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COMPARATIVE ANALYSIS OF A 12 LEVEL FRAME BUILDING UNDER COVENIN 1756-82 AND COVENIN 1756-2001 STANDARDS

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SUMMARY

The purpose of this study was to perform a comparative analysis of a 12 level moment-resistant frame building under COVENIN 1756-1982 and COVENIN 1756-2001 standards from the standpoint of lateral forces, shear forces, torsional moment, displacements and amount of steel.
The building chosen for this study is reinforced concrete structure located in the Caracas Metropolitan Area, where maximum ground acceleration is 0.30g; the building foundation soil profile is type S3 for both standards. The building's structure, both in elevation and plan, is regular with a symmetry axis in the transversal direction, and it has six (6) frames in the longitudinal direction (X) and four (4) in the transversal direction (Y). The building was analyzed in to orthogonal horizontal directions for both standards, using the dynamics method of one degree of freedom per level and the equivalent static torsion method.
An analysis of the comparative study points to the following conclusions: (i) the fundamental period calculated by modal analysis (Td) is 40% greater in direction X and 46% in direction Y than the estimated fundamental period (Ta), in the case of the 1982 standard; whereas it is only 22% and 27% respectively using the 2001 standard; (ii) the 2001 standard shows a 5% reduction in the dynamic shear forces in comparison with the 1982 standard; the dynamic shear forces calculated using the 2001 standard were greater than the minimum base shear allowed, whereas the values were lower with the 1982 standard and had to be adjusted; (iii) the seismic coefficient was 5.7% lower in direction X and 8.4% in direction Y using the 2001 standard versus the 1982 standard; (iv) the equivalent torsional moment calculated using the 2001 is, in general, 43% lower in direction X and 33% lower in direction Y than that calculated using the 1982 standard; (v) the total drifts found with the 2001 standard were lower than with the 1982 standard; (vi) the columns and beams showed a 3% and 5% increase, respectively, in the amount of steel for the 2001 standard in comparison with the 1982 standard.

INTRODUCTION

Venezuela has a history of more than 130 earthquakes that have caused some type of damage in different areas of the country. The most serious of these quakes in the past 10 years was the 6.8 magnitude Cariaco earthquake, on July 9, 1997, that caused, according to Malaver and Barreiro [1], the collapse of five buildings and serious damage to 2,007 single-family homes in Sucre State.

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It is a well-known fact that the most efficient way of reducing damage caused by earthquakes is to make appropriate use of the earthquake resistant standards for the design and construction of buildings. These seismic standards must be updated on a regular basis, so as to incorporate new technologies that allow for improved earthquake-resistant analysis and design for structures and to take advantage of the lessons learned from earthquakes occurring every so often in different parts of the world. Thus, the purpose of this paper is to provide a comparative analysis of a 12 level moment-resistant frame building under COVENIN standards1756-82 [2] and 1756-2001[3], from the standpoint of lateral forces, shear forces, torsional moments, displacement and the steel areas required, as a contribution to the fine-tuning of the COVENIN 1756-2001 standard.

DESCRIPTION OF THE BUILDING

The building chosen for this study is a 12-story reinforced concrete building, located in the Caracas Metropolitan Area (Figure 1). The structure of the building is regular, as regards both elevation and plan , with a symmetry axis in the transversal direction, except for the mezzanine level, and it has six (6) frames in the longitudinal direction (X) and four (4) in the transversal direction (Y) (Figures 2 & 3).

Figure 1 General View of the Building

The floor slabs are 25 cm thick with reinforcement in one direction only. The building is 35.60 m high (Figures 4 & 5) and the floor dimensions are 27.40 m in direction X and 20.80 m in direction Y.

The building has a total construction area of 5,591 m^2 and a total weight of 5,533.56 tons, calculated on the basis of the specifications in the COVENIN 2002-88 standard [4]. The quality of the concrete used was 250kgf/cm^2, with an elasticity module of $2.53 x 10^9$kgf/m^2, and the steel was 4200kgf/cm^2. The building is founded on a soil profile type S3 for both standards according to Sánchez[5].

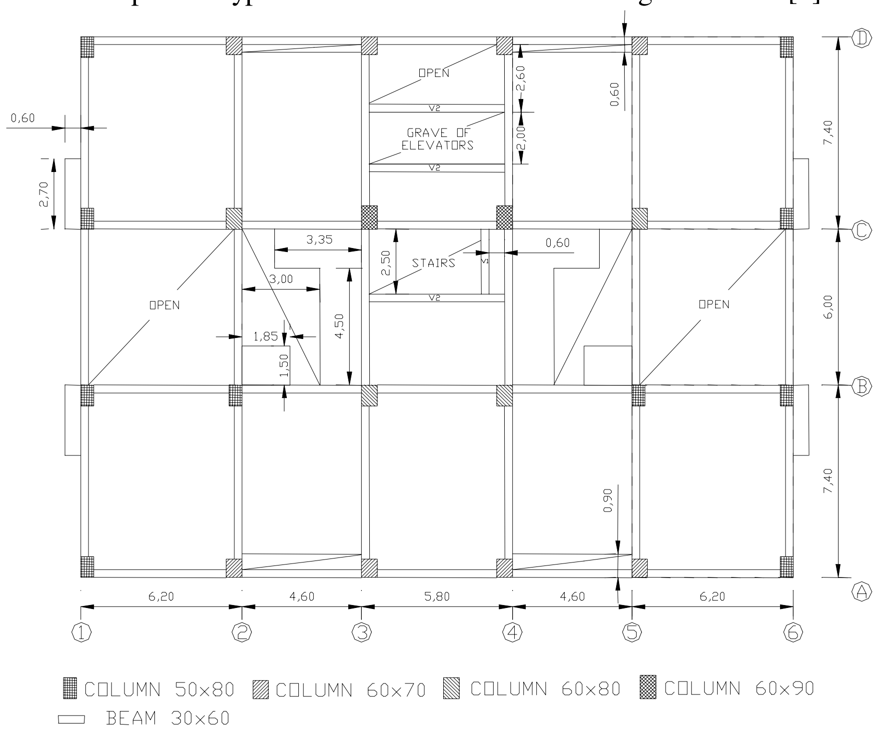

Figure 2 Typical Floor Plan

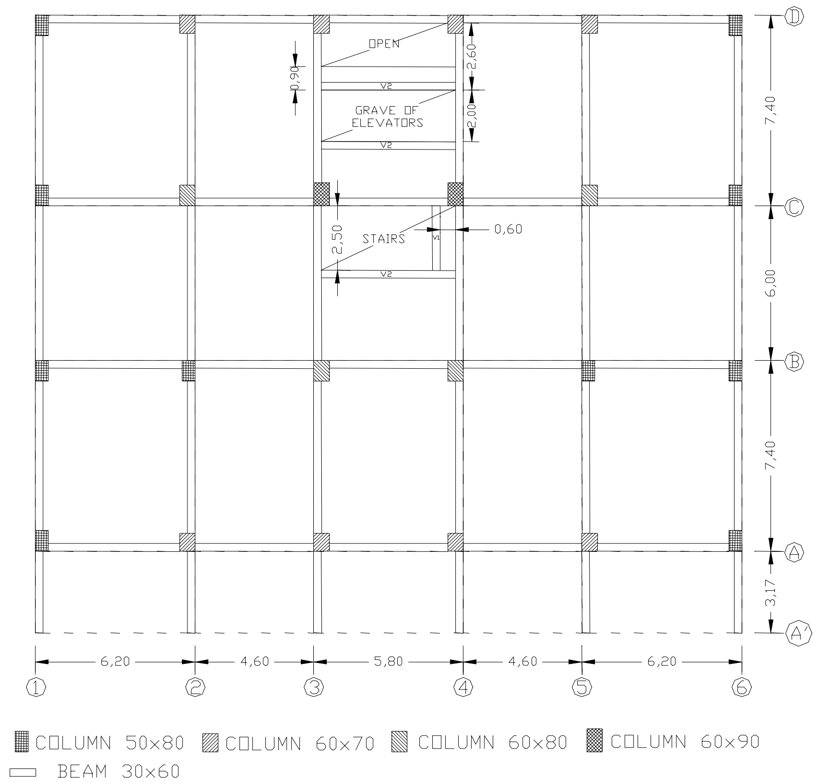

Figure 3 1st Floor Plan

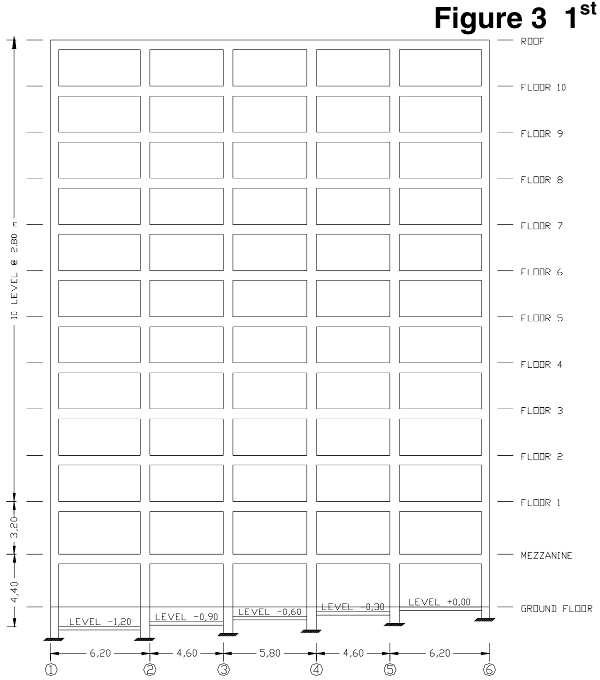

Figure 4 Frame A Elevation

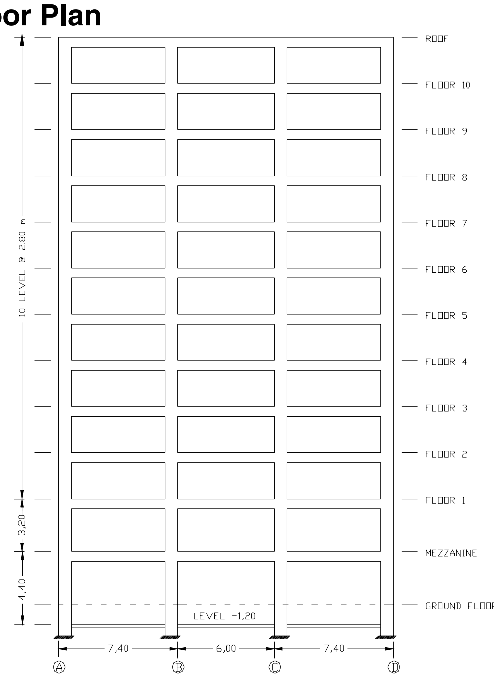

Figure 5 Frame 1 Elevation

CHANGES IN COVENIN 1756-82 AND 1756-2001 STANDARDS

The most important changes made to standard 1756-2001 in comparison with the 1756-1982 standard were:

i). The number of seismic zones was increased from 5 to 8, with a maximum horizontal ground acceleration of 0.40 g in the 2001 standard instead of the 0.30 g under the earlier one.

ii). The soil profiles were increased from 3 to 4 in new standard, and a correction factor φ the was added, changing the value of ground acceleration based on the characteristics of the local soil.

iii). The new standard changed the values of the various terms of the equations of the response spectrum for all types of soil.

iv). The importance factor in the 1982 standard for structures classified in group A (α =1,25), based on the use to which they are put, was raised 4%, and those in group B were subdivided into two groups: B1 and B2; in the case of the first of these two, the importance factor was raised to α =1,15, whereas that for the second subgroup remained the same as under the 1982 standard (α=1.00)

v). The formulas for estimating the fundamental vibration periods under the 1982 standard were changed; furthermore, the values of the response reduction factor were broken down for reinforced concrete and steel structures.

vi). The description of the structural types was expanded and type III was broken down into two subcategories, based on the capacity to dissipate energy generated by the earthquake, with an end result of 5 types of structures.

vii). The estimated period for calculating the minimum base shear was increased from 1.4*Ta under the 1982 standard to 1.6*Ta in the 2001 standard.

viii). The effects of the rotational component of the quake due to uncertainties concerning the location of the centers of mass, centers of resistance and shear (accidental eccentricity) was reduced from 10% of the width of the floor, under the 1982 standard, to 6% in the 2001 standard.

ix). The 2001 standard makes superposition of the orthogonal effects of the quake mandatory, whereas it was only optional under the 1982 standard.

x). The response reduction factors were changed to adjust them to the results of experimental studies and accumulated experience concerning the behavior of buildings affected by strong earthquakes.

xi). Under the 1756-82 standard, total lateral displacement was calculated by multiplying lateral displacement of the level by the ductility; the 2001 standard replaces ductility with 80% of the response reduction factor. The limit values were also reduced under the later standard.

STRUCTURAL ANALYSIS AND COMPARISON OF RESULTS

The building was analyzed in two orthogonal horizontal directions for both standards, using the dynamic method of one degree of freedom per level and the equivalent static torsion method. A three-dimensional mathematical model was created to analyze the structure, thus allowing a coupling of the frames in both orthogonal directions, but limiting the movement to one direction for each direction of the analysis by means of restrictions to the degrees of freedom. The mass at each level is associated with a rigid diaphragm that is displaced on a horizontal plane.

The model was defined as a three dimensional structure made up of beams and columns. In order to simplify the variables included in the analysis, the model did not include the effect neither of partitions nor of the basements. The sections and the masses calculated according to the COVENIN 2002-88 standard were kept constant for this study of the building.

The model consists of 394 nodes, 502 rectangular beams and 288 rectangular columns (Figure 6). The structural analysis was carried out assuming the linear elastic behavior of the building based on Theory-of-Structure principles. The model was made using the program SAP2000 of Computers and Structures Inc. [6]

As the building is located in the Caracas Metropolitan Area the maximum ground acceleration for both standards is Ao= 0,30g; nevertheless, given the characteristics of the local soil, this value is 75% lower under the 2001 standard, as shown in Table 1.

TABLE 1
CHARACTERISTIC SOIL PARAMETERS FOR EACH STANDARD

	MAXIMUM HORIZONTAL GROUND ACELERATION			SOIL PARAMETERS (S3)			
COVENIN	**ZONE**	**φ**	**Ad (g)**	**β**	**T* (sec)**	**T* (sec)**	**p**
1756-1982	4	--	0,30	2,0	1,0	--	0,6
1756-2001	5	0,75	0,225	2,8	1,0	0,4	1,0

Table 1 also gives the values for the initial accelerations of the spectrum and the variations in the soil parameters, where it can been seen that period T^+ has been included for inelastic spectra in the 2001 standard.

In the case of a reinforced concrete frame building, the 1982 standard assigns it a ductility of D=6, which leads to a response reduction factor of 6, whereas the 2001 standard directly assigns a response reduction factor of 6. Figure 7 shows the inelastic response spectra used for the dynamic analysis of the building. In this Figure we see that the COVENIN 1756-2001 standard shows higher acceleration values than does the 1982 standard in the range of periods up to 1.13 seconds. As for the remainder of the values in the descending branch of the spectrum, we see that the values are higher in the case of the 1982 standard; this is due to a lower exponent in the equation. Another important point to be made is that, with the incorporation of the T^+ concept in the 2001 standard, the horizontal plateau in the 2 001 standard is shorter

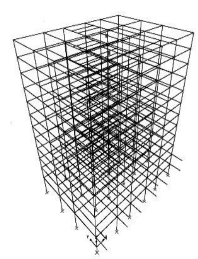

Figure 6 Three-dimensional Mathematical Model.

than when calculated according to the 1982 standard, lading to greater accelerations to more short-period structures.

The result of the modal analysis of the structure was a period of 1,233 sec. in direction X and 1,284 sec. in direction Y (Table 2). This same table shows the estimated fundamental periods for both standards. We must point out that the periods estimated using the equations in the 2001 standard (1,007 sec.) are greater than when using those of the 1982 standard (0,878 sec.), thus indicating that the formulation as given in the 2001 standard gives values that are closer to those resulting from the modal analysis of a frame structure. We must point out that the mathematical model did not include the additional rigidity produced by the partitions and, therefore, the actual period for the building studies must be lower than the one given by the modal analysis.

TABLA 2
FUNDAMENTAL VIBRATION PERIOD

DIRECTIÓN	T Dynamic (seg.)	Ta (Estimated) (sec)	
		1756-1982	1756-2001
X	1,233	0,878	1,007
Y	1,284	0,878	1,007

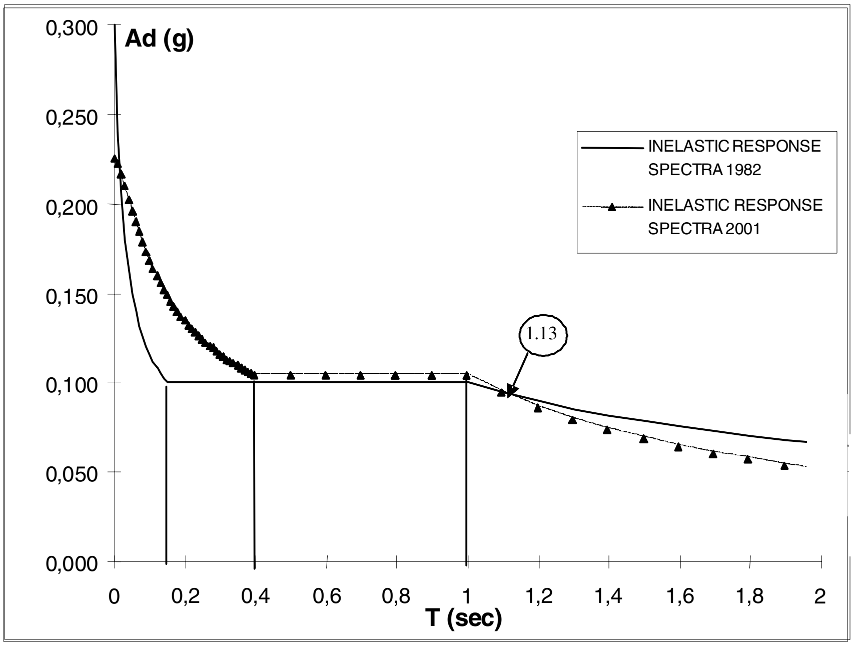

Figure 7 ***Comparison of response spectra under COVENIN 1756-82 and 1756-2001 Standards (R=6.0).***

Table 3 shows the allowable limits for the period calculated using analytical methods following the two standards; this period went from 1.20*Ta under the 1982 standard to 1.40*Ta in the 2001 standard, which allows a wider range of periods for the structure and reflects greater uncertainty in determining the estimated fundamental period versus the real period. This table shows that the dynamic periods calculated are beyond the value allowed by the 1982 standard.

TABLE 3
MAXIMUM ALLOWED ESTIMATED FUNDAMENTAL PERIOD

DIRECTION	T Dynamic (sec.)	T Max. Permisible (sec)	
		1756-1982 1,2* Ta	1756-2001 1,4* Ta
X	1,233	1,053	1,410
Y	1,284	1,053	1,410

Table 4 presents the minimum base shear under both standards; in this table we see that the minimum base shear estimated according to the 2001 standard is considerably lower than the minimum estimated under the earlier standard, the latter having been lower than the base shear determined by means of the dynamic analysis of the structure. On the other hand, the minimum shear estimated on the basis of the 1982 was greater than calculated in direction Y, therefore, the values of the shear distribution per level had to be adjusted in this direction for this standard.

TABLE 4
MÍNIMUM BASE SHEAR FOR THE DESIGN

STANDARD	V (Dynamic)		V Min (estimated) (Ton)
	Direction X (Ton)	Direction Y (Ton)	
1982	408.68	383.98	397.04
2001	385.37	363.67	299.27

The dynamic analysis of the building produced the shear force distribution shown in Figure 8. Looking at this figure, we find that the values of the shear forces in direction X) are 5.3% lower under the new standard than in the 1982 standard, due mainly to the 5% reduction in the acceleration of the response spectrum for the structure's fundamental period in this direction in the 2001 standard; in direction Y , we find an average drop of 7.5% on all the levels, greater than reported in the other direction due to the adjustment made in this direction for the values of the shear forces based on the minimum allowable base shear under the 1982 standard.

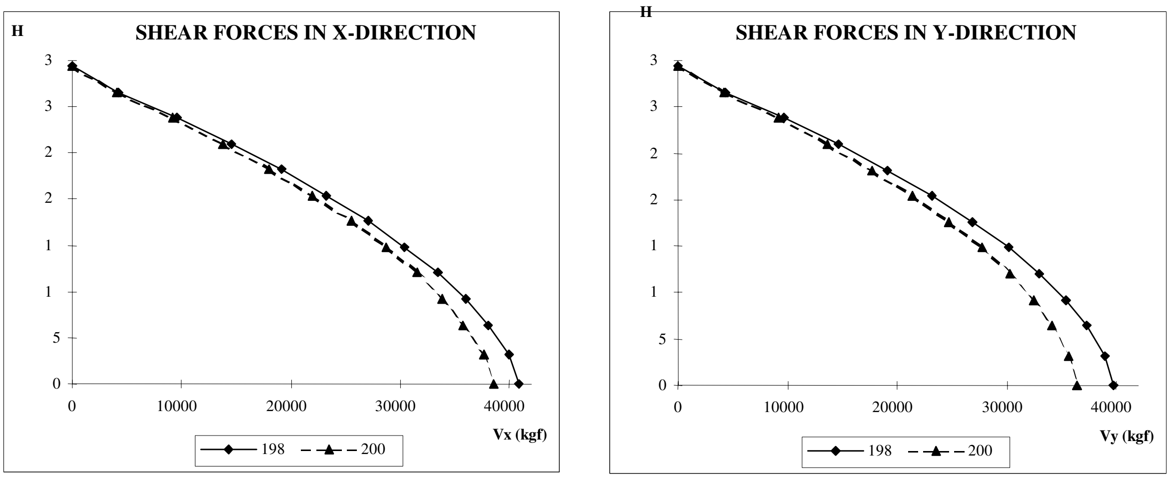

Figure 8 Shear Forces by Level

This reduction of the shear forces in both directions is due to the fact that the structure's fundamental period is located after the crossing point of the descending branches of both spectra (T= 1.13 sec.), which descends more rapidly in the 2001 standard than in the 1982 version.
Table 5 shows the seismic coefficients calculated according to the base shears reported in the dynamic analysis of the building. This table shows that the seismic coefficient under the 2001 standard is lower than under the 1982 standard in both directions.

TABLE 5
SEISMIC COEFFICIENT

STANDARD	DIRECTION X	DIRECTION Y
1982	7,39%	7,18%
2001	6,96%	6,57%

The torsional moments for each level, as shown in Figure 9, were calculated using the equivalent static torsion method; this figure shows that the equivalent torsional moments calculated using COVENIN 1756-2001 standard are 43.2% lower than those calculated on the basis of the 1982 standard in direction X , and 33.1% lower in the other direction.

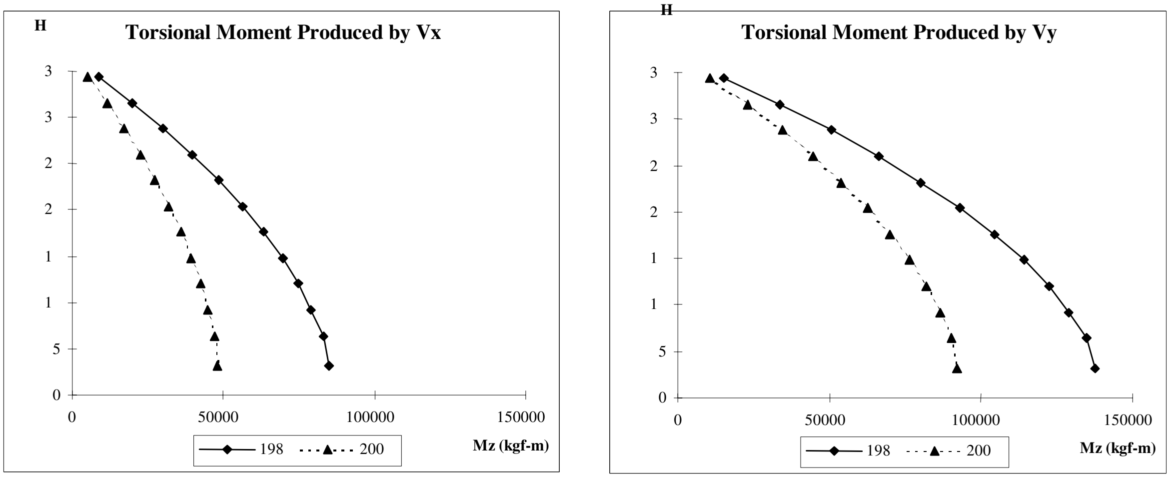

Figure 9 ***Torsional Moments by Level***

The change made in COVENIN 1756-2001 standard for estimating the value of τ is one of the factors causing a major change in the value of the equivalent torsional moments in this standard; under the 1982 standard, the value of τ was selected from three representative values for three types of distributions of plan rigidities. These values did not differentiate between the distribution of rigidities by direction of the analysis and did not depend on the eccentricities of the floor; the changes made in the 2001 standard, which do take these factors into account, providing a value of τ that is more consonant with the actual behavior of the structure.

In Table 6 we see that, for the transversal direction Y , the value of τ_y is increased by the rigidity of the elements in that direction and the eccentricity in relation to the center of rigidity of the story, unlike the value of τ_x, which is lower than the value under the 1982 standard due to the same factors described above.

TABLE 6
DYNAMIC AMPLIFICATION FACTOR (τ)

DIRECTION	STANDARD (τ)	
	1756-1982	1756-2001
X	3.000	2.230
Y	3.000	3.735

Figure 10 shows the building drifts in frames A and 1 respectively (see Figures 4 & 5) oriented in directions X and Y respectively. case This figure clearly shows that the greatest drift is found in frame 1 using the 1982 standard, where drift due to torsion went from representing 22.35% of total drift under this standard to 15.07% under the 2001 standard. In frame A we also find a reduction in the contribution of torsion to the total drift of the building, dropping from 13.19% under the 1982 standard to 7.48% under the new standard. This shows that, under the 2001 standard, displacements caused by torsion on the floor are lower due basically to a reduction of the torsional moments applied.

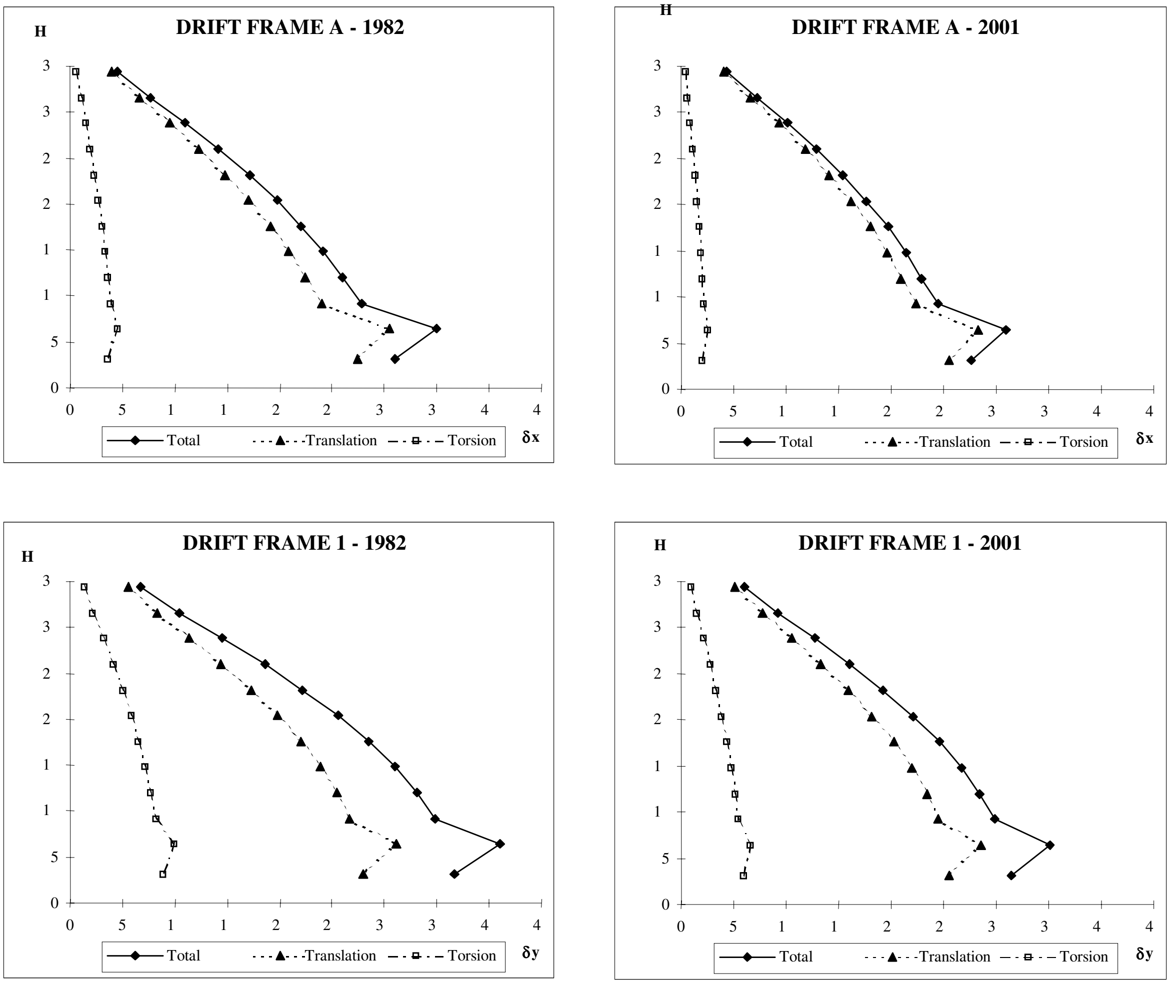

Figure 10 ***Total Drifts in Frame A and Frame 1***

In general, the total drift for frame A was reduced 10.64% under the new standard due to a major reduction in the torsional contribution, which was reduced 76.23% in comparison with the 1982 standard, whereas the drift caused by translation dropped only 4.73%. In the of frame 1, total drift was reduced by 15.33%, with a 48.18% reduction in the torsional drift and an 8.40% reduction in the drift caused by translation, due mainly to the adjustment in the minimum base shear in transversal direction Y . The total drift met all the requirements set in both standards.

In designing the members of the building studied, ten representative beams and ten columns were chosen for application of the specifications in the ACI 318 -2002 [7]. Table 7 shows the most unfavorable load combinations and the members where they occur. Thus we see that: (i) the columns where axial force design is predominant are the central ones and are located on the ground level floor; (ii) the columns with the greatest torsional forces are located on the mezzanine and next floor up, adjacent to the center of that level, which is not the same as the geometrical center of the standard floor, since these are the levels where there are major differences in area and, thus, in mass; (iii) the columns where shear and flexo-compression design is predominant are lateral and are located at the ground-floor level. The beams with the greatest forces are those on the mezzanine and next floor up, levels where there are fewer empty spaces and a slab with a 3.17 m overhang.

TABLE 7
COMBINATIONS OF MAXIMUM LOADS

LOAD	P	V2	V3	T	M22	M33
COMB. COLUMNS	M10	M23	M17	M92	M17	M15
	2	9	5	4	5	9
COMB. BEAMS	--	M128	--	M36	--	M117
	--	9	--	2	--	9

COMB. 2: 1.2*CP+1.6*CV+0.5*CVt
COMB.4 : 1.2*CP+1.0*CV+1.0*CVt
COMB. 5: 1.2*CP+1.0*CV+1.0*CVt
COMB. 9: 1.2*CP+1.0*CV+1.0*CVt

In Table 8 we can see that the internal forces generated by the 2001 standard are lower than those produced by the 1982 standard by 20.41% by shear and 11.17% by moment in direction Y; they are also 5.41% lower in shear and 5.99% lower in moment in direction X. There is no change in the axial force since the vertical seismic effect was not considered in the analysis. In the case of the 2001 standard, however, by torsion there is a 26.39% increase in comparison with the 1982 standard, due mainly to the simultaneous consideration of 100% of the quake in one direction and 30% in the direction orthogonal to that one. In the beams we see an 18.42% reduction of the bending moment.

TABLE 8
INTERNAL FORCES IN BEAMS AND COLUMNS

COLUMNS						
COVENIN	P (kgf)	V2 (kgf)	V3 (kgf)	T (kgf/m)	M22 (kgf/m)	M33 (kgf/m)
1756-1982	499470.48	32832.48	32394.22	1719.65	69113.90	80198.73
1756-2001	499470.48	26130.81	30642.99	2173.39	64970.79	71237.11
% Var.	0.00%	-20.41%	-5.41%	26.39%	-5.99%	-11-17%

BEAMS			
COVENIN	V2 (kgf)	T (kgf/m)	M33 (kgf/m)
1756-1982	26706.27	15099.67	50153.78
1756-2001	25094.31	15099.67	40913.96
% Var.	-6.04%	0.00%	-18.42%

Table 9 gives the areas of steel required in the most stressed beams and columns. There we see a 20.29% and 9.0% increase, respectively, in the areas of steel in perimeter columns (50x80 & 60x70), whereas in the central columns (60*80 & 60*90) we find a reduction of 3.91% and 1.82% respectively. In the case of the beams, we find a small reduction in the lower steel and a slight increase in the upper steel required, both for bending and for shear.

TABLA 9
STEEL AREAS IN THE MOST STRESSED MEMBERS

COLUMN (50 x 80)			
COVENIN	**As-Long. (cm^2)**	**As-Transv X (cm^2/cm)**	**As-Transv Y (cm^2/cm)**
1756-1982	71.29	0.06	0.10
1756-2001	85.76	0.06	0.10
% Var.	20.29%	0.00%	0.00%

COLUMN (60 x 70)			
COVENIN	**As-Long. (cm^2)**	**As-Transv X (cm^2/cm)**	**As-Transv Y (cm^2/cm)**
1756-1982	75.63	0.08	0.08
1756-2001	82.44	0.08	0.09
% Var.	9.00%	0.00%	16.67%

COLUMN (60 x 80)			
COVENIN	**As-Long. (cm^2)**	**As-Transv X (cm^2/cm)**	**As-Transv Y (cm^2/cm)**
1756-1982	49.95	0.08	0.10
1756-2001	48.00	0.08	0.00
% Var.	-3.91%	0.00%	-10.04%

COLUMN (60 x 90)			
COVENIN	**As-Long. (cm^2)**	**As-Transv X (cm^2/cm)**	**As-Transv Y (cm^2/cm)**
1756-1982	54.00	0.08	0.11
1756-2001	54.00	0.08	0.00
% Var.	0.00%	0.00%	-11.30%

BEAM (40 x 60)			
COVENIN	**As-Long. Top. (cm^2)**	**As-Transv Bot. (cm^2/cm)**	**As-Transv (cm^2/cm)**
1756-1982	27.53	20.02	0.11
1756-2001	27.92	19.76	0.12
% Var.	1.01%	-1.32%	1.09%

CONCLUSIONS AND RECOMMENDATIONS

The following are the conclusions and recommendations reached based on the analysis of a 12 level frame building founded on soil type S3 :

i) The fundamental period calculated using the dynamic method (T_d) is 40% greater in direction X and 46% greater in direction Y than the estimated fundamental period (T_a) in the case of the COVENIN 1756-1982 standard, whereas the difference is only 22% and 27%, respectively, using the 2001 standard. This tells us that the estimate of the fundamental period is more accurate in COVENIN

1756-2001 standard; the periods determined dynamically are within the range of maximum periods allowed under the 2001 standard, whereas they are out of range for the 1982 standard.

ii) The COVENIN 1756-2001 standard shows a 5% reduction in the dynamic shear forces in comparison with the COVENIN 1756-1982 standard. The dynamic shear forces under the new standard are greater than the minimum base shear allowed; this was not the case for the values reported using the 1982 and, therefore, they had to be adjusted.

iii) The seismic coefficient for the structure was 5.7% lower in the COVENIN 1756-2001 standard in comparison with the 1982 standard in direction X and 8.4% lower for transversal direction (Y). The minimum base shear in the 2001 standard turned out to be 21% lower than that calculated using the 1982 standard, due to the change from 1.4Ta to 1.6Ta in the estimated period for determining spectrum acceleration.

iv) The torsional moment calculated using the COVENIN 1756-2001 standard is, in general, 43% lower than that calculated using the COVENIN 1756-1982 standard in direction X and 33% in direction Y; this is due to the reduction in the accidental eccentricity and is determined, as it now takes calculation of rigidities

v) based on direction of the analysis into account.

vi) Total drifts under the 2001 standard were lower than with the 1982 standard, due fundamentally to a reduction in the contribution of the torsional effects and shear by level. In both cases, they meet the requirements of the both standards.

vii) The internal forces generated in the columns by the 2001 standard are lower than with the 1982 standard, both as regards shear and bending, but greater from torsion due to the incorporation of the simultaneous action of the quake in two orthogonal directions.

viii) The steel required for the columns increased by 5% and for the beams by 3% with the 2001 standard in comparison with the 1982 standard.

ix) The recommendation is to continue this type of endeavor, analyzing buildings from different periods, types of soil and geographical locations, so as to determine sensitivity in the design of structural members to the seismic actions provided for in the 1756-2001 standard.

REFERENCES

1. Malaver, A. and Barreiro, M. "El Terremoto de Cariaco del 9 de Julio de 1997". Publicación UCAB-OPS, Caracas, 1997.
2. COVENIN. "Edificaciones Antisísmicas, Norma COVENIN-MINDUR 1756-82". Ministerio de Fomento, Caracas, 1982.
3. COVENIN. "Edificaciones Sismorresistentes, Norma COVENIN 1756-01". Fondonorma, Caracas, 2001.
4. COVENIN. "Criterios y Acciones Mínimas para el Proyecto de Edificaciones, Norma COVENIN 2002-88". Fondonorma, Caracas, 1988.
5. Sánchez, A."Análisis Comparativo de un Edificio Aporticado según las Normas COVENIN 1756-82 y COVENIN 1756-98". Tesis de Grado, Facultad de Ingeniería, UCAB, Caracas, 2001.
6. American Concrete Institute. "ACI-318, Building Code Requirements for Structural Concrete and Commentary". U.S. A., 2002.
7. Computer & Structures Inc. (1997). "SAP2000 Integrated Finite Element Analysis and Design of Structures". Detailed Tutorial Including Pushover Analysis, Berkeley, California,1997.