An Evaluation of a Simplified Analysis Model for the Earthquake Response of a Coal Fired Boiler and its Steel Support Structure

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ABSTRACT :

The seismic behavior of a heavy industrial boiler system built in Chile is studied considering pushover analysis method and time-history analysis. The aim is to identify under which circumstances the seismic design obtained is satisfactory when the design procedures used normally for this type of structures are applied.

Results of response spectrum analysis (RSA) and pushover analysis (PA) match for small lateral displacements. However, the element forces distributions do not have a good correlation with the prediction made by RSA when lateral displacements are larger and inelastic behavior in the main elements occurs in PA.

The final design of the steel support structure for the boiler shows what can be considered a satisfactory design due to a rather large over-strength level, which allows matching the general criteria of the Chilean design code for industrial buildings to prevent excessive damage and limit shutdown time.

Given the differences observed in the response results between RSA and PA and also the low correlation between usage ratios in element design (based on the RSA results) and the actual ductility ratios obtained in the PA results it becomes clear that it is necessary that the design of this type of structure is based in procedures that can estimate actual behavior in the inelastic range with greater accuracy. This will allow obtaining a more economic and efficient design of elements avoiding the use of large over-strength factors in all structural elements.

KEYWORDS: boiler supporting structure, industrial buildings, non linear analysis, pushover analysis, time-history analysis.

1. INTRODUCTION

The seismic design for industrial buildings is more demanding than seismic design for other buildings. The need for protecting human life is extended to protect also industry life, meaning that essential services must remain functional and the operations shut down time must be reduced to a minimum (INN, 2003)

Seismic behavior of heavy industrial buildings differs considerably from typical building seismic behavior, since industrial buildings are conceived to serve the operational needs of equipment. Every structure is unique in terms of mass and stiffness distribution; hence it is difficult to fit its behavior within a standard. For this reason is that studies using nonlinear techniques such as pushover method or time history analysis have not been widely carried out in this area, differing from building area where an important number of investigations have been performed (Kilar and Fajfar (1997) , Mwafy and Elnashai (2001), Chopra and Goel (2002), Penelis and Kappos (2002)).

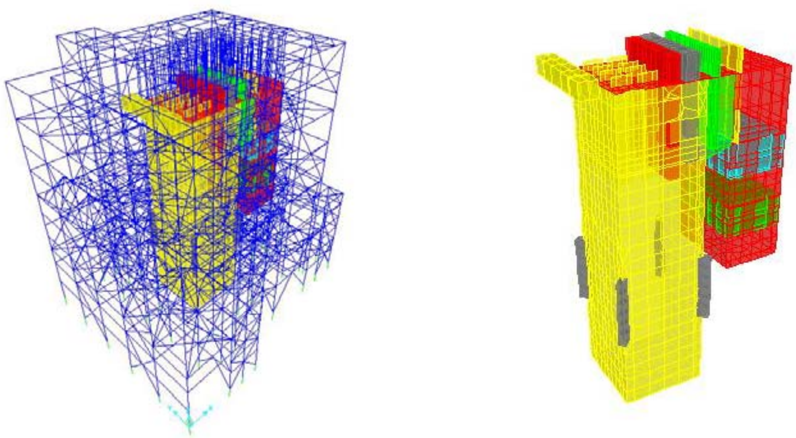
Although normally industrial buildings present considerable over-strength levels in its resisting elements, the design is based in response spectrum analysis (RSA) which considers the structure as having elastic behavior; therefore the element forces distribution loses validity when the structure is in the nonlinear range.

This study intends to evaluate the seismic design of a boiler system in the non-linear range, using pushover analysis method and time-history analysis, which will allow to describe in a more precise manner the structure behavior under large earthquake excitation.

2. BOILER MODEL

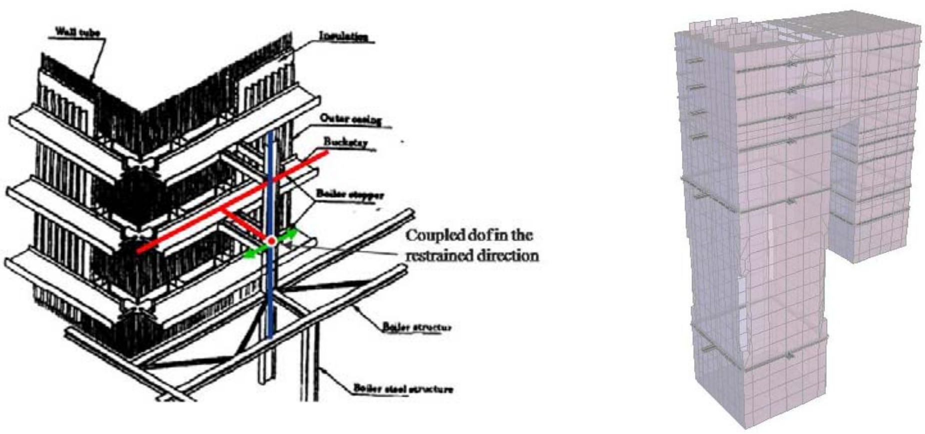
The boiler system considered has a total height of 56.87 m and a total weight of 10.290 tonf. The steel support structure weight is 2264 tonf and the boiler plus equipment weight is 8026 tonf. The boiler internal components include several large mechanical equipments and transfer horizontal seismic forces to the steel support structure through several different elements: backstays, guides, and stoppers.

The support structure is a braced frame system in two perpendicular directions: 13 frames in X direction (longitudinal or along flue gas flow) and 10 frames in Y direction. Connections between beams and columns are shear type, and there are 7 horizontal platforms at different levels over the height also braced. Reinforced concrete foundation is a 2 m thick slab, with a plan of 56 x 35 m where 54 pedestals with 2.25 m height support the steel columns. Total foundation weight is 10200 tonf. In the modeling, foundation is assumed to be rigid. Structure has 5 coal silos with 523 tonf weight each.



(a) (b)

Figure 1 - Structure model: (a) complete model; and (b) boiler representation



(a) (b)

Figure 2 – Backstays and stoppers: (a) details; and (b) location in model (darker elements)

A first simplified model of the internal components considers them as a rigid body with their correct geometry and mass distribution, hanging from the vertical support points and attached to the steel support structure through the “stoppers”, beam type elements that restrain displacement in only one direction.

For this model, the steel support structure is considered using standard modeling assumptions for flexure and compression elements (beam-column type elements). From modal analysis using Ritz vectors, the most relevant natural vibration modes are obtained to describe the dynamic behavior properties of the model (Table 1).

Table 1 – Periods and modal effective mass ratios for simplified model with "rigid internal components"

Mode	Period (s)	UX	UY	UZ	RZ
5	1.167	67%	0%	0%	24%
6	0.976	0%	82%	0%	28%
21	0.617	4%	0%	23%	1%
73	0.255	0%	0%	34%	0%

There is one mode in each direction that concentrates a large fraction of the total effective mass, and the mode shape corresponds to lateral displacement of the entire structure.

3. PUSHOVER ANALYSIS IMPLEMENTATION

The standard "Pushover Analysis" procedure is applied. For lateral load distribution, three cases are considered: (a) Constant distribution in height, (b) Linear distribution in height, (c) Predominant mode distribution (largest effective mass ratio mode). As initial load condition, all dead loads and 25% of live loads are applied. For displacement control, a node at the top platform surrounded by bracing elements is select to avoid local in plan deformation effects. Final displacement was set arbitrarily to 100 cm.

Four types of hinges are considered for modeling nonlinear behavior: (a) axial force hinges for vertical and horizontal bracing, (b) moment-axial force interaction hinge for columns, (c) Moment hinges for stopper elements, and (d) Axial load hinge for anchoring system. As hinge unloading method, the "Restart using secant stiffness" method as implemented in the software SAP2000 is used (CSI, 2007). Effective yield tension is used for steel, according with the AISC 2005 specifications (AISC, 2005).

4. RESULTS FOR PUSHOVER ANALYSIS

A design spectrum with a maximum acceleration of 1.375 (g) and a reduction factor R=2.5 was considered for the steel support structure element design. For evaluating the behavior in the inelastic range, amplification factors were applied to the R=1 spectrum, defining 3 cases: Level I (R=1), Level II (R=1/3), Level III (R=1/6).

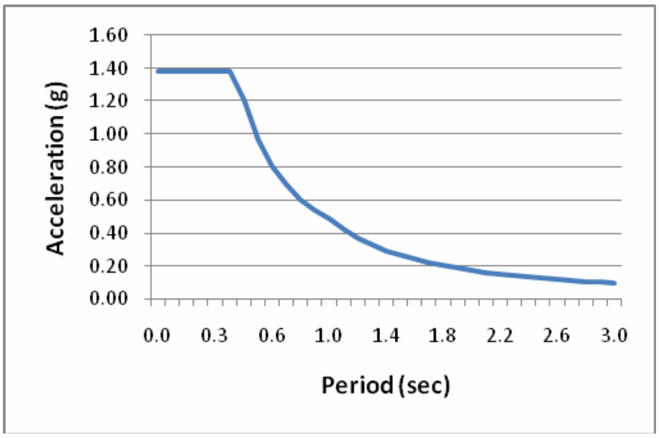


Figure 3 – Pseudo—Acceleration Design spectrum for R=1

For each lateral load distribution, pushover analysis is performed considering 4 cases: X+,X-,Y+,Y-, given that the structure is not symmetric in plan different results are expected when loading acts indifferent directions.

Figure 4 shows the results of base shear vs. roof displacement for several lateral force distributions acting in X direction. The drops in base shear are explained because of “breaking” of a stopper element, that is the element reaches its maximum ductility ratio and then looses all its load carrying capacity. The lateral force distribution corresponding to the “Predominant mode” does not reach 100 cm. roof displacement, because before that all stoppers reach their “breaking” ductility limit. When this occurs, the internal components become disconnected from the support structure and a “pendulum type” mechanism is obtained.

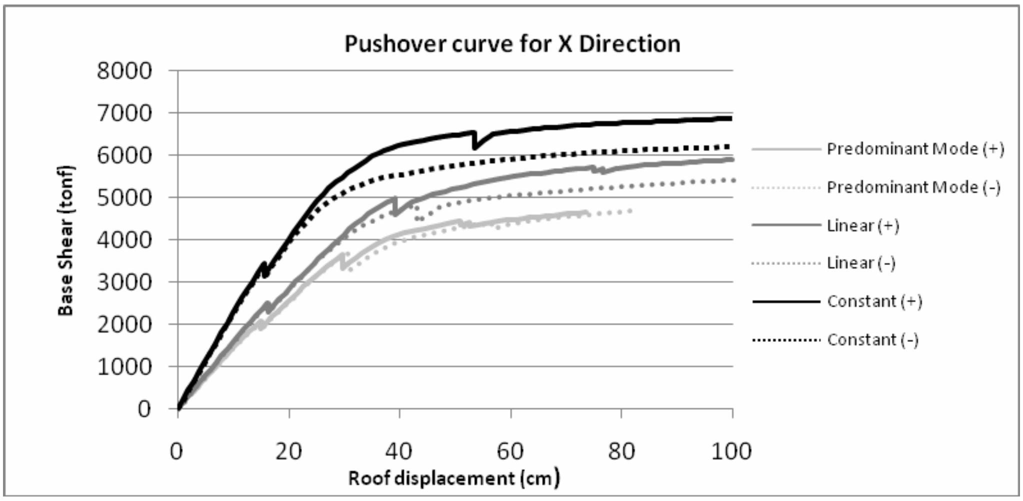


Figure 4 – Comparison of base shear vs. roof displacement for different lateral force distributions.

Figure 5 shows the drifts over the height for the different platform levels. For Earthquake Level I and II the values do not exceed the 1.5% maximum required by the Chilean codes.

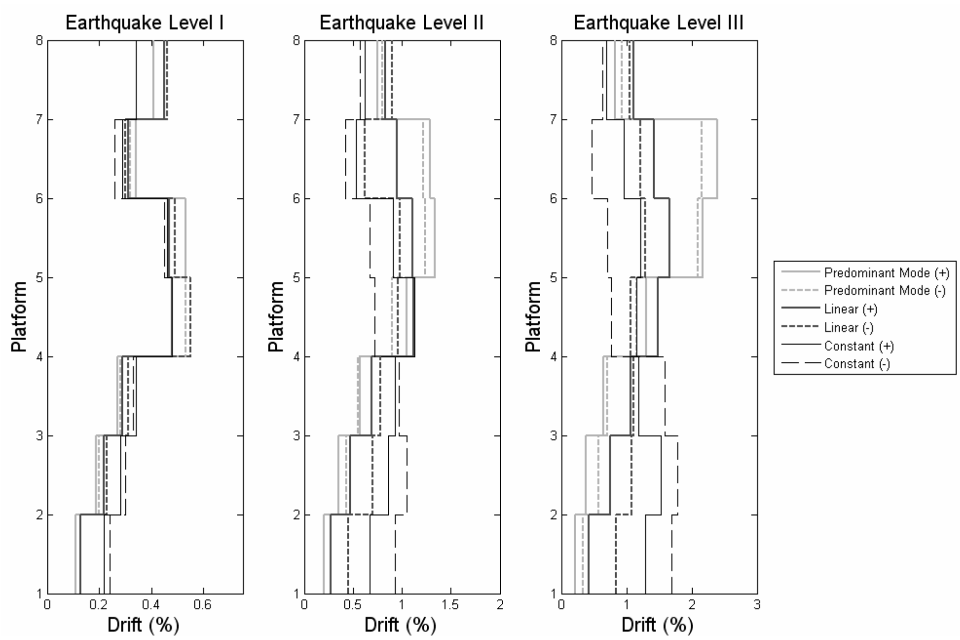


Figure 5 - Drifts over the height

Figure 6 shows a measure of the amount of inelastic behavior observed for each group of elements for lateral forces applied in positive and negative X direction as a function of the top displacement. Vertical braces are seen to concentrate most part of the inelastic behavior, since near 25% of all vertical braces have non linear behavior for roof displacement close to 70cm. Anchoring systems have important yielding behavior in a direction only due to the asymmetry of the structure in the X direction. Horizontal braces and columns have a small number of elements that enter in the nonlinear range.

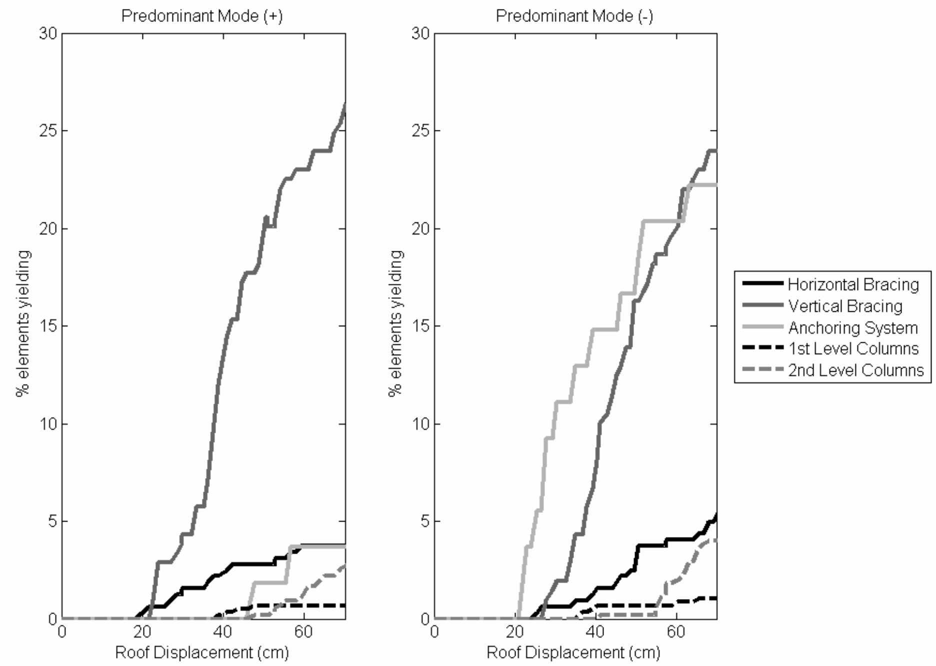


Figure 6 - Inelastic behavior sequence for each group of elements for predominant mode.

Figure 7 shows a comparison between the ductility ratio (maximum inelastic displacement / yield displacement) and the design usage ratio (required strength / provided strength) for vertical bracing elements considering the results of "Predominant Mode" acceleration distribution. There is a trend showing that as usage ratio increases, there are more elements entering into the nonlinear range of behavior; however the results show large dispersion and thus they do not allow claiming with certainty that the design based on response spectrum analysis will have an acceptable behavior in the nonlinear range.

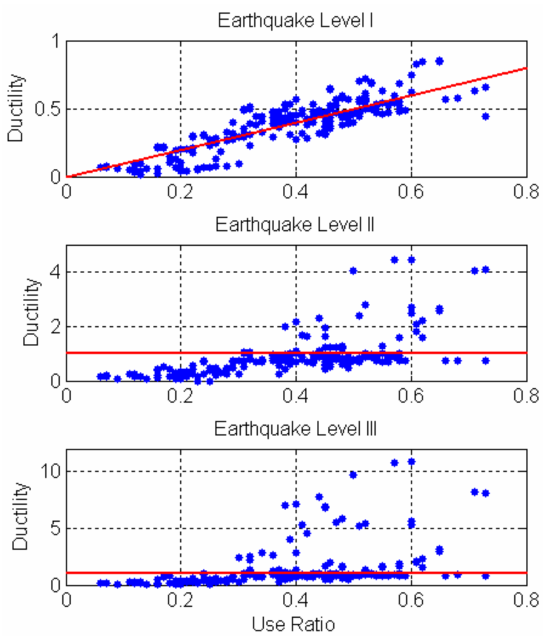
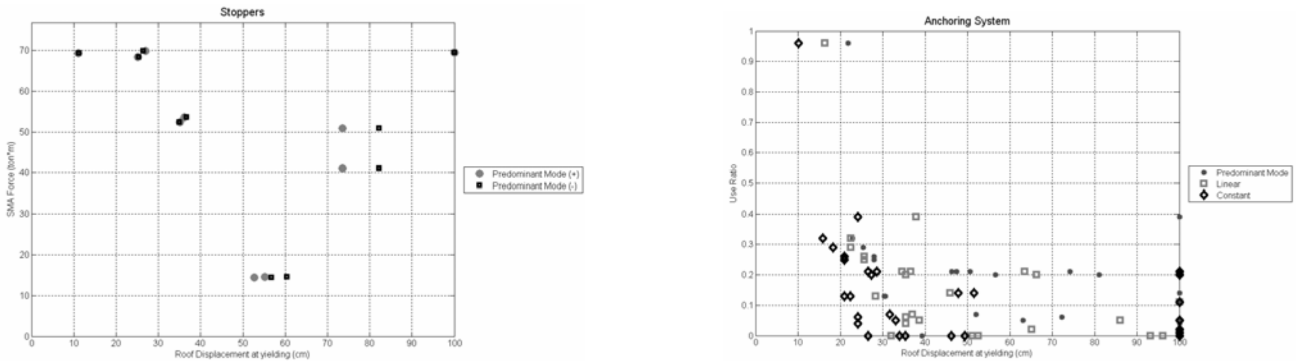


Figure 7 – Comparison between use ratio and ductility obtained by pushover method for vertical bracing for predominant mode acceleration distribution in X direction.

Figure 8 shows the relationship between RSA results and roof displacement at the beginning of yielding for stoppers elements and for anchoring system. Elements that do not have inelastic behavior during PA are shown arbitrarily at 100cm displacement. If there was a linear relation between PA results and RSA results, the points should align along a straight line with negative slope. It is observed that there is no correlation in either case, since elements with the same level of force obtained by RSA, have inelastic behavior for very different displacements for PA. Also, elements that do not have inelastic behavior cover a wide range of forces (RSA results) and they are not restricted to small values of as it could be expected.



(a) (b)

Figure 8: (a) Comparison between stopper forces from RSA and roof displacement at yielding using PA for predominant mode acceleration distribution; (b) comparison between anchoring bolts use ratio for RSA and roof displacement at yielding for three acceleration distributions in PA.

5. TIME HISTORY ANALYSIS

A nonlinear time history analysis (THA) is used with the intent to determine more realistic results of the structure behavior under seismic loads. This analysis uses the same model of the steel support structure as used before for the PA, but it considers more complex force-deformation relations for braces, columns, and stoppers to include effects that are significant under cyclic forces (i.e. stiffness degradation, plastic force redistribution, physically consistent unloading, 3D interaction of forces).

To avoid the huge computational effort required to carrying out the direct integration of the equations of motion (time history analysis) with almost 30000 degrees of freedom, the analysis is made using the Fast Nonlinear Analysis (FNA) procedure (Wilson, 2001). Stiffness and mass orthogonal “Load Dependent Ritz Vectors” are used to reduce the size of the nonlinear system to be solved. The initial loads vectors target both a set of elastic behavior mode shapes that include a large amount of equivalent seismic mass and also a complete projection for the possible deformations of the elements that might go into the nonlinear range of behavior, justifying the use of a reduced amount of “linear modes” in a nonlinear analysis. The elastic modal shapes are obtained directly from SAP2000 and the FNA is implemented using MATLAB (The MathWorks, 2007)

As already shown in the PA carried out, in the preliminary results obtained using THA the stoppers are the most sensitive elements for entering into the nonlinear behavior range, followed by the elements of the bracing system. Two typical results of nonlinear response are shown in Figures 9 for a stopper element with a Yield Surface equal to a softened AISC 3D Interaction Curve for I-shaped section (McGuire, 2000) and in Figure 10 for a bracing element.

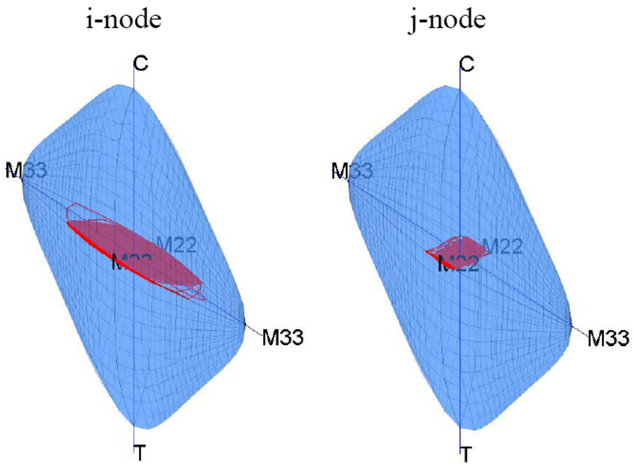


Figure 9: Typical 3D Axial-Moment forces trajectories in Force space, as obtained from THA. For this example, the maximum strength values are Ty=869t, Cy=819t, My22=42t-m, My33=237t-m.

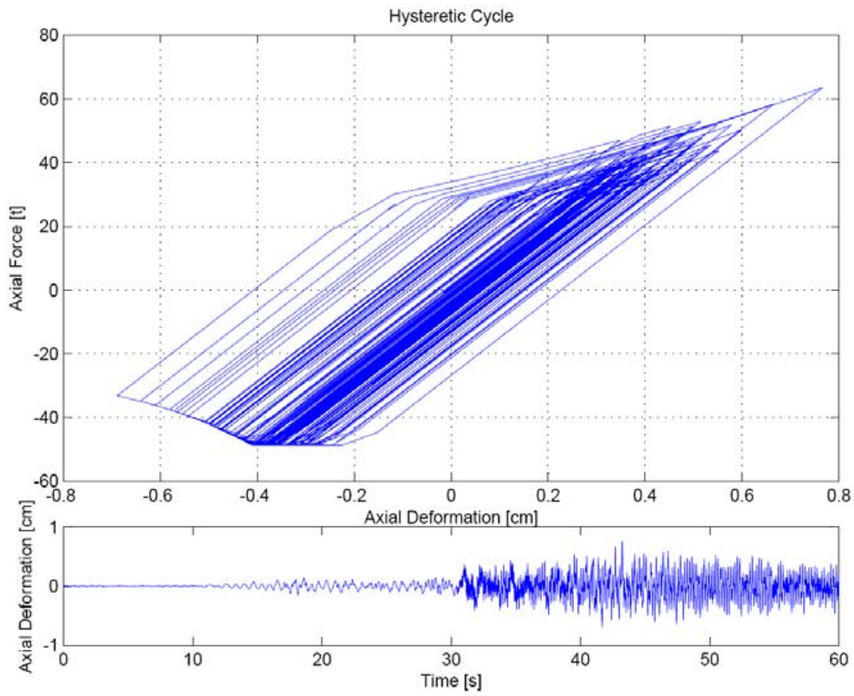


Figure 10: Typical hysteretic cycle of a brace. For this example, the maximum strength values are Ty=95.3t, Cy=48.7t.

6. CONCLUSION

None of the lateral distribution forces used for the PA show consistent results with the RSA when the structure is in nonlinear range (Earthquake Level II and III). However, PA results show that the steel support structure that has been designed using the normal design procedures for this type of structure based on the RSA results behaves adequately when subjected to large intensity earthquake actions. The inelastic deformations occur first in stoppers and then in the vertical bracing system elements, and at very large lateral displacements ending with column damage; achieving the general objective of the seismic design philosophy of preventing collapse and limiting damage at low level earthquake action. Thus the design of the typical boiler steel support structure, carried out by RSA can be considered validated by the PA results, which includes in a limited manner the effects of non linear range behavior of the elements.

Although the behavior of the structure can be considered satisfactory, this comes more from a generalized over-strength in the design rather than from an efficient design. The prediction of the inelastic behavior of elements based on RSA results compared with the results of the PA is only of medium quality for bracing system and of poor quality for stoppers. This last observation is very important considering that the stoppers are the key elements in controlling the earthquake behavior of the boiler system.

Although the pushover analysis was carried out with lateral force distributions that do not represent the actual distribution of force during an earthquake; and recognizing that monotonically increasing loads do not impose the cyclic behavior typical of earthquake actions to the structural elements, and therefore hysteretic effects in element behavior are not included in the analysis; still the results obtained allow to identify the sequence of inelastic behavior in the elements of the structure and their ductility demands. Thus, these results can provide relevant information for the design process.

For earthquake level II, the structure has a middle-range incursion in inelastic behavior and for earthquake level III, there is an important incursion in inelastic behavior. Maximum drifts values allowed in the Chilean code are exceeded only for Earthquake Level III, but "breaking" ductility values are not obtained. Thus, global and local seismic behavior can be considered as satisfactory. Although elements forces and drifts show a trend of being larger in simplified models than in the complex model, it is not possible to establish a reliable design procedure based in the simplified model results assuming an additional safety factor, since the element forces distribution may change considerably from one model to another. The use of simplified modeling is suggested only for preliminary design stages of the project, for general evaluation of the structure. For the final design stage of the elements, a significantly more detailed model should be considered.

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