STRUCTURAL BEHAVIOR OF "RAIMUNDO MARTINEZ CENTENO" HIGH SCHOOL ON 07/09/1997 CARIACO EARTHQUAKE

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SUMMARY

An earthquake with magnitude Ms 6.8 and epicenter located between the towns of Cariaco and Casanay in northeastern Venezuela, ocurred on July 9, 1997. This earthquake produced the collapse of some engineered structures as "Raimundo Martínez Centeno" high school in Cariaco. The high school had a hollow rectangular shape in plan view and was composed of two buildings units having a "c" shape each one and separated by few centimeters. The structures of both buildings was composed of reinforced concrete frames in two perpendicular directions. The earthquake collapsed the first story in both buildings. Because of their similarity only one of the buildings is analysed in this paper to determine the causes of its collapse. The information needed for the analysis was obtained by means of the collapsed structure inspection, the material analysis of some structural members and the revision of the structural plans. To obtain the global shear capacity of the structure, the strength of the structural members was determined considering the actual transverse reinforcement. With the aid of ETABS program the structure was modelled considering the effect of the short columns caused by the mid-heigh infill walls in the first story. Two earthquakes were considered to determine the global demand on the structure. The comparation between the capacity of the structure and the demands derived from the earthquakes showed that the structure did not have sufficient strenght to sustain the demands. Other important aspect considered in this study was the proximity of the building to the fault trace, where a lateral displacement of 27 cm was registered. This situation could produce a sudden displacement of the structure base during the earthquake. A monotonic increasing lateral load analysis showed that a displacement of about 4 cm between the base and the top can endanger the strength of the structure.

INTRODUCTION

On July 9, 1997, an earthquake with magnitude Ms 6.8, focal depth 10.0 Km, and epicenter located between the towns of Cariaco and Casanay in northeastern Venezuela, ocurred due to the rupture of "El Pilar" dextral transcurrent fault. This earthquake produced damages on several engineered and non-engineered structures. Amongst the former is "Raimundo Martínez Centeno" high school (RMCHS), which was located in the town

of Cariaco and less than 600 m northern from the fault trace, where lateral displacement of about 0.27 m was registerd after the earthquake. Unfortunately no accelerograms were registerd in Cariaco during the 7-9-97 earthquake.

The information compiled for this study showed that the RMCHS was under construction between 1985 and 1987. Based on this, it can be said that the high school suffered some slight earthquakes without damages and did not suffer any strong earthquake until July 9, 1997 earthquake produced its collapse. The aim of this paper is to determine the causes of the collapse of the "Raimundo Martínez Centeno" high school.

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To establish the causes of the collapse of the structure two kinds of analysis are considered. First, a linear-elastic dynamic analysis considering two accelerograms of earthquakes suggested by the U.S.G.S. with similar characteristics of that ocurred on 7-9-97 allows to determine the probable demands that could act on RMCHS during the earthquake. Then the comparison of these demands with the seismic capacity of the structure along with its maximum likely strength reduction factor is done. This analysis evaluate the effect of vibration on the structure. The second kind of analysis considered is a push-over analysis to establish the probable effect of the proximity of RMCHS to the fault trace. The rupture of the fault could produce a sudden displacement of the foundations of the structure during the earthquake.

The analysis of the structures are based on the following venezuelan codes: COVENIN-MINDUR 1753-85 "Reinforced Concrete Structures for Buildings - Analysis and Design", which is based on ACI 318-83 code "Building Code Requirements for Reinforced Concrete", and COVENIN-FUNVISIS 1756-82 "Earthquake Resistants Buildings".

DESCRIPTION OF THE STRUCTURE

RMCHS had a hollow rectangular floor plan and was composed of two c-shaped buildings separated by few centimeters. The longitudinal axis of the high school was orientated along E-W direction. The building 1 (B1) was a three-story structure and the building 2 (B2) was a four-story one. The structural system used in both buildings was reinforced concrete frames, with deep beams in the longitudinal and the transverse directions. The floor system was completed by a thin slab and transverse joists with tile block infills. The foundations were composed of foundation slabs. In both buildings there were masonry infill walls; in some cases there were midheight walls and in other complete-height ones.

The earthquake collapsed the first story in both buildings, as can be seen in figure 1. The inspections made to the collapsed structures showed some interesting factors which affected the structural behaviour of the buildings; amongst them can be mentioned: the collapse of both buildings ocurred in the westward direction coinciding with the direction of the fault trace and was compatible with the movement of the fault, there were no interaction between the buildings during the earthquake since there were no evidence of collision, and it was evident that the midheight infill masonry walls influenced the seismic behaviour of some columns in both buildings.

Due to the similarities between the structural configurations and collapse modes of both buildings, the building 1 (B1) was selected to be analyzed in this work to establish the causes of its collapse. Figure 2 shows the floor plan of B1.

STRUCTURAL MODELING

The present analysis is limited to the behaviour of B1 along its longitudinal direction due to the way it collapsed during the earthquake.

The concrete´s compressive strength and the steel reinforcement´s yield stress were experimentally obtained from the evaluation of some structural members of the collapsed structure. The compressive strength of concrete was 24.5 Mpa and the yield stress for steel reinforcement was 414 Mpa. The results obtained coincided with those specified in the structural plans of the building. The concrete´s modulus of elasticity was obtained according to the recommendations of COVENIN-MINDUR 1753-85 code.

To carry out the 3D linear elastic analysis of B1 it was modelled with the aid of ETABS, a commercial program, considering three degrees of freedom at each floor slab. As the direction of the analysis was the longitudinal one it could be properly assumed that the floor diaphragms were rigid.

To define the length of the structural elements it was considered 50% of rigid arm for the beams and 100% of rigid arm for the columns. In determining the bending stiffness in beams, it was considered the interaction with the slabs. The Paulay and Priestley´s [1992] recommendations were considered in determining the effective moment of inertia of the reinforced concrete structural elements. In the elastic analysis axial, shear, torsion, and bending deformations were considered in the columns, but only bending deformations were considered in beams. The cross-section of the first floor columns was of 0.35x0.35 m, with longitudinal reinforcement ratios which ranged between 1.86% and 3.80%, and they tipically had N°3 hoops spaced at 0.10 to 0.20 m. The cross-sections of the beams at the first level were 0.30x0.40 m in the transversal axis and ranged between 0.30x0.40 and 0.30x0.70 m in the longitudinal axis.

The effect caused by infill masonry walls was considered in three ways. First, the forces caused by the self weight of masonry walls were considered on the beams where they were placed. Second, the effect of masonry wall stiffness was taken into account in the model. Finally, the effect of short column caused by midheight masonry walls was considered when analyzing the structure in the collapse direction. In the analyzed building there were the following short column in the first floor level: A-5, B-3, B-4, and E-3 were short columns of 0.40 m height. A-2, A-3, A-4, F-2, F-3, F-4, and F-5 were short columns of 1.55 m height. The others were columns of 2.40 m height.

The masses considered for the analysis assumed that the contributions of the dead and the live loads were concentrated at the floor levels. The total weight of the structure was estimated in 18.33 MN. The viscous damping considered for the analysis was equal to 5%.

DYNAMIC MODELING

To carry out the dynamic analysis several dynamic models were considered. Amongst them the following two are of special interest:

a) Model 1: For this model the effects caused by the masonry infill walls, the interaction between girders and slabs, and the cracking of the structural members on the stiffness of the structure were not considered. Table 1 shows the dynamic characteristics of Model 1.

b) Model 2: In this model the effect on the stiffness of the structure of the masonry infill walls, the short columns, the interaction between girders and slabs, and the cracking of the structural members was considered. Table 2 shows the dynamic characteristics of Model 2. Model 2 considered all the aspects which could affect the seismic performance of the structure.

Table 1: Dynamics characteristics of Model 1.

Mode Shape	Natural Period (sec)	Translational Mass (x) (%)	Translational Mass (y) (%)	Rotational Mass (%)
1	0.89	0.00	84.55	2.06
2	0.78	86.73	0.04	1.30
3	0.77	1.39	2.13	83.08

Table 2: Dynamics characteristics of Model 2.

Mode Shape	Natural Period (sec)	Translational Mass (x) (%)	Translational Mass (y) (%)	Rotational Mass (%)
1	0.69	98.29	0.06	0.22
2	0.31	0.21	30.20	67.27
3	0.19	0.01	61.92	26.39

STRENGTH OF THE COLUMNS

To determine the capacity of the first floor level´s columns their axial load-bending moment interaction curves (*P-M* curves) and shear strengths were determined in the direction of the structure collapse. The shear capacity of the columns was determined as

$$V_u = V_c + V_s \qquad (1)$$

where V_c is the concrete contribution to shear strength and V_s is the shear reinforcement contribution. V_c and V_s can be determined according with the recommendations contained in Park and Paulay (1975).

In determining P-M curves the following aspects were considered: plane sections perpendicular to the axis of a member are assumed to remain plane after deformation, the ultimate compressive concrete strain was equal to

0.003, the concrete strength was 24.5 Mpa, the steel stress at yielding was 414 Mpa, and the flexural strength reduction factor considered was 1.00.

To consider the effect of shear capacity in flexural behaviour the P-M curves associated with the shear strength of the columns were determined [Mahin and Bertero, 1975]. To obtain each P-M curve the column was considered with both ends fixed. As no loads were considered in the column midspan, the moment distribution resulted antisymmetric, and the axial load was considered constant. For a column of lenght H the maximum moment associated with the shear capacity of the column, *Ms*, was calculated as

$$M_s = (V_c + V_s) H / 2 \qquad (2)$$

In this equation M_s depends on the column axial load resulting its plot in a P-M_s curve. Equation (2) also allows us to consider the effect of midheight infill masonry walls in reducing the effective length of the columns. Figure 3 shows as an example the *P-M* interaction curve and the P-M_s curves associated with different column heights.

AXIAL LOADS IN COLUMNS

The axial loads acting on the floor level columns were calculated considering the live and dead loads of the structure, without considering the effect of earthquake forces. Comparing the axial load for each column with its corresponding *P-M* curve it could be observed that in all cases the computed axial load was lower than the axial load corresponding to the balanced point.

BASE SHEAR CAPACITY

The global seismic capacity of a structure can be determined as the total shear capacity of the columns at the first floor level. This base shear capacity can be expressed as the ratio between the first floor level shear strength and the total weight of the structure.

When all of the columns develop their shear strength simultaneously, and the short column effect is considered, the resulting seismic capacity coefficient for the structure is 33%. When the effect of short columns is considered, and also is considered that all the short columns failed, the resulting seismic coefficient is 18%.

SEISMIC FORCE REDUCTION FACTOR

The maximum seismic force reduction factor (R) that the structure could develop was determined based in the comparison of the detailing actually used in the collapsed structure, with the requirements contained in the COVENIN-MINDUR 1753-85 code. The comparison showed that some code requirements were not satisfied in the structure. Amongst them, the following are of special interest:

- The shear strength of the beam-column joints of the first floor level, analyzed in the direction of the structure collapse, was not sufficient to allow the concurrent beams to develop their flexural capacity. This was due to the absence of transverse steel reinforcement in the beam-column joints.
- In 40 % of the beam-column joints of the first floor level, the summation of the flexural capacities of the beams concurrent to each node was bigger than the summation of the concurrent column flexural capacities.
- The actual amount of transverse steel reinforcement in the columns of the structure was less than the amount prescribed by COVENIN-MINDUR 1753-85 code. The hooks of the hoops were of 90-deg in the structure, although they were considered of 135-deg in the structural plans.
- The lap splices of the longitudinal reinforcement of the columns was done at the bottom of them, instead in their midspan.

Based on the detailing of the structure and the maximum ductility coefficients proposed in COVENIN-FUNVISIS 1756-82 code, it is considered that the structure was no able to develop a seismic strength reduction factor bigger than 2.00.

LINEAR ELASTIC DEMAND

To determine the linear elastic demand that could act on the analyzed structure, two records of earthquakes with similar characteristics of that ocurred on 7-9-1997 were selected, since the 7-9-1997 earthquake were not recorded near the town of Cariaco. The selected records with the recommendations of the U.S.G.S. are the following:

a) N-S component of Loma Prieta earthquake registered at Corralitos Station on October 18, 1989. The station was located less than 0.5 Km from the fault trace. The characteristics of the earthquake are as follows: magnitude Ms = 7, epicentral depth = 17.6 Km, and length of the fault rupture = 130 Km.

b) Longitudinal component of Imperial Valley earthquake registered at Station 942, Array 6, on October 15, 1979. The station was located at 1.3 Km from the fault trace. The magnitude of the earthquake was Ms = 6.9 and the length of the fault rupture = 30 Km.

The linear elastic demands produced by both earthquakes were obtained by means of ETABS program. The demands expressed as seismic coefficients are the following:

a) Loma Prieta earthquake: Seismic coefficient = 95 %.
b) Imperial Valley earthquake: Seismic coefficient = 60 %.

COMPARISON BETWEEN DEMAND AND CAPACITY

To compare the linear elastic demands and the structure capacity, considering the vibration of the structure, the strength reduction factor R needed for each one of the earthquakes considered is determined. In this case R is interpreted as a global ductility demand. The values obtained are as follows:

a) Loma Prieta earthquake: $R = 95/18 = 5.28$.
b) Imperial Valley earthquake: $R = 60/18 = 3.33$.

Both values of R are substancially bigger than 2.00, the maximum value of R determined in point 8, so the capacity of the structure is not able to sustain the demands according with the method used to determine them.

PUSH-OVER ANALYSIS

The rupture of the fault during the earthquake surely produced a sudden displacement of the foundations of the RMCHS structure, due to its proximity to the fault trace. Because of this, a push-over analysis was performed to establish the behaviour of the structure.

To carry out the analysis two load patterns were considered. The first of them was an inverse triangle pattern, and the second was a rectangular one. The load was increased in both cases until the ruin of the structure was produced. Figure ... shows the variation of the seismic capacity coefficient with the relative displacement between the top and the base of the structure. When increasing the relative displacement, both analyses showed that the short columns of 0.40 m length failed first, then the 1.55 m length columns failed, and finally ocurred the failure of the 2.40 m length columns. This is illustrated in figure 4 by the changes in the curves slopes.

It is interesting to note that, in this case, the behaviour of the structure when subjected to the two load patterns considered did not show a significant difference between them, and that in both cases a relative displacement of about 4 cm between the top and the base could endanger the strength of the structure.

CONCLUSIONS

In what follows the main conclusions obtained in this investigation are presented:

1) The seismic capacity of the structure was drastically reduced by the short column effect. This ocurred due to the following reasons: the poor capacity of energy dissipation of the short columns, and their premature brittle failure.

2) The detailing of the structure designed in 1978, and built between 1985 and 1987, did not meet some prescriptions contained in COVENIN-MINDUR 1753-85 code and required by COVENIN-FUNVISIS 1756-82 code. Based on this, the seismic strength reduction factor for the structure could not be bigger than 2.00.

3) The comparison between the demands produced by the earthquakes considered in this study and the capacity of the structure showed, considering the maximum reduction factor of the structure as 2, that the structure had no capacity to sustain the demands. This can explain the collapse of the structure when it is considered that the structure vibrated during the earthquake.

4) Considering the push-over analysis, it can be said that a relative displacement between the top and the base of the structure of about 4 cm can endanger the strength of the structure. This situation could easily occur in RMCHS due to its proximity to the fault trace, where a lateral displacement of 0.27 m was registered.

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Figure 1: Building 1 of “Raimundo Martínez Centeno” High school after 9-7-97 earthquake

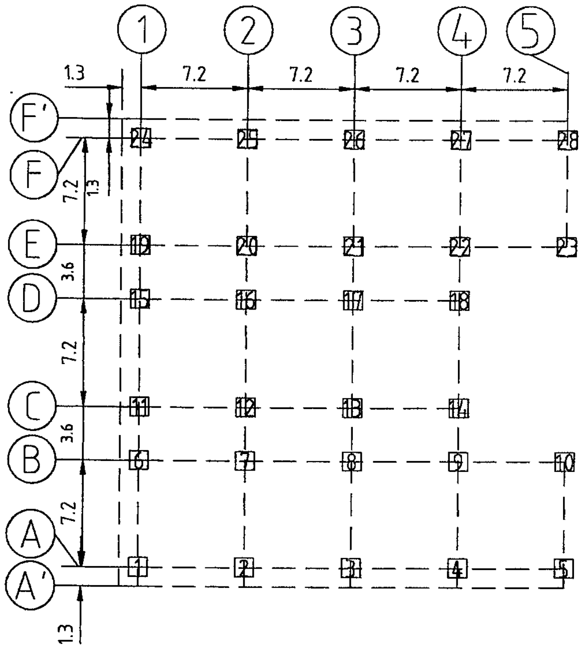

Figure 2: First floor plan of Building 1

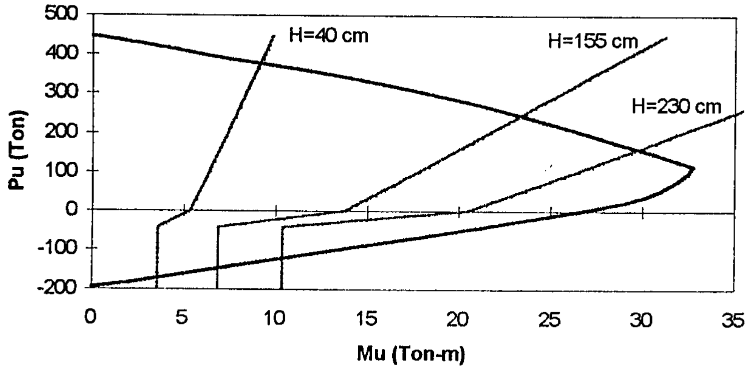

Figure 3: Example of P-M and P-Ms interaction curves of columns in Building 1

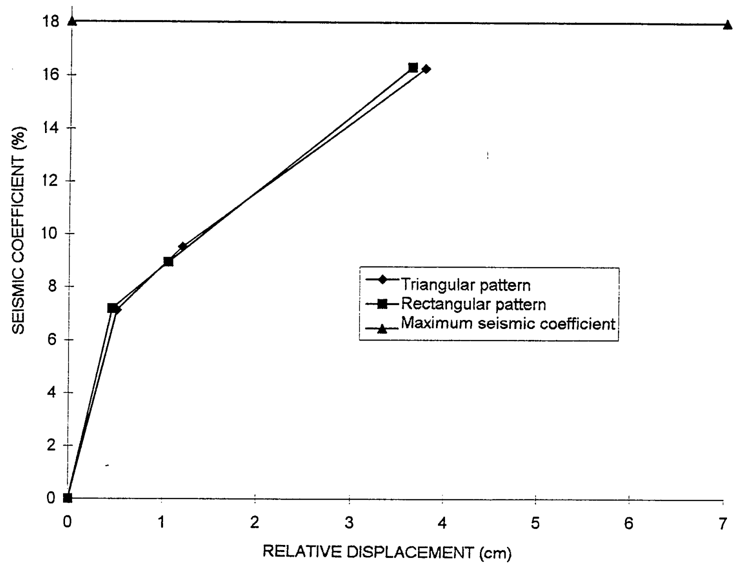

Figure 4: Variation of seismic coefficient vs. relative displacement in push-over analyses