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IMPACT OF SEISMIC CODES ON BUILDING PERFORMANCE IN THE CARIBBEAN

Laura M. Redmond[1], Rebecca M. Stroecker[1], and Reginald DesRoches[1]

[1] Georgia Institute of Technology
North Ave. Atlanta, GA 30332
laura.redmond@gatech.edu, rstroecker3@gatech.edu, reginald.desroches@ce.gatech.edu

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Abstract. *The 2010 Haiti earthquake was a strong reminder of the seismic hazard in the Caribbean and the fact that many buildings in the region are inadequately designed for earthquake loads. Although the Caribbean islands have very similar seismic and hurricane hazard, the design practices across the islands vary greatly. In some regions, building codes are specified and engineer's designs are checked by an authoritative body. In other regions, engineers can choose from among several different codes, and code compliance may not be enforced at all. Finally, there are some countries which do not specify design codes or hazard maps, and the prevailing belief is that designing for the region's large wind forces without considering earthquake design is adequate for the low to mid-rise structures typical of the region. The aim of this study is to determine the influence of different building code choices on the expected performance of mid-rise buildings in the Caribbean.*

The influence of code provisions on the design and performance of structures is characterized by conducting an analytical study of a single concrete moment resisting frame designed by each of the codes permitted in the Caribbean. A total of twelve different building designs were modeled in ETABS and each was subjected to the same suite of ground motions using records with similar period and amplitude characteristics to the present hazard maps from Trinidad. The physical differences between the designs with regard to member sizes and rebar sizes are discussed and the performance of each building as a result of the suite of ground motions is highlighted. The results suggest that all modern code provisions reviewed have similar capacities and seismic performance and if older seismic codes are used to obtain design forces, the design of typical low to mid rise structures in the Caribbean may be sufficient if seismic detailing from a modern concrete code is followed.

1 INTRODUCTION

Although the frequency of seismic activity in the Caribbean is low, the 2010 Haiti earthquake was a major tragedy that spurred new concern for the seismic vulnerability of infrastructure in the region. Although the Caribbean islands have very similar seismic and hurricane hazard, the vulnerability of infrastructure varies greatly. In some regions, building codes are specified and engineer's designs are checked by an authoritative body. In other regions, engineers can choose from among several different codes, and code compliance may not be enforced at all. Finally, there are some countries which do not specify design codes or hazard maps, and the prevailing belief is that designing for the region's large wind forces without considering earthquake forces, but including seismic detailing, is adequate for the low to mid rise structures typical of the region. This study seeks to compare the code provisions and determine the influence of these different choices on the performance of mid-rise buildings in the Caribbean. The influence of code provisions on the design and performance of structures is characterized by conducting an analytical study of a single concrete moment resisting frame designed by each of the codes permitted in the Caribbean.

2 PREVIOUS RESEARCH ON CODE PROVISIONS IN THE CARIBBEAN

Previous research efforts have been conducted by Myron Chin et al. [1] to review existing codes in the Caribbean and make recommendations for improvements of these codes. Additional work has been done by Tony Gibbs reviewing the wind maps in the Caribbean and return period basis [2]. These works have emphasized that the design forces in many of these codes may be inadequate for what is currently accepted as the level of seismic and wind hazard in the Caribbean. However, many of these codes also have more stringent deflection requirements and different resistance factors for material and behavior patterns complicates the ability to assess what impact lower values of design base shear or wind pressure have on a structures vulnerability. To the author's knowledge, a complete analytical study to determine the implications of the variations in code procedures on the final building design has not previously been conducted.

3 SCOPE AND DESIGN OF THE ANALYTICAL STUDY

The scope of this study is limited to the countries of Puerto Rico, Trinidad, Belize, Haiti, the Dominican Republic and Jamaica, selected based on the availability of information from those regions. Knowledge about the current design practices in the region was obtained from in-person interviews [3-19] and phone conversations [20] with design engineers and government officials, and information published by the various national standards associations, research institutions and building authorities [21-26].

In order to isolate the effect of code provisions on the seismic performance of structures in the Caribbean, a single set of wind and earthquake hazard data was used to design a structure by each of the codes used in the Caribbean. Trinidad was selected because it has very detailed seismic maps for PGA, 0.2 second period, and 1.0 second period ground motions at return periods of 2475 years, 975 years, and 95 years [26]. It also has a design peak 3-second gust speed defined by the Ministry of Works [16].

Within the Caribbean, the most common building design is a mid-rise (2-7story) concrete moment frame in-filled with masonry, which is lightly reinforced or unreinforced (Fig. 1). The concrete frame is used to resist lateral and part or all of the gravity loads, and the masonry walls are used to resist wind pressure, self weight, and in some cases, part of the gravity loads**.** The stiffness contribution of the masonry walls are typically ignored in engineering models and only their weight is considered. In this study, the concrete moment frame is designed to resist all of the lateral and gravity forces and the masonry infill weight is accounted for with a heavy cladding load (500lb/ft).

Figure 1: Traditional infill construction in Belize.

The design of the masonry walls were not considered in this study for several reasons. Firstly, and most importantly, the masonry is not designed to resist seismic forces, and is unlikely to perform well in an earthquake. Secondly, the connections between the concrete moment frame and the masonry vary greatly, and the connection types discussed during our interviews may not comprise the entirety of masonry connections in the region. Finally, masonry codes were not able to be obtained from all the regions considered in this study.

3.1 Analytical Procedure

The final analytical study consists of a five-story concrete moment frame building modeled in ETABS. First, a baseline model was assembled and gravity loads were determined using ASCE 7-10, as these loads met or exceeded the gravity loading requirements of all the codes in this study. Gravity loads were kept constant for each subsequent model so that the influence of the seismic and wind codes could be isolated. Designs were based on a concrete strength of 4ksi and a reinforcement yield strength of 60ksi. The soil type was assumed to be equal to an IBC 2006 site class D and a damping ratio of 5% was assumed.

A new model for each of the codes was created using the same initial geometry and gravity loadings as the baseline model. Appropriate seismic and wind loadings were added to reflect the requirements of the equivalent lateral force procedures for the code provision being considered in that model. Each model went through multiple design iterations to satisfy both force requirements and drift limitations for the applicable building code. Drifts were obtained using the applicable displacement amplification factors provided in each code. The forces were output from ETABS and checked for p-delta effects, adequacy of member capacity, biaxial bending and drift requirements.

This resulted in one final building model for each code provision. Each model was then subjected to the same suite of ground motions in order to compare the seismic performance of each structure. A non-linear time history analysis was chosen because the results would best simulate performance in an actual earthquake, and subject the structure to both non-orthogonal and vertical earthquake loads, which are not included in several of the code design procedures. The IBC was used as a benchmark for seismic hazard as many of the Caribbean countries have adopted the IBC, and one can easily compare the buildings' performance to similar construction in the United States. The ground motions selected were scaled to the elastic design spectrum for IBC 2009, using the spectral acceleration maps from Trinidad. The un-factored spectral accelerations were S_s= 1.72g, and S_1=0.391g. The spectrum used to select the ground motions was based on the elastic design spectrum for a site class D, for which the factored spectral accelerations were S_{DS}=1.15g, and S_{D1}=0.422g.

Ground motions were gathered from the PEER ground motion database, using a scaled search to the ASCE 7 design spectrum. Seven ground motions each containing two orthogonal and one vertical component were selected for the ground motion suite. The aim was to get a mix of ground motions that closely matched the design spectrum, particularly around the fundamental period of the structure (Fig. 2).

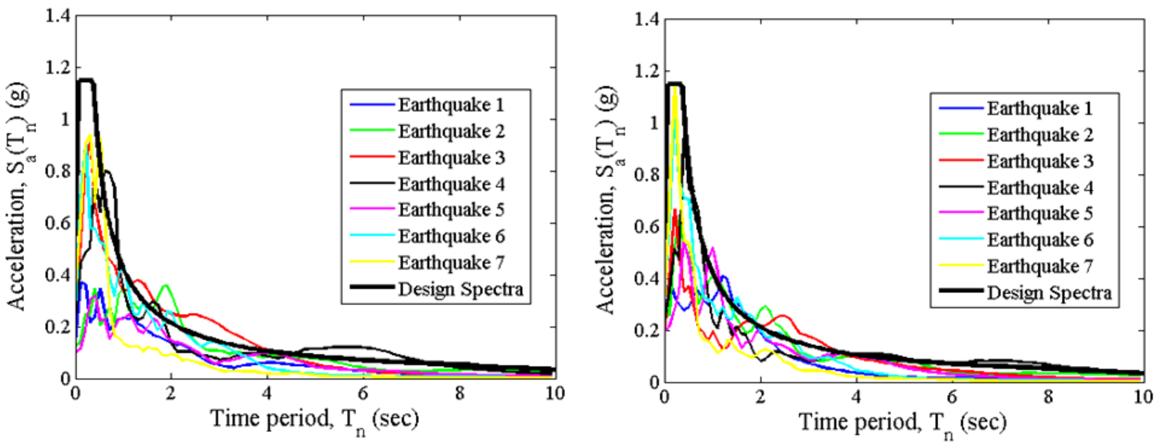

Figure 2: Ground motion suite for x (left) and y (right) directions plotted against the IBC elastic design spectrum.

After each model was subjected to this suite of ground motions, the physical differences between the designs with regard to member sizes and rebar sizes are discussed and the performance of each building as a result of the suite of ground motions is highlighted.

4 PARAMETERS USED IN SEISMIC AND WIND DESIGN

As the IBC was used as a benchmark for this study, the design level values for spectral acceleration and wind speed were based off the design level values for the IBC provisions in Trinidad. For codes based on different return periods, the equivalent ground acceleration for the new return period was calculated from the short period ground acceleration S_s. For wind codes which used the 1-hour average wind speed or 10 minute average wind speed for design, the 3-second gust speed from ASCE 7-10 was converted using the Derst curve. In cases where the return periods were not specified, the methodology for calculating the design spectral acceleration or seismic zone is briefly explained in the following section. Differences between the main design parameters can be found in Table 1.

4.1 Dominican Republic seismic code 2011

The structure was designed as a zone 1 except that the spectral acceleration values were taken as S_s =1.72 and S_1 =0.391 instead of 1.55 and 0.75, respectively. This was done to design each building to a consistent hazard level.

4.2 Dominican Republic seismic code 1979

This code provision contains a procedure based on the fundamental period of the building instead of a spectral map. The procedure was followed exactly, but design earthquake forces were amplified to reflect a slightly greater earthquake risk found in Trinidad. The 0.2s spectral acceleration for Trinidad is 1.72g, which is 11% greater acceleration than the current Dominican Republic code specifies (Ss= 1.55g). For the purposes of this study the design earthquake forces specified in the 1979 Dominican Republic Code were amplified by 11%.

4.3 Caribbean uniform building code (CUBiC)

The design forces were taken directly from the code using the tabulated values for Trinidad.

Code Provision	Initial Design Base Shear (kips)	Load Combinations for Seismic Actions	Drift Limits	Displacement Factor for Equivalent Inelastic Displacement	Resistance Factors for Concrete Design
IBC 2006 & 2009	651.2	1.2D +/- (0.2S_{ds}D) +/- E + 0.5L 0.9D +/- (0.2*S_{ds}D) +/- E **in this study:** 1.43D+E+0.5L 0.97D-E+0.5L 1.13D+E 0.67D-E	0.02	C_d/I **in this study:** 5.5	0.9 flexure 0.75 shear and torsion
UBC 1997	839.0	1.2D +/- (0.5C_aD) +/- E + 0.5L 0.9D +/- (0.5C_aD) +/- E **in this study:** 1.38D+E+0.5L 1.02D-E+0.5L 1.08D+E 0.72D-E	0.02	R **in this study:** 8.5	0.9 flexure 0.75 shear and torsion
DR 1979	504.6	0.75*(1.4D+1.7L+1.7*(1.1E)) 0.75*(1.4D+1.7*(1.1E)) 0.9D+1.43E	0.016	Cd **in this study:** 5.6	0.9 flexure 0.85 shear and torsion
DR 2011	958.6	1.2D +/- (0.3S_{ds}D) +/- E +0.5L 0.9D +/- (0.3*S_{ds}D) +/- E **in this study:** 1.544D+E+0.5L 0.856D-E+0.5L 1.244D-E 0.556D+E	0.016	Cd **in this study:** 4.75	0.9 flexure 0.75 shear and torsion
CUBiC	467.2	0.75*(1.4D+1.7L+1.7*(1.1E)) 0.75*(1.4D+1.7*(1.1E)) 0.9D+1.43E	0.005	1/K **in this study:** 1.25	0.9 flexure 0.85 shear and torsion
CNBC	633.3	1.0D+1.0E+0.5L	0.025	R_0R_d/I **in this study:** 6.8	factors based on material 0.65 concrete 0.85 reinforcement
Eurocode	1318.0	**no national annex for Caribbean countries, used recommended values** 1.0D +/- E +0.3L 1.0D +/-E	None req. for ULS	NA	1/1.5 concrete 1/1.15 reinforcement

Table 1: Values for design of analytical models.

4.4 Summary

The 1979 Dominican Republic Code and CUBiC are much lower than the base shear calculated by the IBC procedure. However, the drift limit for the 1979 Dominican Republic code is more stringent than the IBC and the displacement amplification factor C_d, is larger. CUBiC's displacement amplification factor is very low, but the drift requirements are also much lower than the IBC code, so it is difficult to determine the vulnerability of each structure based solely on the values of design forces.

5 COMPARISON OF MEMBER CAPACITY

The following section compares the final building models designed by the equivalent lateral force procedure for each of the code provisions. The design shear capacities of each building, found in Table 2 were calculated by summing the shear capacities at the base of all the columns at the foundation. It is interesting to note that the design shear capacity of the building designed by the 1979 Dominican Republic is actually greater than the IBC building, although the initial design forces were 22.5% less than the IBC design forces. The base shear capacity of the building designed by CUBiC is 19% less than the IBC capacity, which is nearly the same as the deficiency in initial base shear forces compared to the IBC design forces. Eurocode appears to be much more stringent than the other provisions, but for Ultimate Limit State design there are no drift requirements, so force-based design must be sufficient to ensure good seismic performance.

Code Name	**Base Shear Capacity (kips)**
CNBC	5612
Eurocode	15082
CUBiC	3504
DR 2011	12194
DR 1979	8607
UBC with BAPE	5064
UBC	5730
IBC with BAPE	4340
IBC	4340
Neglect EQ forces	3137

Table 2: Base shear capacity of each model.

The final section sizes and rebar details for the corner columns can be seen in Fig. 3. It is apparent that the section size of the columns when earthquake forces are not considered is significantly smaller than all of the other designs. It is interesting that all of the seismic code provisions resulted in comparable section sizes. However, the shear reinforcement spacing for the columns designed by the 1979 Dominican Republic code and CUBiC are much larger than present in any of the other columns. Large shear spacing may not provide enough confinement and may lead to poor performance of joint connections in seismic events. Additionally, large shear reinforcement spacing is the major reason for lower shear capacity in the CUBiC design compared to the IBC design. If shear reinforcement spacing were reduced to 4", as required by ACI 318-08, the shear capacity of the CUBiC design would increase from 3504 kips to 5252 kips.

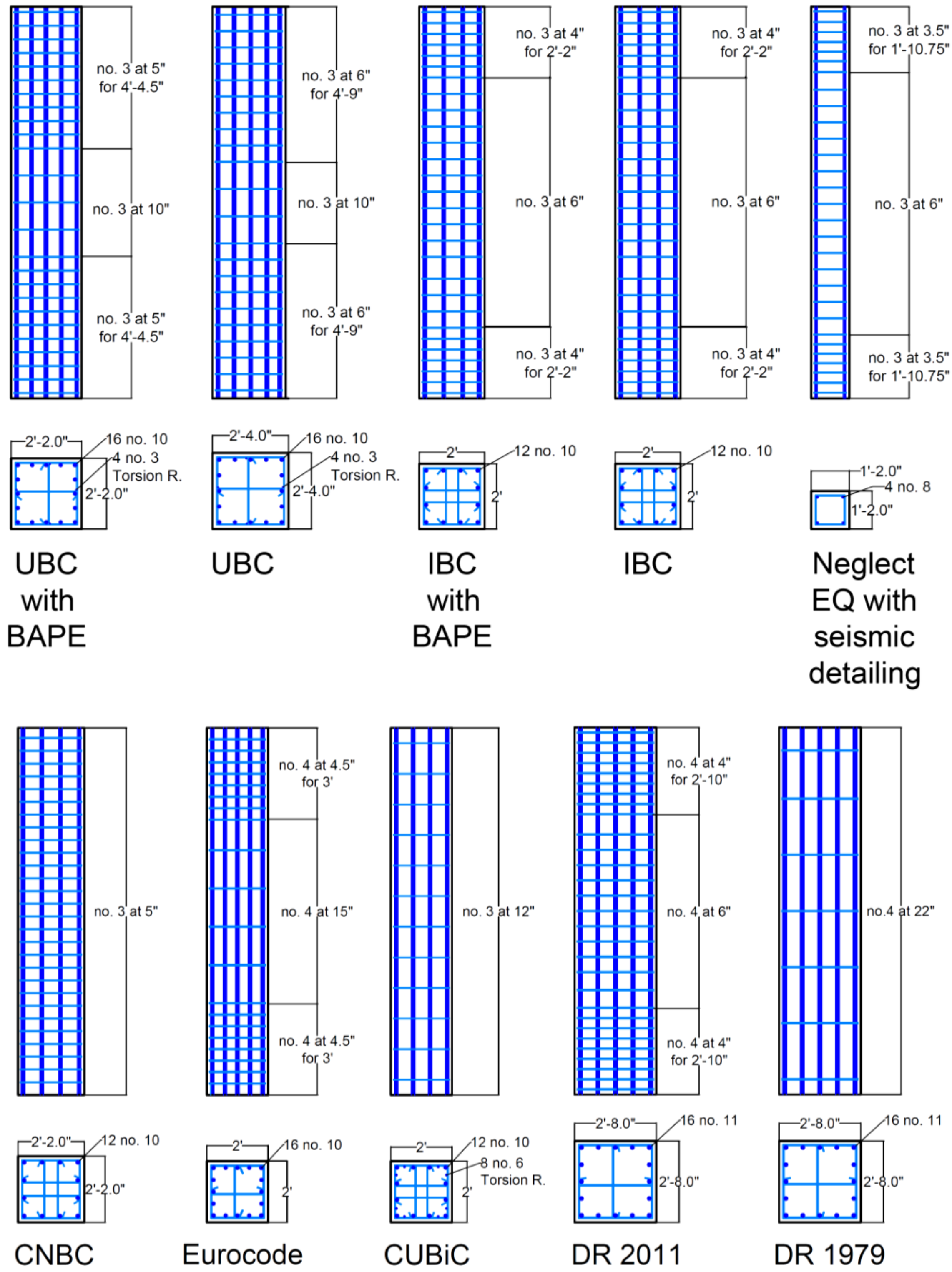

Figure 3: Reinforcement details for all models.

6 COMPARISON OF SEISMIC PERFORMANCE

All of the design buildings were subjected to a suite of ground motions representative of an IBC design-level earthquake. The maximum inter-story drifts for each structure are shown in

Fig. 4. The peak roof displacements for each structure are shown in Fig. 5. The average base shear demand to capacity ratio and the number cases in which base shear capacity was exceeded over the 28 non-linear time history loading combinations is shown in Fig. 6.

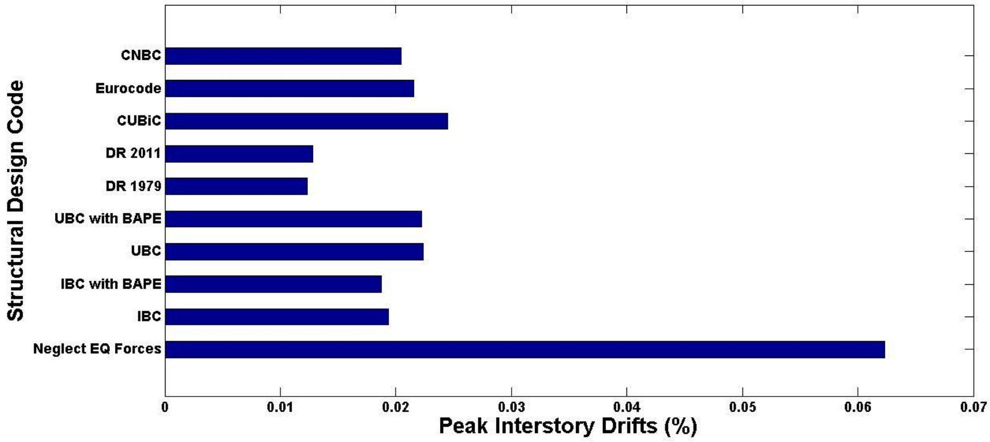

Figure 4: Maximum inter-story drifts.

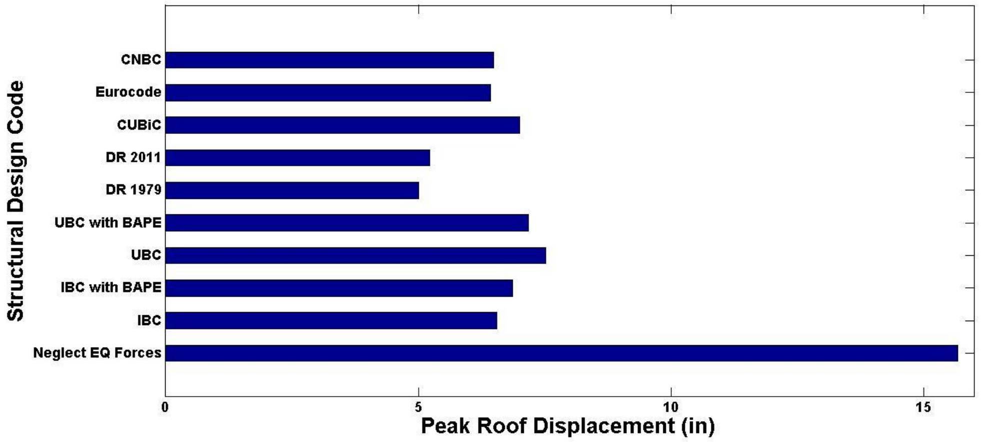

Figure 5: Maximum roof displacement.

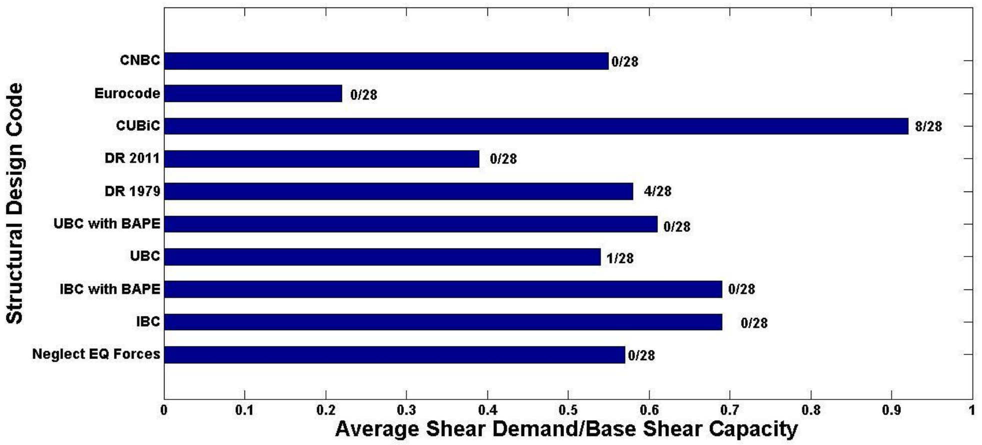

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Figure 6: Shear demand to capacity ratios and number of times base shear was exceeded.

Although all the structures have a demand to capacity ratio less than one, the structure designed by CUBiC has a ratio of 0.92. It is also important to note that the structures designed by 1979 Dominican Republic code and CUBiC experienced base shear forces which greatly exceeded their capacity at some instance within several the seismic events. The capacities of the structure designed by the 1979 Dominican Republic code and the structure designed by CUBiC can be greatly increased by following modern seismic detailing requirements. If the shear reinforcement spacing were reduced to 4" in the 1979 Dominican Republic code, the shear capacity would increase from 8607 kips to 15166 kips. The demand to capacity ratio with the additional detailing would be 0.33 and at no loading cases would have maximum shear demands which exceeded the base shear capacity. If shear reinforcement spacing were reduced to 4", as required by ACI 318-08, the shear capacity of the CUBiC design would increase from 3504 kips to 5252 kips. This would decrease the demand to capacity ratio to 0.61 and base shear capacity would only be exceeded in one out of twenty-eight time history loading combinations. Although the neglect earthquake procedure had sufficient base shear capacity, it also has excessive drifts (>6%) and roof displacements that are over double any other structure considered in this study.

7 CONCLUSION

In completing the study, the various building codes can be separated into three classes of structures. The first, which includes the design procedure neglecting earthquake forces but complying with seismic detailing, has significantly lower capacity than the IBC design and excessive drifts, but sufficient base shear capacity. The second class of structures, which includes the 1979 Dominican Republic code and the CUBiC, had comparable capacity when to the IBC design. However, these structures had large section sizes with less reinforcement and poor reinforcement detailing, which resulted in permissible levels of inter-story drift, but insufficient shear capacity. The third class of structures, which includes the CNBC, Eurocode, the 2011 Dominican Republic code, the UBC and the IBC, are both adequately designed and detailed.

It does not appear to matter significantly which design procedures are used to design buildings in the Caribbean, provided they are modern provisions. If older seismic codes are used to obtain design forces, the design of typical low to mid-rise structures in the Caribbean may be sufficient if seismic detailing from a modern concrete code is followed as the base shear capacity is significantly increased. However, even with the addition of modern seismic detailing, tall or irregular structures may still be poorly designed according to some of the seismic provisions due to the lack of irregularity checks, the use of only orthogonal seismic loadings and the failure to consider p-delta effects. It is difficult to determine if neglecting to design for earthquake forces and merely providing seismic details of the reinforcement is sufficient for good performance in a design-level earthquake, because the structure had adequate base shear capacity but excessive drifts.

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