaths or injuries, but it damaged 23 multifamily buildings. Among the damaged buildings is Miramar, located in the town of Tucacas, Silva Municipality, Falcon State, Venezuela. The Miramar building, is a reinforced concrete residential-vacational complex made up of three units place in a C floor layout, two of which have six levels (East and North) and the other five levels (West) (Figure 1). The three units are structurally separated by construction joints and were designed with a Seismic Buildings Provisional Standard issued by the Ministry of Public Works in November 1967 (MOP, 1967) after the Caracas earthquake the same year. The three units of the building are rectangular in plan and the frame structure is reinforced concrete with beams in both directions, except the presence of certain flat beams in some frames for architectural reasons (Bolivar, et al, 2010). Damage caused by the earthquake in the three units of the building was mainly columns restricted to the ground floor with minor damage on the upper floors. Seismic analysis performed according to the building in its original state, found that many of his columns and beams failed due to low strength and stiffness. According to the Covenin 1756-2001 "Venezuelan Earthquake Resistant Building Code" (Covenin, 2001), this building classified as Group B2 and therefore its importance factor α is equal to 1. Additionally, because it is located in seismic zone 4 the horizontal ground acceleration is Ao = 0.25 g.

2. EARTHQUAKE CHARACTERISTICS

On September 12th, 2009, a magnitude 6.4 earthquake took place on the Caribbean Sea north of the Venezuelan coast with its epicenter located at coordinates 10.81 ° N and 67.91 ° W and a focal depth of 5.8 km. (Funvisis, 2009). The quake caused no deaths or injuries, although damage in 23 buildings in the towns of Boca de Aroa, Tucacas and Chichiriviche in Falcon State, which are located about 50 km west of the epicenter. In the populations of Tucacas and Chichiriviche the phenomenon of liquefaction in the vicinity of the beach was reported. The quake also was felt with some intensity in various populations of the states of Carabobo, Aragua and Lara and the Capital District, but only minor damage was reported. Importantly, in the tourist towns of Boca de Aroa, Tucacas and Chichiriviche, there are about 300 buildings, most vacational usage, of which only three suffered severe structural damage, and cracks and fissures were reported in some beams and columns in twenty buildings as well as damage to the partitions.

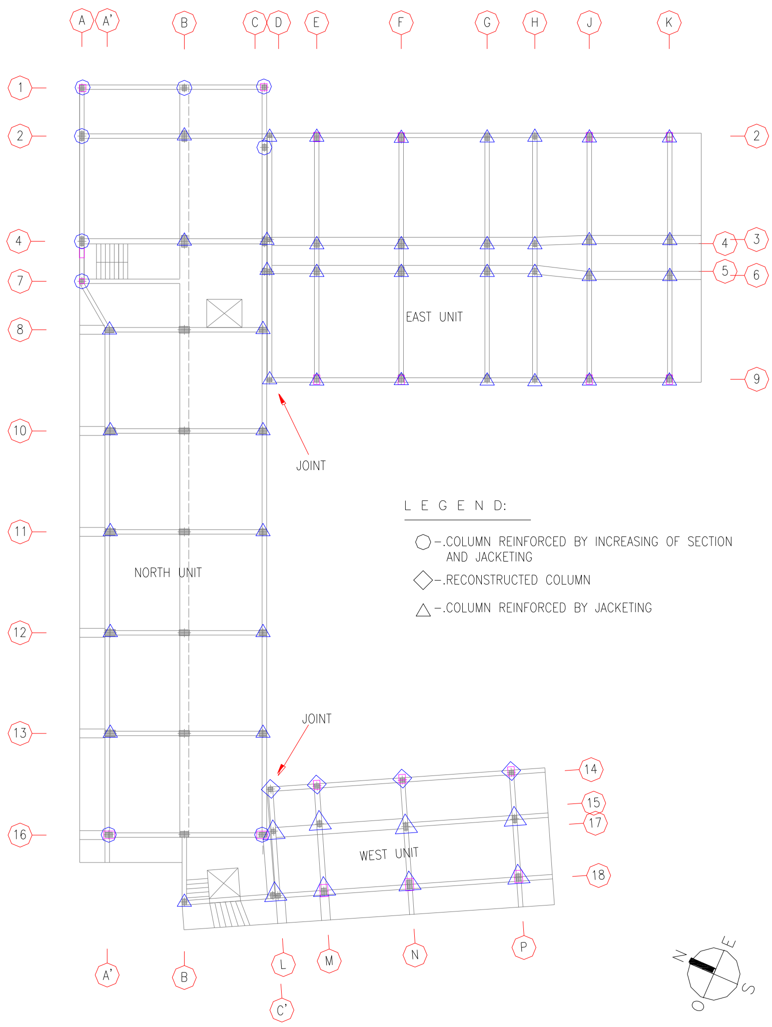

Figure 1. Plan configuration of Miramar Building. Column reinforced from Ground Floor to Level 1

3. DAMAGE REPORTED IN THE BUILDING MIRAMAR .

The three units of Miramar building suffered damage in columns and partitions, which are concentrated mainly between the ground floor and first level. In the West Unit severe damage in the four columns of the frame 14 was observed (shoring was required) due to short column effect caused by interaction with the masonry wall that partially confines the frame (Figures 2 and 3); in the East Unit the damage was severe and widespread in columns of ground floor to level 1 from 2 to 6 axes due to the presence of soft-story (Figures 4 and 5), and in the Unit North damage was moderate and localized in the structure of the staircase between axes 4-7 and AB on the ground floor (Figure 6). The beams showed no significant damage. Damage to partitions was similar in the three units of the building and concentrated mainly on the ground floor and level 1 (Figure 7).

Figure 2. View of damage in short column. West Unit (A. Urich)

Figure 3. Detail of damage in short column of Figure 2 (A. Urich)

Figure 4. Cracking and crushing of concrete on top of the column (plastic hinge) in East Unit (A. Urich)

Figure 5. Detail of cracks at the top of the column of Figure 4 (A. Urich)

Figure 6. Damage in East staircase in the ground floor in the North Unit

Figure 7. Damages in partitions in ground floor in the North Unit

4. STRUCTURAL OBSERVED FEATURES .

4.1. Concrete and reinforcing steel

According to the plans indicated in structural design, the quality of concrete is 210 kgf/cm^2. However, the results of the 12 cores of concrete tested at the Materials Laboratory of the Catholic University Andres Bello in Caracas and statistical analysis Sclerometry Index (SI) of 38 structural elements made by BRS Engineering, C.A. (2009) notes that detect two different specific populations concerning their characteristic resistance within the universe of the concrete used in the structural skeleton of the three units of Miramar building, namely : (i) concrete structure from ground floor to level 1: f´c = 320 kgf/cm2 and (ii) Concrete from levels 2 to ceiling: f´c = 244 kgf/cm^2. Therefore, for the analysis of the original structure, we used a concrete with a compressive strength f´c= 250 kgf/cm^2. According to the document data, we assumed a reinforcing steel with yield point fy = 4200 kg/cm^2.

4.2. Exploration of reinforced beams and columns

From the results of the 12 scans of existing beams and columns made by the company BRS Engineering, C.A. (2009), the following conclusions are noted: (i) reinforcement steel bars correspond to grooved carbon steel, grade A-42 (fy = 4.200 kgf/cm^2), (ii) no stirrups were placed at the nodes (beam-column joints), (iii) with the random search performed, it was found that some main columns were less in size and reinforced bars areas than the values described in the project, and (iv) the longitudinal beams on the edge nodes are not developing properly by anchoring with hooks or bends, this leads to brittle failure due to lack of anchorage.

4.4. Soil profile and foundation

According to a soil survey conducted in 2010 by the company Sueling Engineering, C.A. (2010) at the place where is located the Miramar the overall soil profile is S3 according to the standard Covenin 1756-2001. Likewise, exploration of the foundations concluded that the building is supported on deep foundations: precast piles driven into place with a drum machine. Although no plans were available, information collected on the site indicated that the piles are grouped in two, three and four.

5. SEISMIC EVALUATION

The Covenin 1756-2001 leads to the following considerations: (i) the building is located in seismic zone 4 and therefore the value of the peak horizontal ground acceleration is Ao = 0.25 g, (ii) the importance factor α = 1.00, and the soil profile S3 leads to φ = 0.70. Therefore, the peak acceleration considering the soil effect is 0.175 g. Based on the detailing of the reinforcing steel, the reduction factor is taken as R = 3. Figure 8 shows the response spectra used in the analysis of the structure in its original state. For the seismic evaluation the guidelines established in Chapter 12 "Existing Buildings" of the Covenin 1756 was used (Covenin, 2001). The minimum seismic coefficient C in the code gives 8.30%. The dynamic analysis of the building was done with the program ETABS - v9.7.0 (Computers and Structures, Inc, 2010) using the method of three degrees of freedom per level.

5.1. East unit

This unit with a rectangular plant has 4 frames in the longitudinal direction and 7 frames in the transverse direction (see Figure 1). All beams, except for the two central longitudinal frames, are deep. The slabs are ribbed, 30 cm thick. This unit has 6 levels with a height of 18.15 m, an area of 2940 m^2 and a weight of 2412 tons. The dynamic analysis showed that the fundamental period in the longitudinal direction (X) is 1.26 seconds and in the transverse direction (Y) is 0.76 seconds, values which are larger than that estimated with the empirical formula (0.62 sec.) (Covenin, 2001).

Base shear in X and Y are 264 and 307 tons respectively. These shear leads to the following values of seismic coefficient Cx=10.94% and Cy=12.72%, which are above the regulatory minimum (8.33%) (Covenin, 2001).
As for the drift, we note that the maximum in X occurs at level 2 (12.21 ‰) and the highest in Y direction occurs at level 1 (10.93 ‰). These drifts are inferior to those in the Covenin 1756-2001 (18‰). Checking the strength capacity of the structure considering the gravity and seismic loads, we found that the demand exceeds the capacity of the columns in almost all levels and in some beams. Therefore, this unit should be strengthened.

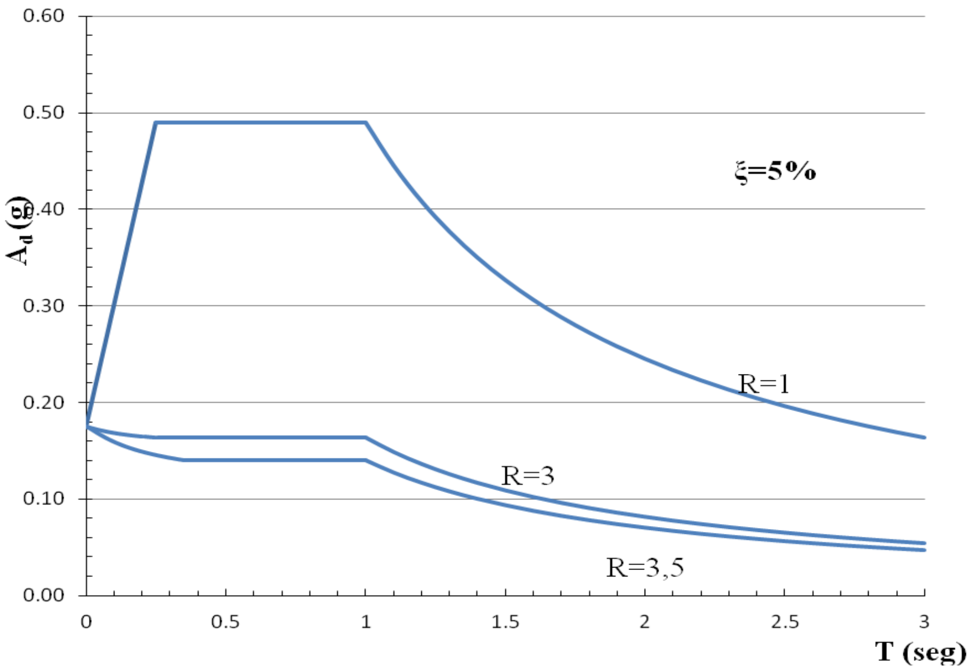

Figure 8. Elastic and design response spectra for values of the reduction factor R and soil profile S3

5.2. North unit

This unit with a rectangular plant has 3 frames in the longitudinal direction and 10 frames in the transverse direction (see Figure 1). All beams, except in the central longitudinal frames, are deep. The slab is ribbed, 30 cm thick. This unit has 6 levels with a height of 18.35 m, an area of 4140 m^2 and a weight of 2862 tons. The dynamic analysis showed that the fundamental period in the longitudinal direction (X) is 0.85 seconds and in the transverse direction (Y) is 0.69 seconds, values which are larger that estimated with the empirical formula (0.62 sec.) (Covenin, 2001).
Base shear in X and Y are 365 and 360 tons respectively. These shear leads to the following values of seismic coefficient Cx =12.75% and Cy =12.58%, which are above the regulatory minimum (8.33%) Covenin, 2001).
As for the drift, we note that the maximum in X occurs at level 2 (7.01‰) and highest in Y direction occurs at level 1 (7.75‰). These drifts are inferior to those in the Covenin 1756-2001 (18 ‰). Checking the strength capacity of the structure, considering the gravity and seismic loads, we found that the demand exceeds the capacity of the columns in almost all levels and in some beams. Therefore, this unit should be strengthened.

5.3. West unit

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This unit building rectangular plan has 3 frames in the longitudinal direction and 4 frames in the transverse direction (see Figure 1). All beams, except in the central longitudinal frames LN, are deep.

The slab is ribbed, 30 cm thick. This unit has 5 levels with a height of 15.45 m, an area of 875 m^2 and a weight of 714 tons. The dynamic analysis showed that the fundamental period in the longitudinal direction (X) is 1.01 sec. and in transverse direction (Y) is 0.58 sec., values which are larger that estimated with the empirical formula (0.55 sec.) (Covenin, 2001).
Base shear in X and Y are 105 and 89 tons respectively. These shear leads to the following values of seismic coefficient Cx = 14.70% and Cy =12.46%, which are above the regulatory minimum (8.33%) (Covenin, 2001).
As for the drift, we note that the maximum in X occurs at level 1 (12.10 ‰) and highest in Y direction occurs at level 1 (8.67 ‰). These drifts are inferior to those in the Covenin 1756-2001 (18 ‰). Checking the strength capacity of the structure, considering the gravity and seismic loads, we found that the demand exceeds the capacity of the columns in almost all levels and in some beams. Therefore, this unit should be strengthened.

6. STRUCTURAL RETROFITTING

6.1. Retrofitting strategy

Since the damage reported in the three units of the building were due mainly to lack of strength, stiffness and ductility, some of the structural adjustment was to increase the stiffness and strength of structural member at each of the three units that form the Miramar building to meet the guidelines, established in Chapter 12 of the Covenin 1756 code (Covenin, 2001).
The dynamics analysis of the building was done with the program ETABS (Computers and Structures, Inc., 2010) using the method of three degrees of freedom per plant. For the analysis a response reduction factor value of R=3.5 (see Figure 8) was assumed. The reinforcement of the superstructure consisting essentially of the following: (i) placing metal reinforcements in the form of an inverted V in several frames in longitudinal and transverse direction in units East and West, and (ii) reinforce columns by increasing cross section and jacketing

6.2. Foundation system

A structural adjustment proposed in this work does not require intervention of foundations of the three units of the building, since the increase of lateral forces can be absorbed by existing foundations. Additionally, the increased weight of the reinforcement effect in each of the three units of the building Miramar is insignificant compared to the original weight

7. BUILDING STRUCTURAL STRENGTHENING

7.1. East unit

7.1.1. Description of reinforcement
The reinforcement of this unit of the building consists of the following: (i) metallic bracing inverted V shape in the longitudinal frames 2 and 9 between the axes EF and JK at the ground floor. The inverted V brace is interrupted by a horizontal beam from level 1 to level 4, as shown in Figures 9 and 10. This particular form of reinforcement is due to the architectural features of east and west facades, (ii) placement of the inverted V-shaped brace in transversal frames E to K from ground floor to level 1, and (iii) jacketing of all columns from ground floor to level 1 (see Figure 1), and (iv) jacketing of columns E-2, F-2, J-2, K-2, E-9, F-9, J-9 and K-9 from level 1 to level 4. Figure 11 shows a detail of jacketing of columns.
7.1.2. Results of dynamic analysis
The fundamental periods were as follows: Tx = 0.88 sec and Ty = 0.52 sec. This indicates that they were down 30.16% in X and 31.58% in Y directions, which means that the building substantially increased its stiffness.

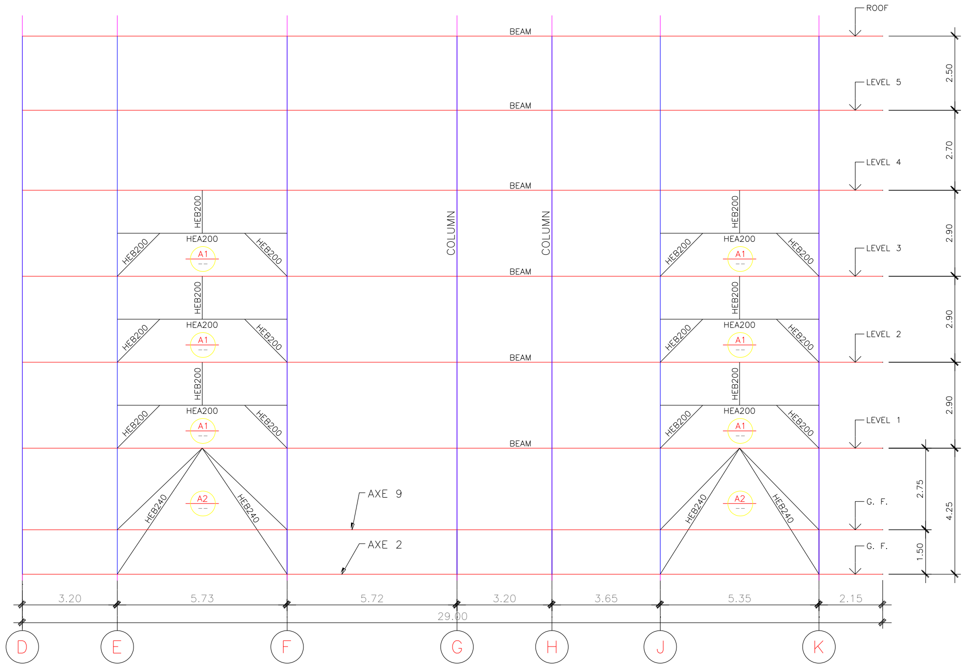


Figure 9. Elevation of reinforced frames 2 and 9 in the East Unit

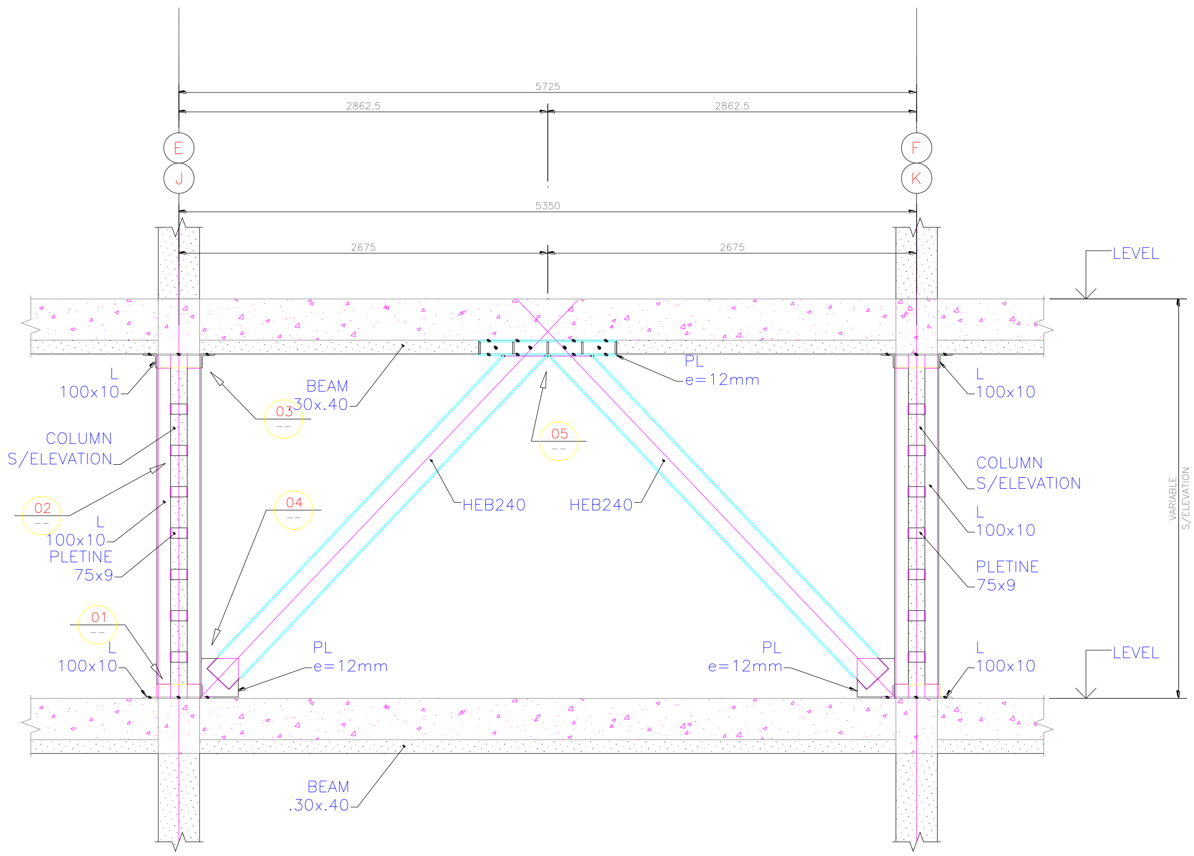
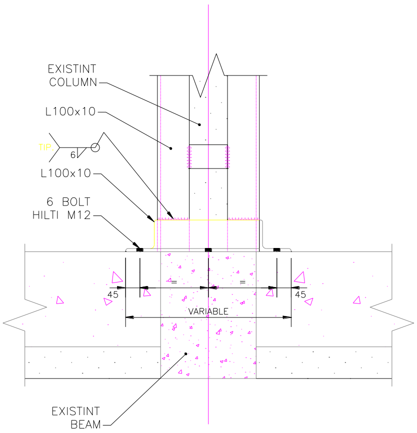
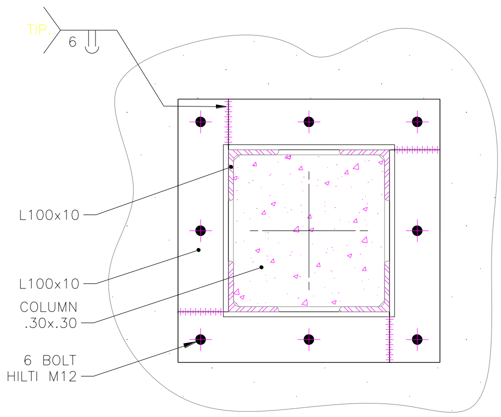


Figure 10. A2 detail of Figure 9

The shear forces are: Vx = 252 tons in the direction X and Vy = 209 tons in the direction Y. The corresponding seismic coefficients are: Cx = 10.41% and Cy = 8.63%, which are above the minimum (7.14%) prescribed by the Covenin 1756-2001. The largest drift in the X direction is 8.87 ‰ and occurred at level 5. For the Y direction a value of 3.85 ‰ was obtained at level 3. These values are below the prescribed allowable drift (18 ‰) in the Covenin 1756-2001. By comparing these drifts with those obtained in the unreinforced building, we noticed that they were reduced by 28.17% in X and 64.78% in Y directions, indicating a significant increase in the stiffness.



View of the reinforcement in the base



Plan view of the reinforcement in the base

Figure 11. Jacketing of column

7.2. North unit

7.2.1. Description of reinforcement

The reinforcement of the leather of the building consists of the following: (i) increasing the cross section (Figure 12) and jacketing of columns A-1, B-1, C-1, A-2, C-2, A -4, A-7, A-16 and C-16 from ground floor to level 4 (see Figure 11), (ii) jacketing of columns A'-4, A'-10, A'-11, A-12, A-13, C-4, C-8, C-10, C-11,C-12, C-13, C-18 and B-18 from ground floor to level 1 (see Figure 1); (iii) reconstruction of the stair East from ground floor to level 1.

7.2.2. Results of dynamic analysis

The fundamental periods were as follows: Tx = 0.77 sec. and Ty = 0.64 sec. This indicates that they were down 9.41% in X and 7.25% in Y directions, which means that the building substantially increased its stiffness.

The shear forces are: Vx = 363 tons in the direction X and Vy = 402 tons in the direction Y. The corresponding seismic coefficients are: Cx = 12.51% and Cy = 13.86%. These seismic coefficients are above the minimum (7.14%) prescribed by the Covenin 1756-2001. The largest drift in the X direction is 6.99 ‰ and occurred at level 2. For the Y direction a value of 5.89 ‰ was obtained at level 1. By comparing these drifts with those obtained in the unreinforced building, we noticed that they were reduced by 0.31% in X and 21.99% in Y directions, indicating a significant increase in the stiffness.

7.3. West unit

7.3.1. Description of reinforcement

The reinforcement of this unit of the building was as follows. (i) metal bracing inverted V-shaped in frames M, N and P between 18-17 and 17-14 axes (Figure 13) and the frame L between 17-18 axis from ground floor to level 1; (ii) jacketing of columns of frames 17 and 18 from ground floor to level

1(see Figure 1) and (iii) reconstruction of the four columns of the frame 14 by encasing in high strength concrete with appropriate longitudinal and transverse reinforcement and separating them from the masonry wall in order to avoid the effects of short columns.

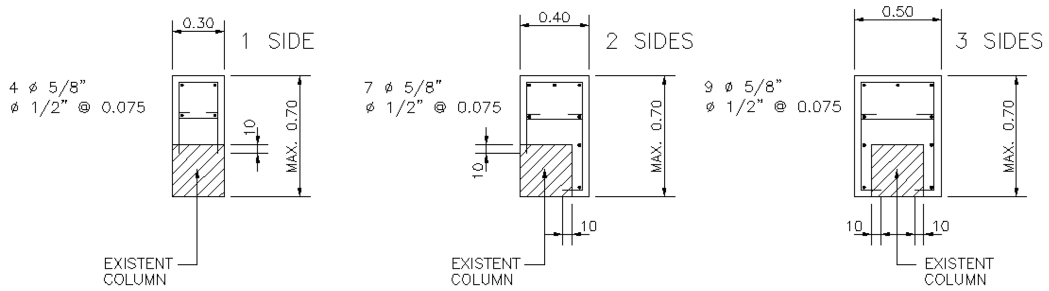


Figure 12. Detail of column with increasing cross section

7.3.2. Results of dynamic analysis

The fundamental periods were as follows: Tx = 0.82 sec and Ty = 0.50 sec. This indicates that they were down 18.81% in X and 13.79% in Y directions, which means that the building substantially increased its stiffness.

The shear forces are: Vx = 93 tons in X direction and Vy = 88 tons in the direction Y. The corresponding seismic coefficients are: Cx = 13.00% and Cy = 12.30%. These seismic coefficients are above the minimum (7.14%) prescribed by the Covenin 1756-2001. The largest drift in the X direction is 10.80 ‰ and occurred at level 2. For the Y direction a value of 0.94 ‰ was obtained at level 3. By comparing these drifts with those obtained in the unreinforced building, we noticed that they were reduced by 10.79% in X and 89.16% in Y directions, indicating a significant increase in the stiffness.

8. CONCLUSIONS

The following conclusions are obtained: (i)The poor performance of the Miramar building was mainly due to the concurrence of deficiencies in concept, design and construction, because: (a) short column effect, (b) presence of soft-story, (c) column dimension and reinforcing steel bars were less than the values reported in the project drawings and (d) no stirrups were placed at the beam-column joints, (ii) The proposed structural reinforcement consisted of: (a) adding inverted V-shaped braces in suitably selected spans in longitudinal and transverse directions in the Eastern and Western units, (b) reinforcing existing columns in several frames of the three units by increasing of sections and jacketing, and (c) repairing damaged columns on the frame 14 of West Unit and separating them from de masonry wall in order to avoid the effect of short column, (iii)The strengthening of the three units of the whole building did not require intervention of the foundations, as the new acting lateral force could be absorbed by the existing precast piles, and (iv)A good performance of the reinforced building is expected during future seismic events similar to what occurred on 12/9/09 and even more intense as the one required by the Covenin 1756-2001.

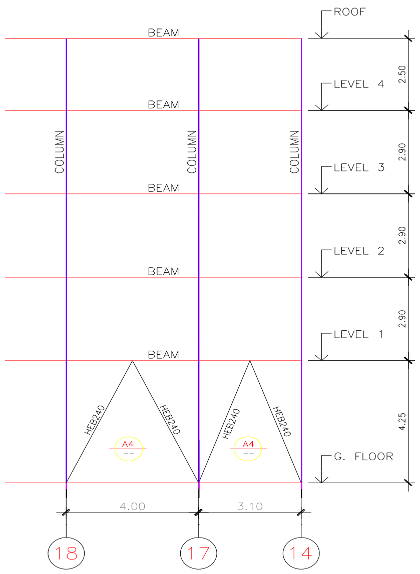


Figure 13. Elevation reinforced frame P in the West Unit

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