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SEISMIC PERFORMANCE ASSESSMENT OF A 24-STORY BUILDING WITH COMPOSITE MOMENT RESISTING FRAMES (CFT-MRFs) AND COMPOSITE BUCKLING-RESTRAINED BRACED FRAMES (CFT-BRBFs)

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Abstract

Reinforced concrete is the preferred material used in Ecuador for most buildings. Moment Resisting Frames (MRFs) is the most common structural system chosen for reinforced concrete construction. However, steel construction has gained some popularity during the last 15 years in Ecuador. Most of the steel buildings constructed in this period are low-rise structures composed mainly of MRFs. This paper presents the initial steps of a research program that aims to provide the local steel industry with other alternatives for structural systems of taller buildings. Composite Moments Frames (CMRFs) consisting of concrete-filled steel tube columns and I-shaped steel beams (CFT-MRFs) as well as Buckling-Restrained Braced Frames (BRBFs) are the structural systems that are proposed in this paper to be considered in the seismic design of mid-rise and high-rise buildings. Ecuadorian seismic code (NEC 2015) does not include design provisions for the CFT-MRFs or the BRBFs due to a lack of knowledge relative to the analysis, design, construction and behavior of buildings using this type of structural systems. Therefore, there is a need to carry out both analytical and experimental research on these systems in order to make feasible their application in future projects. This work addresses the analytical studies of the proposed research program. The main objective of this investigation is to evaluate the global seismic response of a prototype building composed of CFT-MRFs and CFT-BRBFs. The first part of the paper provides a review of the most relevant aspects related to the design of a 24-story prototype building where CFT-MRFs and CFT-BRBFs constitute the main lateral load resisting system. This design was based on the seismic provisions from American codes and recommendations from related research. The second part of the paper describes the analytical modelling of the prototype building and the calibration that was done for every structural component (beams, CFT columns, beam-to-column connections and BRBs) pertaining to both structural systems. Two types of nonlinear analyses were conducted: static pushover analyses; and time-history analyses with several ground motions. Finally, the results of the nonlinear static and dynamic analyses performed on the prototype building are presented. The results of the seismic evaluation of the prototype building show that both systems, the CFT-MRFs and the CFT-BRBFs, have an adequate seismic performance and consequently represent an interesting alternative for the construction of mid-rise and high-rise buildings in Ecuador.

Keywords: seismic evaluation; composite moment resisting frames; CFT columns; buckling-restrained braced frames; buckling-restrained braces

1. Introduction

This paper presents the initial steps of a research program that aims to study analytically and experimentally two structural systems commonly used for mid-rise and high-rise buildings in several countries. Composite Moment Resisting Frames consisting of concrete-filled steel tube (CFT) columns and steel beams (CFT-MRFs) as well as Buckling-Restrained Braced Frames (BRBFs) are the structural systems proposed in this program as an alternative to those used for conventional steel and reinforced concrete construction in Ecuador. The research program has been developed so that it can be conducted in four phases: (1) analytical study on the seismic design and behavior of the CFT-MRF and BRBF systems; (2) testing and qualification of CFT moment connections including T-shaped and cruciform-subassemblage specimens; (3) testing and qualification of buckling-restrained braces (BRBs) including individual brace and brace-subassemblage specimens; and (4) development of provisions for the design and construction of buildings composed of CFT-MRFs and/or BRBFs in Ecuador.

This paper highlights the results of phase 1 of this research program. The criteria for the analysis and design of the CFT-MRF and BRBF structural systems are presented. The design of a 24-story prototype building, where CFT-MRFs and BRBFs constitute the main lateral load resisting system, is summarized. Finally, the most relevant results of the nonlinear static and dynamic analyses performed on the prototype building are discussed.

2. Prior Research

Reinforced concrete is the preferred material used in Ecuador for most buildings. Moment Resisting Frames (MRFs) are the most common structural system chosen for reinforced concrete construction. However, steel construction has gained some popularity during the last 15 years in Guayaquil, Ecuador. Most of the steel buildings constructed in this period are low-rise structures composed mainly of MRFs. The authors believed that after the 2016 Pedernales earthquake steel construction will become even more popular due to the inadequate performance of several hundred reinforced concrete buildings [1].

A research project was conducted in 2009 to study the state of the art in the design and construction of steel buildings located in Guayaquil city [2]. The results of this project revealed that some of the steel buildings constructed before 2008 had some deficiencies relative to the design, construction details (e.g., use of non-prequalified connections, inadequate welding), workmanship, and inspection. In addition, some of these steel buildings were designed without considering earthquake load effects since previous editions of the Ecuadorian Building Code (CEC) did not include seismic requirements for steel structures. As a consequence, these structures may exhibit a non-ductile behavior under strong ground motions [2].

The current edition of Ecuadorian Building Code (NEC-15) [3], the standard that rules the design and construction of structures in Ecuador, does include seismic design criteria and construction requirements for steel structures. These provisions are included in the section *"NEC-SE-AC – Estructuras de Acero"* of the NEC-15. However, it only provides three alternatives for structural systems that may be used as the lateral load resisting system in steel buildings. These three systems are: Special Moment Resisting Frames (SMRFs), Special Concentrically Braced Frames (SCBFs) and Special Eccentrically Braced Frames (SEBFs). The SMRFs have been the most used while the SCBFs have had very few applications. The authors know that the EBFs are currently being used for first time in the rehabilitation of a hospital building in Guayaquil. The design provisions of these structural systems are based upon current American codes. Composite construction is not included in the Ecuadorian building code. Furthermore, most of the tallest buildings in the country are made of reinforced concrete and the authors are unaware of steel buildings exceeding 20 stories high. For this reason, a study on the CFT-MRF and the BRBF structural systems for a 24 story building is proposed to provide the local steel construction industry with other alternatives for structural systems of taller buildings.

Extensive analytical and experimental research has been carried out on these systems (CFT-MRFs and BRBFs) in several countries, mostly in the United States and Japan. They have demonstrated superior performance under earthquake loading and have shown more benefits compared with traditional steel or reinforced concrete moment resisting frames and steel braced frames [4, 5].

Composite Moment Resisting Frames having CFT columns have economical and practical advantages in comparison with common reinforced concrete and steel moment frames. CFT columns provide the system with greater lateral stiffness than steel members and the interaction between steel and concrete results in members with larger axial and flexural capacity. The steel tube provides a confining effect to the concrete as well as the local buckling of the steel tube is delayed by the restraining effect of the concrete infill. CFT columns can optimize the amount of steel and the use of fireproof material can be reduced due to concrete fire resistance. In addition, use of formwork is not necessary which results in workmanship cost savings and project time reduction [6, 7].

Buckling-Restrained Braced Frame (BRBF) is a particular type of concentric braced frame that incorporates buckling-restrained braces (BRBs). A BRB consists of a slender steel element, named as steel core, which carries the entire brace axial load, and a restraining mechanism that eliminates the buckling failure mode of the core under high compression forces. A BRB is able to achieve yielding under both tension and compression without significant degradation of strength or stiffness. Consequently, BRBFs present stable hysteretic response, significant ductility and large energy dissipation capacity when subjected to cyclic loading. Furthermore, BRBs have superior seismic performance compared to conventional braces [8, 9].

3. Design of Prototype Building

A prototype building was designed using seismic provisions from American codes and recommendations from related research. The building is a 24-story office building located on stiff soil in Guayaquil-Ecuador, an area catalogued as a high seismic risk zone. A typical floor plan of the prototype building is shown in Figure 1. The floor plan has an area of 432.00 m^2. Elevation views of a typical frame in the X-direction and typical frames in the Y-direction are presented in Figure 2. The height of the first story is 4.25 meters and the height of the remaining stories is 3.25 meters, resulting in a total height of 79.00 meters above the ground floor level. The building has a basement with dimensions in plan of 45 by 28 m and the foundation (not shown in Figure 2) is located 5 meters below ground floor level. The plan and vertical dimensions of the building are typical in Ecuador. There are no vertical or horizontal irregularities in the building.

3.1 Prototype frames

The lateral load resisting system of the building is comprised of CFT-MRFs in the X-direction and a combination of CFT-MRFs and CFT-BRBFs in the Y-direction. The exterior frames are CFT-BRBFs while the interior frames are CFT-MRFs in the Y-direction. Contrary to the US construction practice, all frames (four in each direction) are assumed to resist both gravity load and seismic load effects. The building has three 9.00 meters bays in the X-direction; and two exterior 5.00 meters bays plus a central 6.00 meters bay in the Y-direction. As shown in Figure 2(b), a single diagonal brace configuration is used in the exterior bays of the CFT-BRBFs.

The columns are welded built-up square CFT columns made of ASTM A992 steel plates (345 MPa nominal yield strength) and normal strength concrete infill (34.3 MPa nominal compressive strength). The beams are I-shaped welded built-up members made of A36 steel plates (247 MPa nominal yield strength). The dissipative core of the buckling-restrained braces consists of a flat plate made of A36 steel (247 MPa nominal yield strength) while the buckling-restraining system is composed by a steel tube, formed by two welded cold-formed channels, filled with concrete mortar.

To justify the use of composite moment frames in one direction and composite braced frames in the other, the prototype building dimensions were established such that its slenderness ratio (H/B) is about 3 and 5 in the X-direction and Y-direction, respectively.

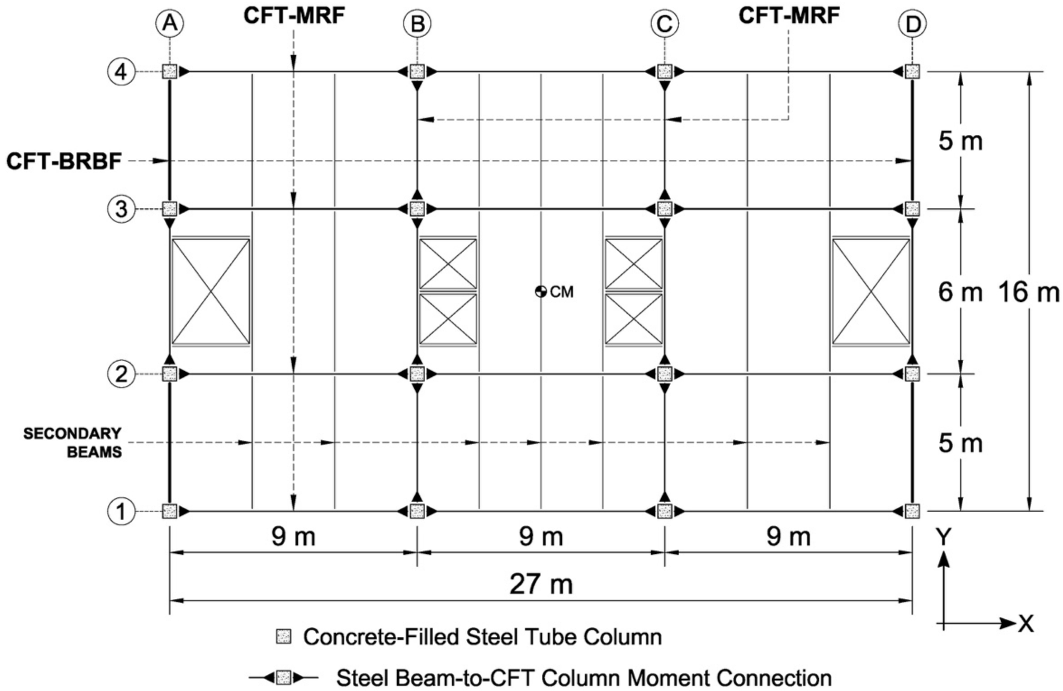


Fig. 1 – Typical floor plan of the prototype building

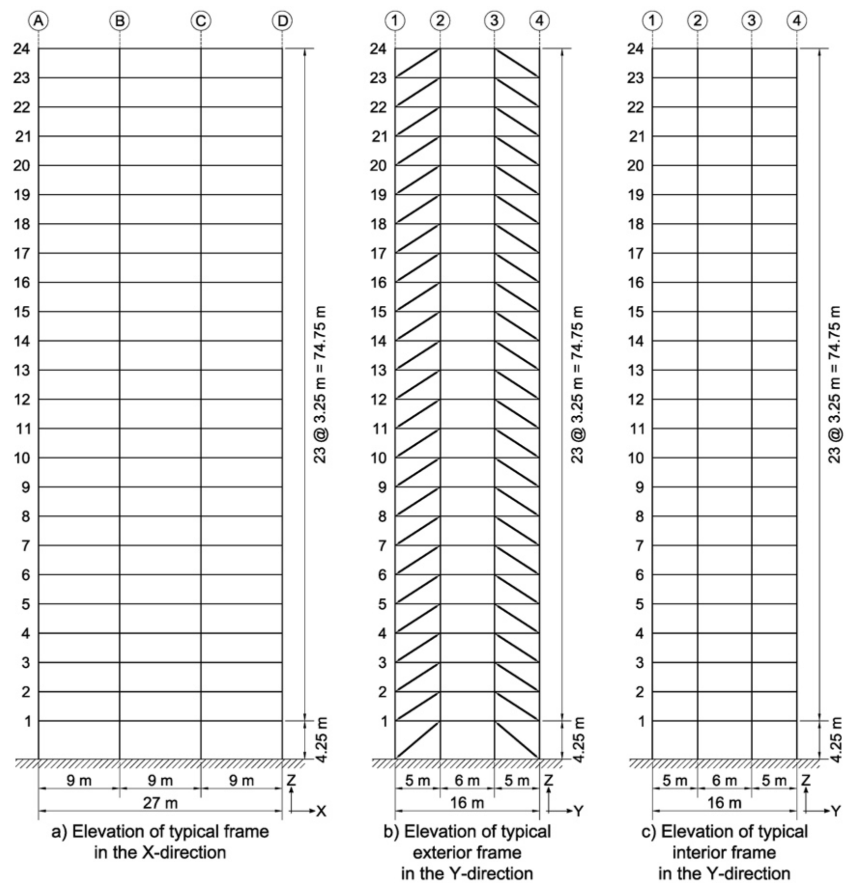


Fig. 2 – Elevation of the typical frames in the X and Y direction

3.2 Proposed seismic design approach

Performance-based design (PBD) philosophy was used for the CFT-MRFs and CFT-BRBFs studied in this project. Two seismic hazard levels were used: the Maximum Considered Earthquake (MCE) level, having a 2% probability of being exceeded in 50 years; and the Design Basis Earthquake (DBE) level, defined as two-thirds of the MCE. The proposed design approach for CFT-MRFs and CFT-BRBFs has two performance objectives: (1) to achieve the Life Safety (LS) performance level when subjected to a DBE level ground motion; and (2) to achieve the Collapse Prevention (CP) performance level when subjected to a MCE level earthquake.

3.3 Design of prototype frames

The prototype frames were designed according to the ASCE/SEI 7-10 Standard [10] and the AISC Seismic Provisions [11]. The modal response spectrum analysis (MRSA) described by ASCE/SEI 7-10 was used to estimate the earthquake load effects on both frames. The design accelerations were determined using the deterministic limit of the ASCE/SEI 7-10 site-specific procedure. The effective seismic weight of the prototype building, W, was estimated as 7280 tons. As established in ASCE/SEI 7-10, the system design coefficients used are those listed in Table 1. The final member sections are presented in Table 2. ASCE/SEI 7-10 provisions allow a maximum interstory drift ratio equal to 0.020 for both structural systems. The building was assigned to Seismic Design Category D with a seismic importance factor of 1.0 according to the Table 1.5-2 of ASCE/SEI 7-10 [10].

Table 1 – System design coefficients

Lateral Load Resisting System	Response Modification Coefficient, R	Overstrength Factor, Ω_0	Deflection Amplification Factor, C_d
Composite Moment Resisting Frames	8	3	5.5
Buckling-Restrained Braced Frames	8	2.5	5

Table 2 – Prototype frames member sizes

Floor	Columns (mm)	Beams (mm)				BRBs (mm)	
		X-direction frame		Y-direction frame		Y-direction frame	
		Flanges	Web	Flanges	Web	Steel Core	Steel Casing
21 to 24	CFT 500x500x15	250x18	500x8	200x16	400x8	PL60x15	[] 200x200x3
17 to 20	CFT 500x500x15	250x20	500x8	200x16	400x8	PL80x15	[] 200x200x3
13 to 16	CFT 500x500x18	275x20	550x10	250x18	450x8	PL100x15	[] 200x200x4
9 to 12	CFT 500x500x18	275x22	550x10	250x18	450x8	PL100x18	[] 200x200x4
5 to 8	CFT 500x500x20	300x22	600x10	300x20	500x8	PL120x18	[] 250x250x6
1 to 4	CFT 500x500x20	300x25	600x10	300x20	500x8	PL120x20	[] 250x250x6

To estimate the member design forces and to determine the structure displacements, a three-dimensional (3D) finite element model of the prototype building was developed and a linear elastic analysis was performed using the SAP2000 software [12]. Rigid diaphragm constraints were defined at each floor level and torsional effects due to accidental eccentricity were included. Additional criteria for adequate modelling of the CFT columns and BRBs stiffness was taken into account [7, 9]. The vibration period of the prototype building was 3.11 seconds for the X-direction and 2.90 seconds for the Y-direction. The design base shear was 0.044W for both frames which is the minimum value allowed by ASCE/SEI 7-10 provisions [10]. Figure 3 shows the amplified elastic story displacements and interstory drift ratio distribution. The amplified elastic roof displacements were 877 mm (0.0111 times the total building height) and 741 mm (0.0094 times the total building height) for the X-direction and the Y-direction analysis, respectively. The maximum interstory drift ratios were 0.0143 and 0.0114 for the CFT-MRFs and the CFT-BRBFs, respectively. The greater stiffness of the CFT-BRBFs is shown in Figure 3 since the profile displacement along the height of the building is less than that of the CFT-MRFs. A similar conclusion can be inferred when observing the interstory drift profiles of both frames along the height of the building except at the upper stories. It is known that braced frames are usually stiffer at the lower stories than at the upper stories.

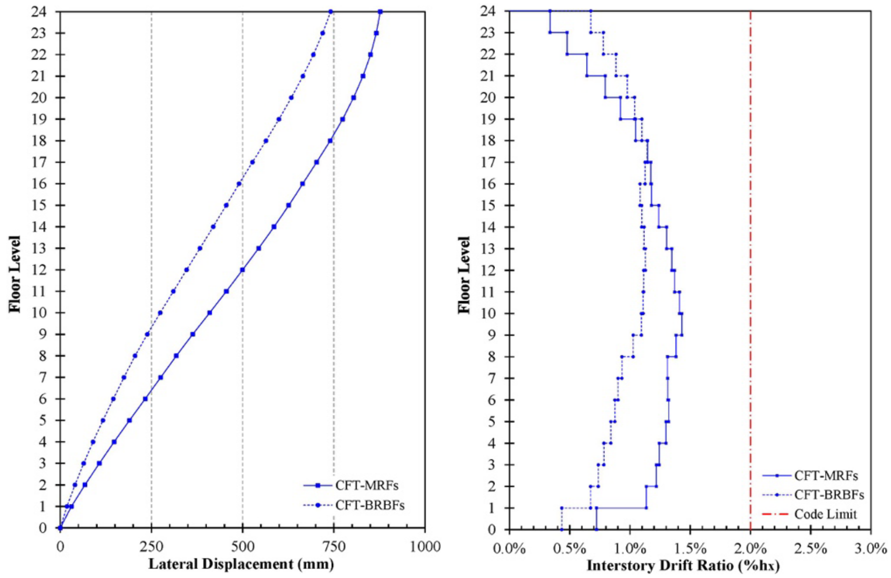


Fig. 3 – Story displacement and interstory drift ratio distribution along prototype building height

As can be seen in Figure 3, the maximum interstory drift ratios for both frames are well below the code limit value of 0.02. During the design process, the authors decided to keep these values since interstory drift ratios obtained from nonlinear time history analyses, for DBE ground motions, are usually larger than those estimated with linear elastic analyses [13]. Thus, structural and nonstructural damages are going to be less detrimental than those if the building were designed with lighter sections which result with maximum interstory drift ratios close to the 0.02 code limit value. It is important to mention that during the 2016 Pedernales Earthquake, in addition to the RC buildings that collapsed or were severely damaged in the areas affected by the earthquake, some of the RC buildings that did not experience structural damage had important nonstructural damage due to large interstory drifts ratios [1] since the nonstructural components used in Ecuador generally are inadequate to resist seismic loads.

3.3.1 CFT-Moment Resisting Frame (CFT-MRF)

Story drift requirements controlled the selection of the structural member sizes. In addition, the members of the CFT-MRFs were proportioned to satisfy a strong column-weak beam design, according to the AISC Seismic Provisions [11]. The column-to-beam moment capacity ratio is established as 1.0 as explained in the AISC Seismic Provisions. The CFT columns and I beams were designed according to the AISC 360-10 Specifications [14] and AISC Seismic Provisions [11]. The combined axial and flexural capacity for each CFT column size was determined based on the Plastic Stress Distribution Method as described in AISC Specifications, and compared with the obtained demand. The dimensions of all members were established to satisfy the limits of the width-to-thickness ratio for highly ductile members as required by the AISC Seismic Provisions. A CFT moment connection with interior diaphragms (continuity plates) was used in this study. The prequalified reduced beam section (RBS) moment connection type was implemented in the CFT-MRFs. The beam-to-column connections were designed following the design procedure described in AISC 358-10 [15] for RBS moment connections.

3.3.2 CFT-Buckling-Restrained Braced Frame (CFT-BRBF)

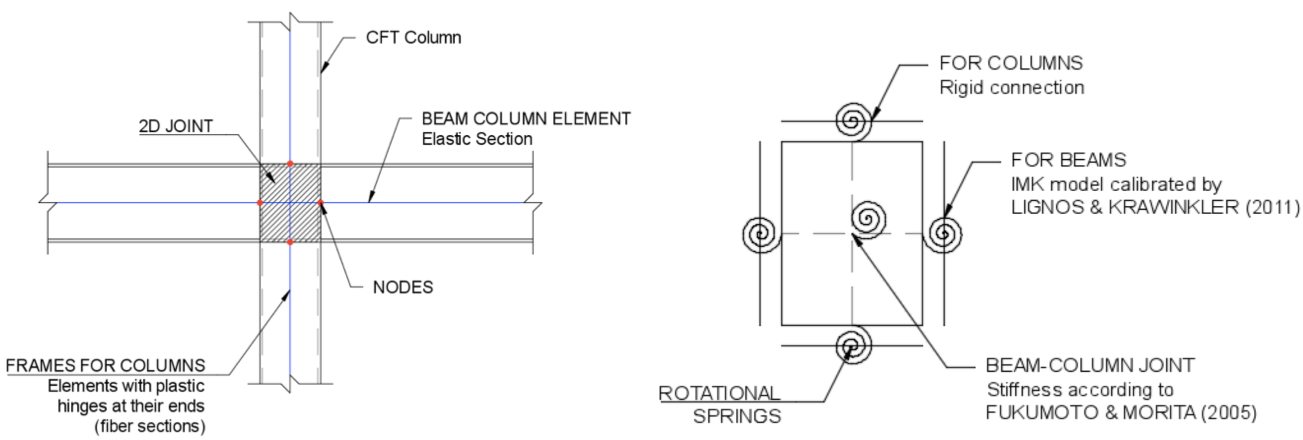
In the case of the CFT-BRBFs, the design was governed by strength. The BRB steel core is assumed to carry the entire axial load developed in the brace due to seismic action. The required cross-sectional area of the steel core was calculated based on the limit states of tension and compression yielding according to the AISC Seismic Provisions[11]. The BRB design axial strength (LRFD), ϕP_{ysc}, is given by Equation (1) as follows:

$$\phi P_{ysc} = \phi F_{ysc} A_{sc} \tag{1}$$

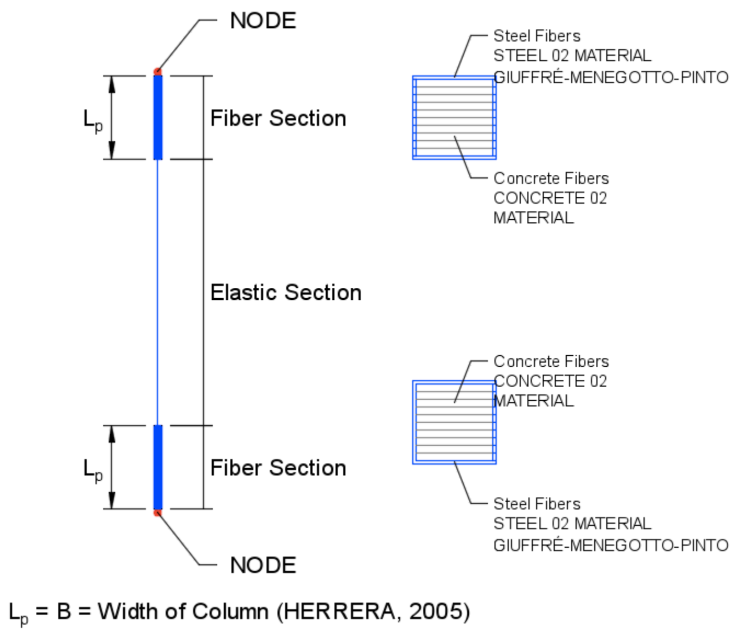
where ϕ is the strength reduction factor, equal to 0.9; F_{ysc} is the yield stress of the steel core; and A_{sc} is the cross-sectional of the steel core. The outer steel tube was sized using the criteria proposed by Watanabe et al. [16]. This criterion establishes that the Euler buckling load of the tube must exceeds the yielding load of the steel core, $F_{ysc}A_{sc}$, to avoid global instability of the brace. A safety factor between 1.5 and 2 may be used. The surrounding frame members (i.e., beams, columns and connections) were designed using the capacity design principles. This procedure considers the maximum force that could be developed in the brace after strain hardening. A standard bolted connection type was used in the CFT-BRBFs. The BRB connections were design based on the procedure developed by Christopulos [17]. In the CFT-BRBFs, the beam-to-column connections were assumed to be fully restrained as those used in the CFT-MRFs.

4. Nonlinear Analytical Model of the Prototype Frames

Two linear analytical models have been developed for the prototype building to investigate the seismic performance with the aid of OpenSees Software [18]. The first model consists of conventional frames in the X-direction (e.g., CMRF), while the second model has external frames in the Y-direction (e.g., BRBF). Figure 4 shows a schematic view of the analytical model for the structure with frames in the X-direction (CMRF). In this model, the CFT columns were modeled using beam-column elements with fiber sections at their ends in a length equal to the cross section dimension of the columns [4]; the remainder part of the column was modeled with beam-column elements with elastic sections. The beams were modeled with beam-column elements with elastic sections, and rotational springs at the beam-end joints. These springs take into consideration the reduction of resistance and stiffness at the beam-column connection according to the model developed by Lignos & Krawinkler [19]. The stiffness of the panel zone was modeled following the recommendations of Fukumoto & Morita [20]. Figure 5 shows the schematic view of the analytical model for the conventional frame in the Y-direction (BRBF). This model is the same as the CMRF model with the addition of buckling restrained bracing frames. The BRBs were modeled as uniaxial elements (truss elements), in the segment corresponding to the yield zone, with a calibrated hysteretic response recommended by Santelices [21]. In this model, the connection zones were modeled with beam-column elements with elastic behavior.



a) Elements b) Beam-Column Joint



c) Column

Fig. 4 – Open Sees nonlinear analytical model for the CMRF

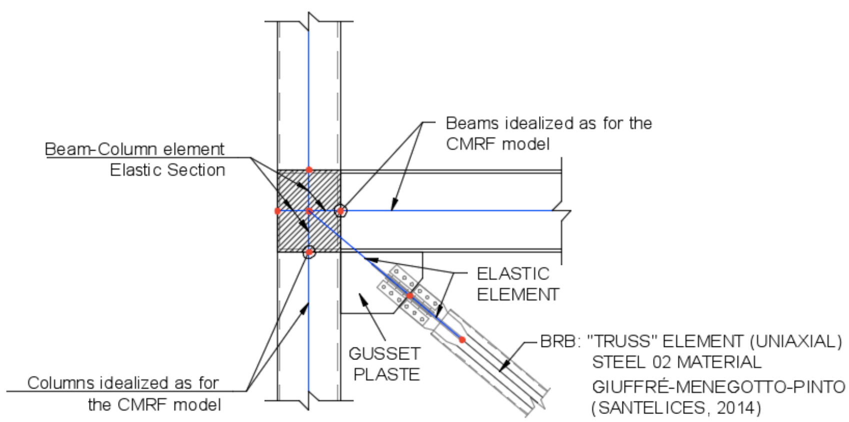


Fig. 5 – OpenSees nonlinear analytical model for the BRBF

5. Nonlinear Analyses of the Prototype Frames

5.1 Pushover

A nonlinear static pushover analysis for the two models was conducted. The lateral loads were distributed over the height of the frame in accordance with the ASCE/SEI 7-10 requirements [10]. Figure 6 shows the relationship for both models between the normalized base shear (base shear divided by the seismic weight of the building, W=7280 Ton) and the roof drift index (maximum displacement at the top floor divided by the total height of the building, θ_{Total}). The analysis results for the CMRF show that first yielding occurs when the applied base shear is 0.12W (θ_{Total} = 1.10%). The maximum overstrength Ω_o, of the CMRF is 4 (0.172W) at a corresponding θ_{Total} of 2.75%. The overstrength Ω_o, is defined as the base shear divided by the ASCE/SEI 7-10 design base shear [10]. For the BRBF model, Figure 7 shows the results for the non-linear static analysis. It is demonstrated that first yielding occurs when the applied base shear is 0.11W for a θ_{Total} of 1.25%. The maximum overstrength Ω_o, of the BRBF is 4.32 (0.19W) at a corresponding θ_{Total} of 3.5%.

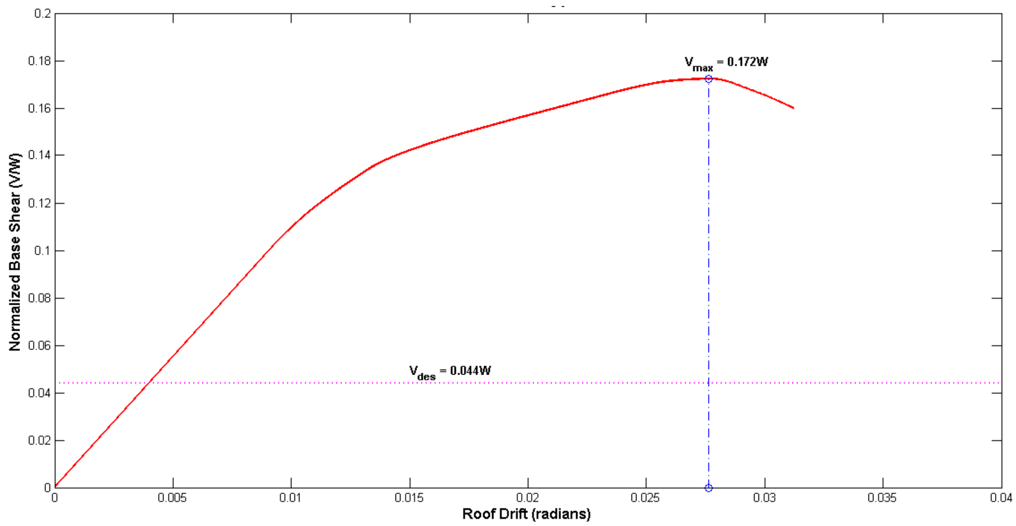


Fig. 6 – Results of the Non Linear Static Analysis (Pushover) – X - Direction (CMRF)

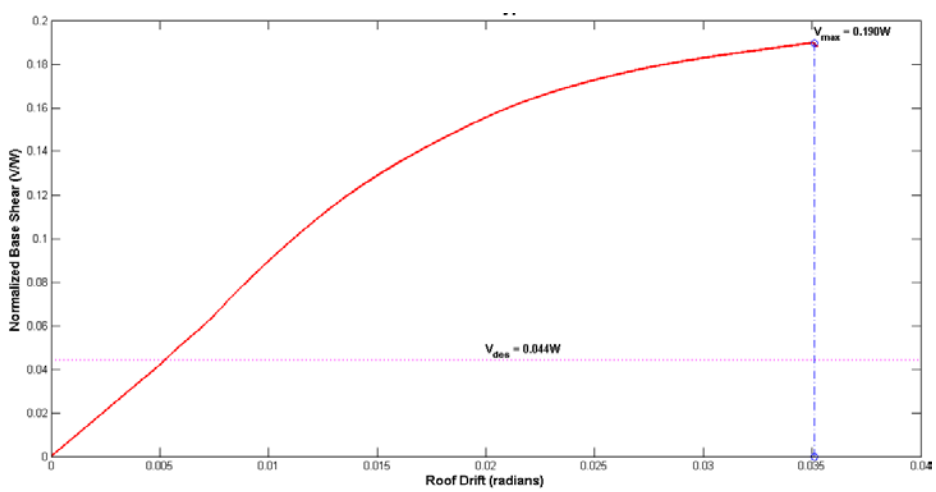


Fig. 7 – Results of the Non Linear Static Analysis (Pushover) – Y - Direction (BRBF)

5.2 Dynamic analyses

Three records, scaled to the DBE and MCE levels [22], were used to investigate the seismic behavior of the CMRF and the BRBF. Figure 8 shows the elastic response spectrum of these records. Figures 9 and 10 show the maximum roof displacement for each record scaled to DBE and MCE levels for CFT and BRBFs, respectively. Table 3 and Table 4 present the results of dynamic analysis of both systems (PMRC and PAPR) concerning the roof drift and story drift, for the different accelerations records scaled to the DBE and MCE levels. According to these tables, as expected the BRBF system presents lower roof drifts and story drifts than the CMRF system.

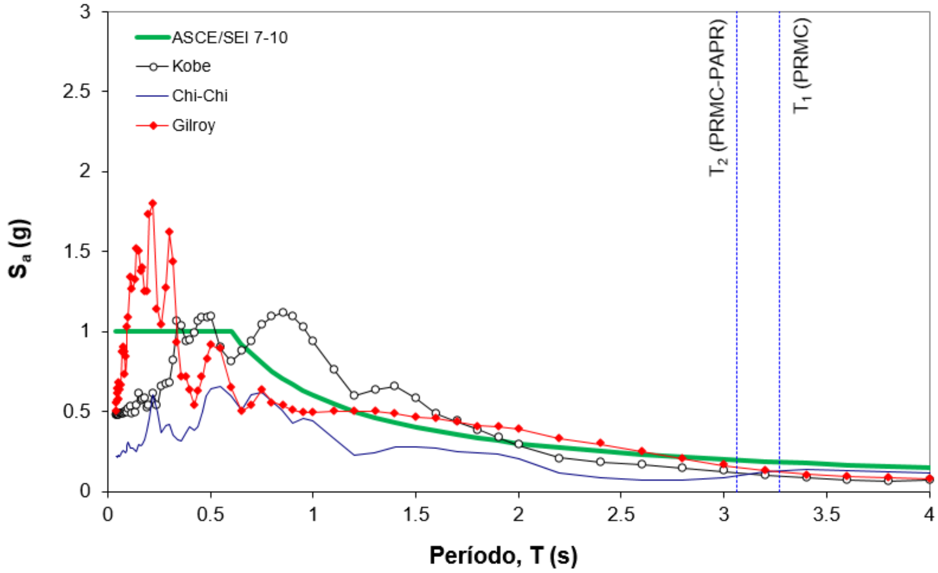


Fig. 8 – Response Spectra – DBE Level

Table 3 – Results of the Dynamic Nonlinear Analyses – Roof Drift

Rooft Drift (%)	**CMRF**		**BRBF**	
	Level DBE	**Level MCE**	**Level DBE**	**Level MCE**
Kobe	1.03	1.10	0.61	0.82
Chi-Chi	1.33	1.71	0.31	0.46
Gilroy	1.02	1.41	0.79	0.95

Table 4 – Results of the Dynamic Nonlinear Analyses – Interstory Drift

Interstory Drift (%)	**CMRF**		**BRBF**	
	Level DBE	**Level MCE**	**Level DBE**	**Level MCE**
Kobe	2.34	2.89	1.28	1.78
Chi-Chi	2.03	2.96	0.60	0.86
Gilroy	2.49	3.80	1.15	1.50

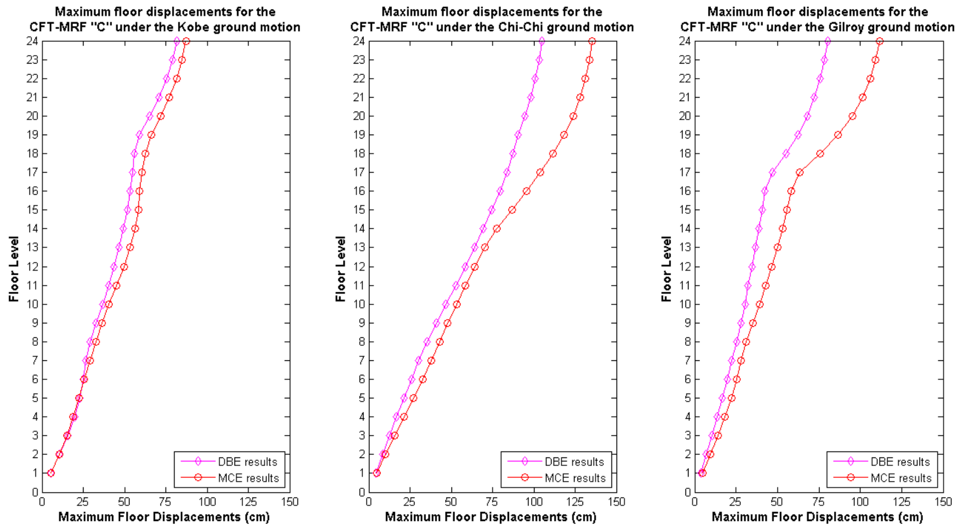


Fig. 9 – Maximum floor displacements for the CMFR under Kobe, Chi-Chi and Gilroy ground motions

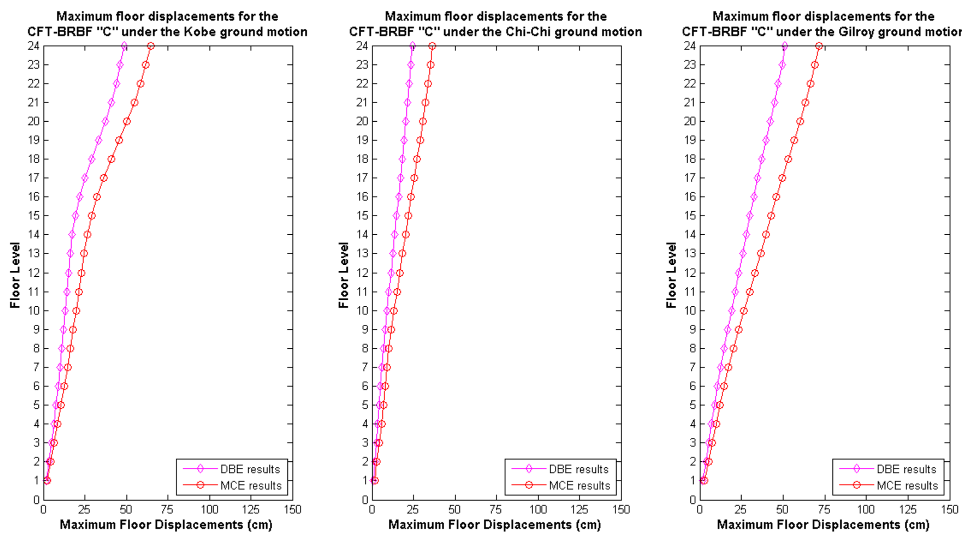


Fig. 10 – Maximum floor displacements for the BRBF under Kobe, Chi-Chi and Gilroy ground motions

6. Summary and Conclusions

This study seeks to provide the Ecuadorian steel industry with other alternatives for structural systems to mid-rise and high-rise buildings different from those of conventional steel and reinforced concrete construction. For this purpose, a 24-story prototype building, where the Lateral Load Resisting System was comprised of CFT-MRFs and CFT-BRBFs, was analyzed and designed using an elastic modal spectral response analysis following the guidelines included in ASCE / SEI 7-10 provisions and recommendations from related research. The results of the three-dimensional SAP 2000 model of the prototype building show that the structure satisfies the strength and drift requirements included in American codes.

The seismic performance of the prototype designed building was evaluated by nonlinear analysis of two types: (1) lateral static (pushover); and (2) dynamic (time-history). For the nonlinear analysis, analytical models of the systems used in this study were developed (CMRF and BRBF) with the help of OpenSees software. In this nonlinear dynamic analysis, three registers scaled to seismic accelerations levels DBE and MCE were used. The results of the seismic evaluation of the prototype building indicate that both systems have a satisfactory seismic performance regarding strength, ductility, and energy dissipation. Consequently, these systems offer an attractive alternative for the construction of mid-rise and high-rise buildings in Ecuador. Finally, it is recommended to carry out additional nonlinear analyses using local ground motions to evaluate the seismic performance of the prototype building. Future work shall also include the development of construction details for moment beam-column connections and buckling restrained bracing using local technology and materials.

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