Seismic Analysis and Design of an Industrial Complex of Buildings Located in Ecuador

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Abstract. This paper presents the seismic analysis and design of an industrial complex of buildings located near the city of Guayaquil-Ecuador. The industrial complex is composed of 4 structural steel buildings with a height of 48 meters. Since the buildings are located in a high seismic region, a detailed and careful study was performed in order to select the most suitable structural systems to withstand the design earthquake forces. Moment Resisting Frames having Concrete-Filled Tube columns (CFT-MRFs) and Concentrically Braced Frames (CBFs) were the structural systems used for the seismic load resistant of the buildings. Each of the buildings has irregularities in elevation. This paper provides a review of the most relevant aspects related to the analysis and design of CFT-MRF and CBF systems for industrial buildings. A design procedure was developed following the Ecuadorian and American Codes seismic requirements. One of the most important criteria evaluated for the seismic analysis and design was the building separation to minimize the possibility of pounding of the buildings. Special attention was given to the connection detailing since the owner required the use of field bolted connections instead of field welded connections. Finally, some constructive issues that emerged during the execution of the construction of the project are summarized.

Keywords: seismic analysis and design; concentrically braced frames; moment resisting frames; concrete-filled tube columns; industrial buildings.

1 Introduction

This paper presents the seismic analysis and design of an industrial complex of four buildings located near the city of Guayaquil-Ecuador. The industrial complex is engaged in the production of balanced feed for canine, equine, livestock, and aquaculture species. The material used for the superstructure of the buildings is structural steel. The industrial complex is located in a high seismic region and characterized by soft clays soils with thicknesses greater than or equal to 3.0 m. The first part of the paper describes the structural systems selected to withstand the design earthquake forces and the approach followed for the analysis and design of the buildings. Ecuadorian and American seismic codes were used for the analysis and design of each of the buildings. The second part of the paper presents the design of the structural systems and their components including the connections. Finally, some constructive issues that emerged during the execution of the construction of the project are summarized.

2 General and Structural Description of the Industrial Complex

2.1 General description

The industrial complex located near the city of Guayaquil-Ecuador, consists of 4 structural steel buildings with height of 48 meters. Fig. 1 shows the L-shaped schematic plan view of the industrial complex. Typical floor areas are 455.40 m^2 for wing 1, 871.50 m^2 for wing 2, 1677.40 m^2 for wing 3, and 1608.30 m^2 for wing 4. Each building is part of the same line or system that produces animal food for canine, equine, livestock, and aquaculture species. Each building has heavy equipment systems for the production, such as silos with capacities ranging from 45 m^3 to 180 m^3 and heights larger than 12 meters, extending over several floors, which are located especially on intermediate floors; belt conveyors that move the product from one building to another, extruder equipment, and heavy equipment that generates high vibrations. Also, each building has bridge cranes to move bulk bagged products, water tanks, etc.

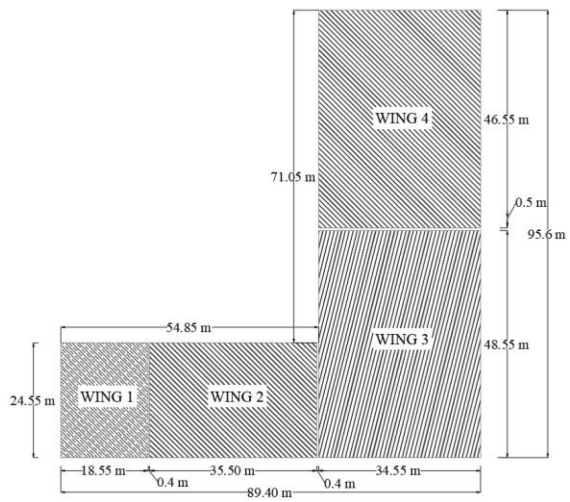


Fig. 1. Plan view of the industrial complex

2.2 Structural description

The substructure of the buildings consists of a deep foundation with prestressed concrete piles due to the fact that the industrial complex is located in an area where soil conditions are not suitable for a shallow foundation system. Furthermore, the vertical and lateral loads imposed on the foundation are significantly large. The piles are arranged in groups under each column. The group is capped by a reinforced concrete cap pile that distributes the column loads to all piles in the group. The pile caps are connected by grade beams in both directions.

The superstructure of the buildings consists of a dual system of steel frames. The dual system includes Moment Resisting Frames having Concrete-Filled Tube columns (CFT-MRFs) capable of resisting at least 25% of prescribed seismic forces and Concentrically Braced Frames (CBFs) in both the X and Y directions. The CBFs are located on the exterior frames. Contrary to the US construction practice, all frames are assumed to resist both gravity load and seismic load effects.

For the filling of the columns, normal weight concrete with a compressive strength of f'_c=28 MPa (nominal compressive strength) at 28 days was used. The design was made with hot-rolled sections (W-Shaped) for main beams, secondary beams, and bracings that correspond to the profiles of the standard of the American Institute of Steel Construction (AISC). ASTM A572 Gr.50 steel (350 MPa nominal yield strength) was used for the structural elements of the building such as columns (welded built-up square section), main beams of the MRFs and beams of the CBFs. However, ASTM A36 steel (250 MPa nominal yield strength) was used for other structural elements such as secondary beams, main roof beams and bracings. As required by the client, bolted connections were considered for beam-column and beam-column-bracing connections. The structural bolts used were ASTM A490 and ASTM A325.

The buildings have three 4.00 meters and 5.00 meters bays plus ten 6.00 meters bays in the X-direction, and one 6.00 meters bay and eleven 8.00 meters bays in the Y-direction. A typical floor plan of the four buildings is shown in Figure 2. The buildings have vertical irregularities as they have a wide variety of mechanical equipment on each of their floors. The story heights vary from 2.6 to 7 meters, resulting in a total height of 48.00 meters. An elevation structural view of typical frames in the X-direction is given in Figure 3.

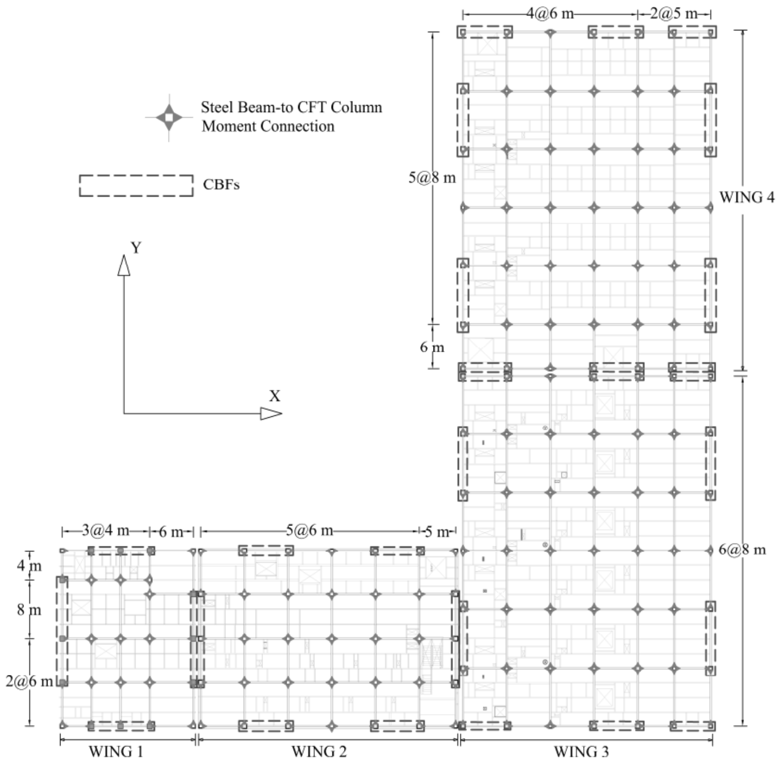


Fig. 2. Plan structural layout of the industrial complex

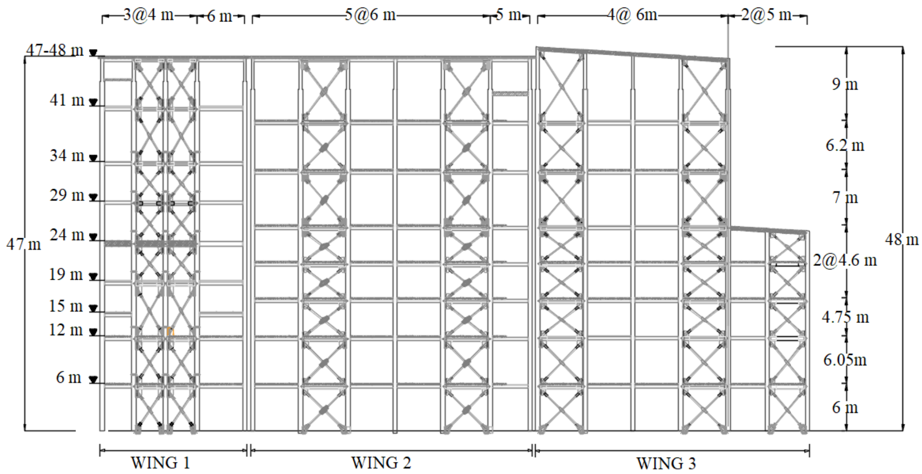


Fig. 3. Elevation structural view of the industrial complex

3 Seismic Design Approach

3.1 Standards and Codes used

The standards and codes used in the design of the superstructure of the buildings were the ASCE/SEI 7-16 Standard [1], the Ecuadorian seismic design code [2], and the seismic provisions specified in [3] and [4].

3.2 Gravity Loads

Since industrial buildings of the type presented in this paper have large gravity loads, a detailed computation of the gravity loads was carried out in every single floor. Therefore, the gravity loads considered in the structural analysis and the design of the buildings were the following: the self-weight of the structure D; the superimposed dead Load (S_{D1} and S_{D2} for all floors and for the roof floor, respectively); the Live Load L_L; the weight of the Product (balanced feed) P_L and the self-weight of equipment W_{Mech}. The superimposed dead and the live loads were applied only where there is no equipment according to the mechanical drawings.

3.3 Seismic Loads

Considering that the industrial plant is located in a high seismic risk zone and characterized by soft clays soils with thicknesses greater than or equal to 3.0 m, the appendix 10.5 of the Ecuadorian seismic code [2] specifies that this type of soil shall be classified as type E. The modal response spectrum analysis (MRSA) described by ASCE/SEI 7-16 [1] was used to estimate the earthquake load effects in the structural elements of each building. The seismic hazard level used was an earthquake having a

10% probability of being exceeded in 50 years with a seismic importance factor of 1.0.

The effective seismic weight for the structural analysis of foundations and superstructure was taken as: W_{sm} = 1.00*D + 1.00*S_D + 1.00*W_{Mech} + 0.50*P_L. These values were estimated at 3919 tons for wing 1, 4503 tons for wing 2, 6414 tons for wing 3, and 6364 tons for wing 4. The response modification coefficient R used for all the buildings was 6 according to [1]. In addition, the seismic forces were increased by 11% to consider the building vertical irregularities. A plot of the seismic response coefficient, C_s, versus the building period, T, is given in Figure 4.

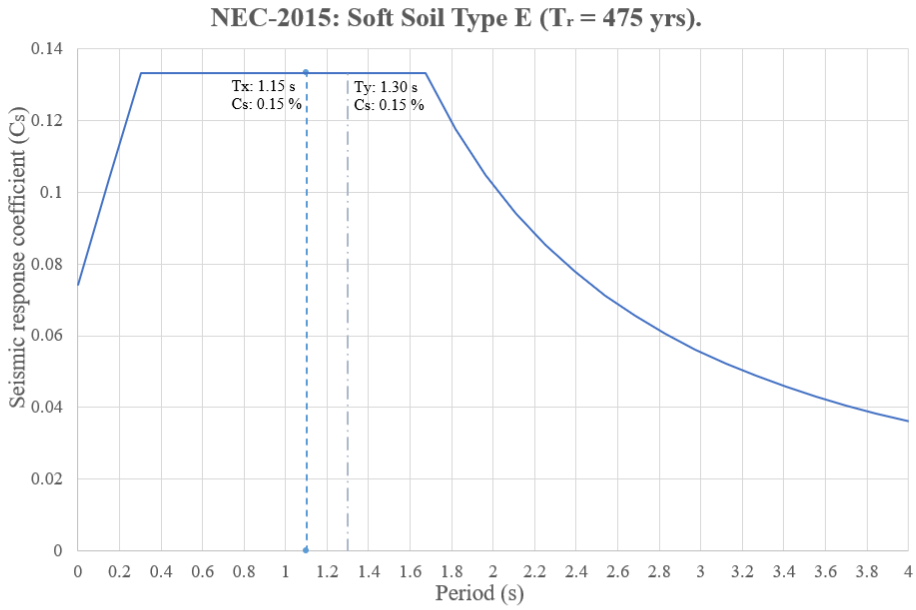


Fig. 4. Seismic response coefficient vs building period

3.4 Load combinations

The demand on the structural elements was determined from the following six load combinations based on the Ecuadorian seismic design code [2] and ASCE/SEI 7-16 Standard [1].

$$1.4(D+S_{D1+}\ S_{D2+}\ W_{Mech}) \quad (1)$$
$$1.2(D+S_{D1+}\ S_{D2+}\ W_{Mech})+1.6(L_{L+}P_L)+0.5L_r \quad (2)$$
$$0.7(D+S_{D1+}\ S_{D2+}\ W_{Mech})+E_x+0.3E_y \quad (3)$$
$$0.7(D+S_{D1+}\ S_{D2+}\ W_{Mech})+E_y+0.3E_x \quad (4)$$
$$1.4(D+S_{D1+}\ S_{D2+}\ W_{Mech})+0.5(L_{L+}P_{L+}L_r)+E_y+0.3E_x \quad (5)$$
$$1.4(D+S_{D1+}\ S_{D2+}\ W_{Mech})+0.5(L_{L+}P_{L+}L_r)+E_x+0.3E_y \quad (6)$$

4 Design of the Structural Systems

4.1 Three-Dimensional Analytical Model

A three-dimensional (3D) linear elastic analysis of the structures was carried out using the SAP2000 software [5] in order to estimate the member design forces and to determine the structure displacements under the different load combinations presented in Section 3.4. 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 were taken into account [6]. The typical vibration period of the buildings was 1.15 seconds for the X-direction and 1.30 seconds for the Y-direction. Fig. 5 presents views of the SAP 2000 three-dimensional analytical models for all the building wings.

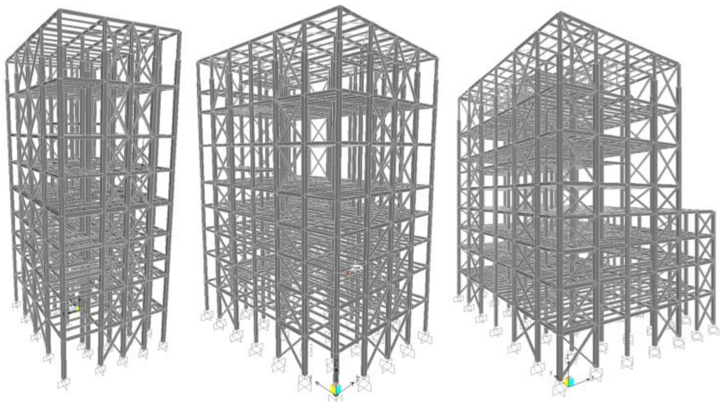
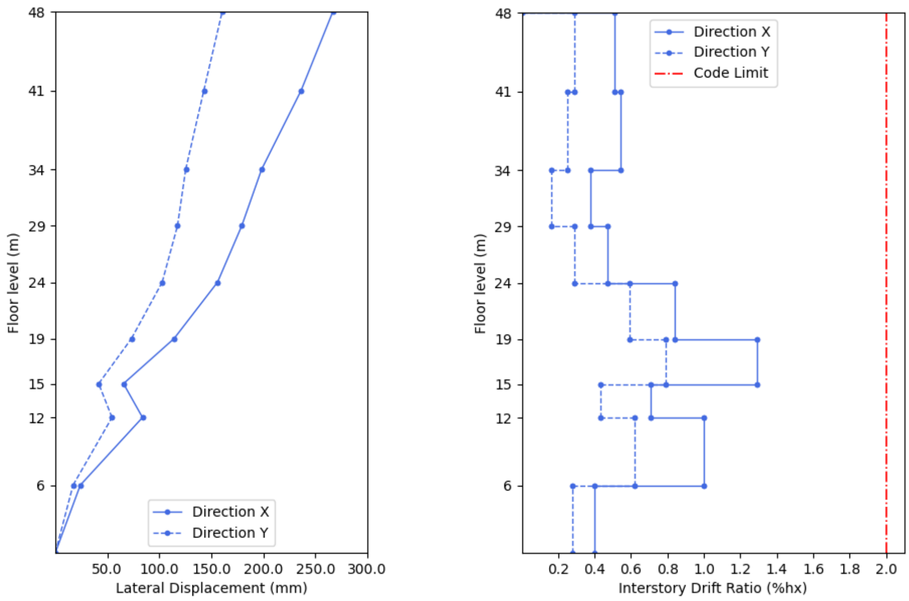


Fig. 5. View of the SAP2000 three-dimensional analytical models (left-to-right; wing 1, 2 and 3-4 respectively).

4.2 Concrete-Filled Tube columns Moment Resisting Frames (CFT-MRFs) and Concentrically Braced Frames (CBFs)

Story drift requirements controlled the selection of the structural member sizes. One of the most important criteria evaluated for the seismic analysis and design was the building separation to minimize the possibility of pounding of the buildings. Seismic gaps of 500 and 400 mm were provided between the buildings as can be seen in Fig. 1a. In addition, the members of the CFT-MRFs were proportioned to satisfy a strong column-weak beam design, according to the AISC Seismic Provisions [3]. The column-to-beam moment capacity ratio is established as 1.0 in the AISC Seismic Provisions [3]. The CFT columns and beams were designed according to the AISC Specifications [7] and the AISC Seismic Provisions [3]. 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 [7] and compared with the 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 AISC Seismic Provisions.

Figure 6 shows the amplified elastic displacements and the interstory drift ratio distribution for Wing 1. The amplified elastic roof displacements were 267 mm (0.0057 times the total building height) and 160 mm (0.00341 times the total building height) for the X-direction and the Y-direction analysis, respectively, as seen in Fig. 6a. The maximum interstory drift ratios were 1.29% and 0.79% for the X-direction and Y-direction, respectively. As seen in Fig. 6b, the maximum interstory drifts are well below the allowable interstory drift (0.02 radians) specified in the Ecuadorian seismic design code [2].



a) Amplified elastic displacements b) Interstory drift ratios

Fig. 6. Amplified elastic displacements and interstory drift ratio vs Floor Level for Wing 1.

5 Connection Design

Special attention was given to the connection detailing since the owner required the use of field bolted connections instead of field welded connections. Figure 7 shows side views of typical exterior connections of a MRF beam-column connection and a CBF diagonal brace connection used for the buildings of the industrial complex.

5.1 Bolted Flange Plate Moment Connection

A bolted flange plate (BFP) moment connection with interior diaphragms (continuity plates) was used in this project. The BFP moment connections utilize shop plates welded to concrete filled columns and bolted to the beam flanges at the construction site. The top and bottoms plates must be identical. Flange plates are welded to the columns using complete joint penetration (CJP) grove welds and beam flange connections are made high-strength bolts. The beam web is connected to the columns using a bolted shear tab with bolts in short-slotted holes. Details of this connection are presented in Fig. 7a. Initial yielding and plastic hinge formation are intended to occur in the beam in the region near the end of the flange plates. The BFP moment connections were designed following the procedure described in AISC 358-16 [4]. The following criteria were considered in the design process.

The probable maximum moment at the plastic hinge, M_{pr}, and the required shear strength, V_u, of the beam were computed from Eqs. (7) and (8).

$$M_{pr} = C_{pr} R_y F_y Z_e \tag{7}$$

$$V_u = 2M_{pr}/L_h + V_{gravity} \tag{8}$$

where C_{pr} is a factor that account for peak connection strength, R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y, Z_e is the effective plastic section modulus at the location of the plastic hinge, L_h is the distance between plastic hinge locations, and $V_{gravity}$ is the beam shear force. Values of 1.2 and 1.1 were taken for C_{pr} and R_y as specified in references [4] and [3], respectively.

Bolt size is chosen to prevent the beam flange tensile rupture limit state. The number of bolts is estimated considering the BFP moment connection and then to establish the length of the connection. This length is further used to estimate the moment expected at the face of the column using Eq. (9).

$$M_f = M_{pr} + V_h S_h \tag{9}$$

where V_h is the larger of the two values of shear force at the beam hing location at each end of the beam and S_h is the beam plastic hinge location.

The flange plate strength shall exceed the acting force, F_{pr}, for the limit states of tensile rupture, block shear rupture, and compression buckling. The need for continuity plates in the columns was also reviewed, in accordance with AISC 358-16 [4], although they were not necessary in any case. The capacity of the panel zones were compared with the required shear strength following the requirements of AISC 358-16 [4].

The shear connection consists of a single vertical plate welded to the column and bolted to the beam web as seen in Fig. 7a. The limit states of creep and shear fracture in the connection plate, shear resistance in the bolts, crushing in the shear plate and crushing in the web of the beam were reviewed.

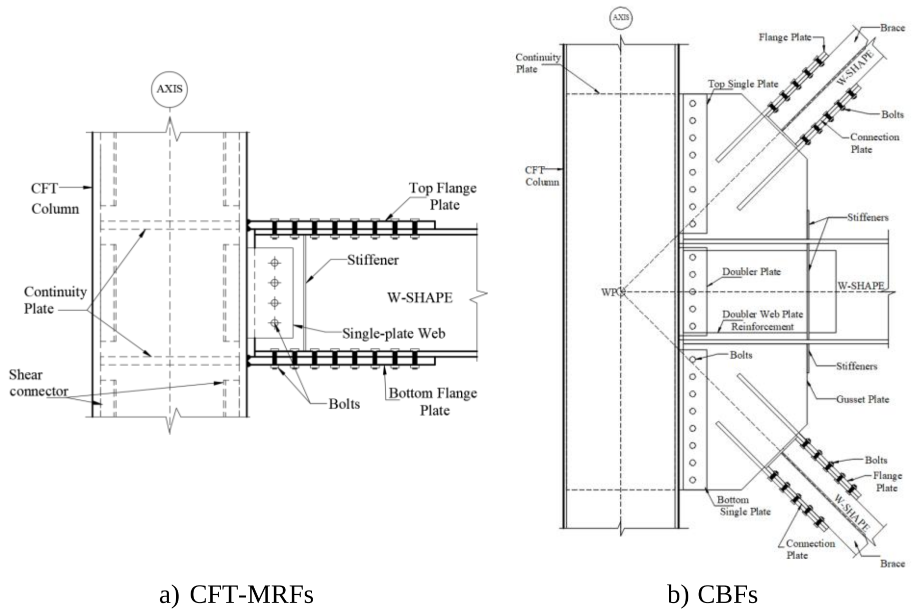


Fig. 7. Typical connections for the CFT-MRFs and the CBFs

5.2 Diagonal Brace Connection

W shapes were used for the diagonal braces of the CBFs. As seen in Fig. 7b, bolted connections were used to connect the diagonal brace flanges with flange plates welded to the gusset plates. The gusset plates are bolted to vertical single plates and shop welded to the W beams. An interior vertical continuity plate is welded to the CFT columns.

According to the requirements of section F2.3 of AISC 341-16 [3], the seismic demand for the connections was estimated according to capacity criteria, in terms of the maximum probable tensile strength of the diagonal brace. The expected post-buckling brace strength was taken with a maximum contribution of 30% of the expected brace strength in compression. The resistance for each of the limit states was checked according to AISC 360-16 Specifications [7]. The sizes of the plates and bolts were designed so that a limit state other than tensile yielding of the diagonals does not govern the tensile strength of the system in any case.

6 Construction issues

Some difficulties arose during manufacturing and field erection of steel components of the main structure. The most important construction issues are summarized next.

Inaccessibility for welding one side of the continuity plate. According to the design, the continuity plates are attached to the column with (CJP) groove welds all around the column section. Due to current local construction technology in Ecuador and the column type used (built-up box columns according to AISC 358-16 [4]) only three sides of the plates were welded to the walls of the HSS.

Column-Base Plate connection. The Complete Joint Penetration (CJP) groove weld designed for this connection demanded more time for its execution. In addition, the uncomfortable welding position and the limited visibility of the joint demanded highly qualified welders. The welding process of this connection, due to the construction methodology used, was made in-situ after casting the slab with a perimeter narrow gap around the columns without casting for this purpose. Moreover, the limited space between the slab edge and the joint and the proximity of the anchor rods to the built-up box column made the cleaning and grinding processes difficult.

Distortions in built-up box columns. Deformations in the columns induced by heat from welding during manufacturing add deviations in the straightness of the columns. This issue affects the verticality of the element and related connections (bolted or welded joints). The sum of deviations was evident as elements were assembled at higher levels of the building. These deviations were corrected in the welds, between column segments, causing excessive root separations or misalignments that exceed those allowed within the prequalified joints according to the AWS D1.1 structural welding code [8]. These out-of-tolerance deviations were solved, as possible, by developing welding procedures that guarantee both the sanity and mechanical properties of the joints.

Lack of control in welding sequence of column splices. Welding in specific sequences was used in built-up column splices (field connections) and base plate – column joints in order to control angular distortion in the multi-pass butt welds. The lack of control of this technique introduced non-recoverable deviations in the elements.

Hole mismatch in bolted connections. For these cases, the solutions was to enlarge the holes with magnetic drills. The use of manual oxy-cutting was not allowed due to the possibility of generating notches.

7 Summary and Conclusions

This paper presented the seismic analysis and design of an industrial complex of four buildings. Since the buildings are located in a high seismic region, Moment Resisting Frames having Concrete-Filled Tube columns (CFT-MRFs) and Concentrically Braced Frames (CBFs) were the structural systems selected for the seismic load resistant of the buildings. A design procedure was developed following the Ecuadorian and American Codes seismic requirements. One of the most important criteria evaluated for the seismic analysis and design was the building separation to minimize the possibility of pounding of the buildings. The results of the three-dimensional finite element model of the prototype building show that the building structures satisfy the strength and drift requirements included in the codes. Future work shall address the evaluation of the seismic performance of the buildings.

Acknowledgments

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