



STRENGTH METHOD FOR EXPOSED BASE PLATES WITH EXTENDED ANCHOR BOLTS

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Abstract

Column Base Connections are one of the most critical components of Special Steel Moment Frames (SMFs). These building components transfer the demands from the super-structure to the concrete foundation and constitute an interface between them. Because of their importance, these connections have been extensively studied during the last years. Experimental and analytical studies have been conducted to determine their primary modes of response, limit states, strength and stiffness methods, ending up in design guidelines (e.g. Steel Design Guide 1). Traditionally, exposed base plates are preferred for low to mid-rise buildings while embedded base connections are the norm for taller buildings. This research presents a methodology to design and estimate the strength of a ductile type of exposed base plates with extended anchor bolts. This type of connection is common in seismic areas such as Chile, especially for industrial facility buildings. This method aims at concentrating all plasticity in the extended region of the anchor bolts. The rest of the components (i.e. base plate, steel chair and stiffeners) are detailed by Capacity Design Criteria to sustain the anchor bolts strength. The extended length of the rods relies on an assumed target rotation capacity of the connection (e.g. 0.02, 0.04 rad). The method is validated against sophisticated nonlinear finite element simulations. These simulations serve for two purposes: i) provide general insights into the connection behavior that permit develop the strength method; ii) constitutes a test-bed to validate the proposed strength method. General recommendations for design are suggested, and limitations of the method and recommendations for future research are provided.

Keywords: exposed base plates; column base connections; ductile base plates



1. Introduction

It is worldwide accepted that column base connections (CBCs) are one of the most important components of Steel Moment Frames (SMFs) since they transfer the loads (e.g. gravity, seismic, wind, etc.) from the entire superstructure to the concrete foundation. Depending on the height of building and the demands at the base, two types of CBCs are used. For low- to mid-rise buildings, exposed base plates (EBPs) are usually preferred. This detail consists of a steel column welded to a base plate, bolted to the concrete foundation, with a grout pad at the interface between the base plate and the foundation (Fig.1). On the other hand, in high-rise and tall buildings, embedded base connections are the norm. These connections consist of a steel column welded to a base plate and embedded into the concrete foundation. Sometimes shear studs are added to column flanges in these configurations.

It is well accepted that EBPs play a key role in the seismic performance of SMFs. Because of this, EBPs have been extensively studied over the last decades. Early studies (e.g. [1], [2], [3], [4]) on these connections focus on modes of failure and connection behavior. These studies led to strength methodologies such as the one proposed by Drake and Elkin [5]. At least 200 specimens of these connections have been tested in experimental programs over the last 30 years (Grauvilardell et al. [6]). These publications led to the development of design guidelines such as the Steel Design Guide 1 [7], which details a strength method for the exposed base plate of the type shown in Fig. 1. Refinements of this strength method have been proposed during the last ten years after full-scale experimental programs (e.g. Gomez et al. [8] and Kanvinde et al. [9]). Moreover, the Seismic Provisions (AISC 341 [10]) and the Design Manual's Example 8 [11] provide guidelines for the seismic design of EBPs. These documents and the Steel Design Guide 1 form the basis of modern connection design in the context of US practice.

Although Steel Design Guide 1 provides a strength method for EBPs, it does not present methodologies either for the estimation of their rotational stiffness nor recommendations to achieve ductile behavior. Recent studies have addressed some of these issues. For example, Kanvinde et al. [12] presented a methodology to compute the rotational stiffness of EBPs by calculating the initial secant stiffness by adding the deformation of the components within the connection. Torres-Rodas et al. [13] presented a phenomenological model to predict the hysteretic behavior of these connections. Stamatopoulos and Ermopoulos [14] and Kanvinde et al. [15] developed sophisticated nonlinear finite element simulations suggesting improvements in the available strength methods (i.e. Steel Design Guide 1 [7]).

Other studies, such as Zareian and Kanvinde [16], and Aviram et al. [17] show the impact of base flexibility in the seismic performance of SMFs. Falborski et al. [18] conducted a parametric investigation of the correlation between base connection strength, stiffness and rotation capacity with the probability of collapse of the SMFs. The results of this study indicate that seismic demands on CBCs could be reduced (respect to capacity design criteria) in order to leverage base ductility. Moreover, results from various experimental programs (e.g. [1], [2], [3], [8]) indicate that these connections pose a significant deformation capacity. In contrast, data collected by Lignos and Krawinkler [19] suggests that ductility in columns may be compromised due to local buckling problems. These findings suggest that ductility of base connections could potentially be leveraged in seismic designs. Research on ductile base connections has been conducted by Trautner et al. ([20], [21]). Interestingly, the results of these studies suggest that an increase in anchor bolt stretch length entails an improvement in rotation capacity.

Motivated by these issues, this paper presents a method to characterize the strength of a ductile exposed base plate. The method is based on the insights gained from sophisticated nonlinear finite element (FE) simulations conducted on computational models, developed herein, and supplemented with the available information on the topic. The FE simulations were performed on the finite element ABAQUS platform [22] and had twofold purposes: i) provide overall insights of connection behavior (i.e., patterns of deformations, stress distribution; ii) constitute a testbed to validate the proposed strength method. The connection proposed in this study (Fig. 2) consists of a steel column welded to a base plate. Anchor bolts are extended over the exposed base plate to concentrate all plasticity in these regions. A steel chair accompanied by vertical and horizontal stiffeners (Fig. 2) assembly this connection detail. These types of configurations are usual in



industrial facilities in seismic regions, such as Chile. It is important to point out that studies such as Soules et al. [23] highlight the excellent performance of these connections in the 2010 Maule, Chile earthquake. These arguments motivate the necessity to explore different ductile base connection configurations that might improve the seismic response of SMFs. The study is divided into four sections. First, the problem is propounded in this introductory section. This is followed by a complete description of how the FE models were developed. In total, nine models were built. The effects of the two main parameters were investigated. The first one is the target rotation of the connection. This parameter defines the extended length of the anchor bolts over the base plate, given a maximum allowed anchor bolt elongation capacity (in this study, a 10% strain for the target rotation was considered). The second one is the axial compressive force. Subsequently, the strength method and design recommendations are set, and finally the results, limitations of the current work, and future lines of research are discussed.

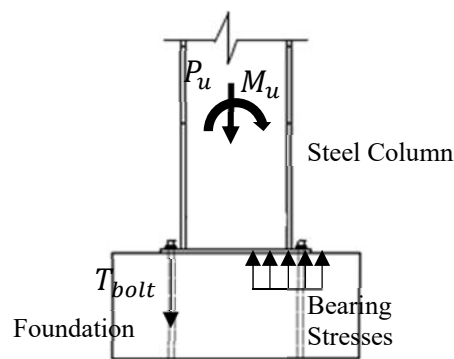


Fig. 1: Exposed Base plate as per DG1

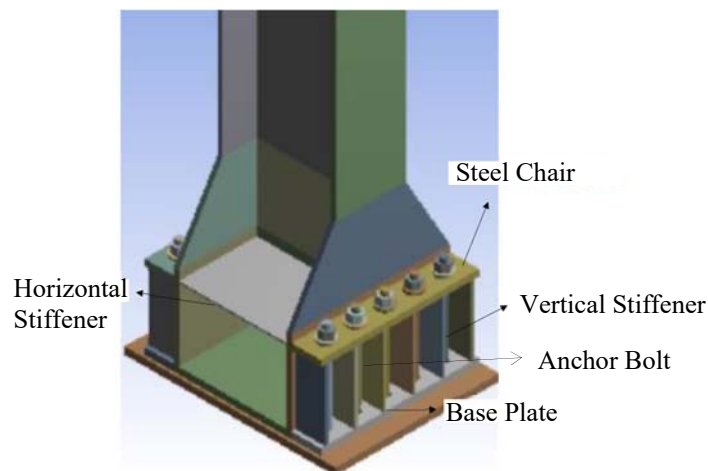


Fig. 2: Exposed Base plate with extended anchor bolts

2. Finite Element Simulations

Fig. 3a indicates a representative FE model developed in this investigation. All models were constructed in the ABAQUS [22] simulation platform. The finite element models seek to capture all the important aspects that influence the connection response (i.e., strength, initial stiffness, and hysteretic behavior). Earlier studies on exposed base connections indicate that the interaction between the connection components primarily controls their response (e.g., base plate, concrete foundation/grout pad, anchor bolts and nut-washer arrangement). Therefore, all models consider appropriate algorithms to simulate component interaction and contact (i.e., base plate and concrete foundation, nut-washer arrangement and steel chair, anchor bolt, and concrete foundation)



as well as nonlinear material constitutive response. Fig. 3b illustrates the contact and gapping within the components.

The key aspects of the models are discussed. The interface between the concrete foundation and the base plate was simulated through surface-to-surface contact properties. This criterion intends to assign finite sliding. The normal contact was set as “hard contact,” whereas the tangential behavior was defined as an isotropic friction formulation. A coefficient of friction of 0.45 was used for the tangential behavior based on the findings from Gomez et al. [8]. This is consistent with nonlinear FE simulations conducted on EBPs (and validated with test data) by other researchers (e.g., Kanvinde et al. [15]). A similar criterion was applied to model contacts between steel chair, and nut-washer arrangement and anchor bolts– concrete foundation. The only difference is that the friction coefficient for the tangential behavior was 0.80 in the case of steel-steel contact, based on the recommendations from Grigoriev et al. [24]. The rest of the connection components were modeled as monolithic, with the use of “tie constraints” since they are welded together, and welds (CJP) are expected to fracture after rotations beyond 0.06 rad. Evidence of this has been reported by Gomez et al. [8].

Regarding constitutive material response, two types of material models were employed. For steel components (including steel column, base plate, steel-chair, stiffeners), the Von-Mises surface with isotropic hardening was specified. For the concrete foundation, the Drucker-Prager [25] pressure dependent material-model was used. Expected values of the materials (AISC 341 [10]) were considered in the simulations since no experimental program was conducted in this research. All the models were subjected to the ATC-SAC load protocol (Gupta and Krawinkler [26]) in the presence of an axial compressive load. In order to avoid the induction of P-delta moments at the base, the axial load was strategically applied. As shown in Fig. 3a, this load was concentrated at the bottom surface of the steel column. The nodes of this surface were constrained through a “rigid body constraint.” The metrics monitored in all simulations are force reactions at the base of the connection, top column displacements, bearing stresses between the base plate and concrete foundation, and anchor bolt forces. A total of nine models were developed. The difference between the models is the design loads, and the target rotation of the connection. Table 1 summarizes the models presented in this paper.

The results of this set of simulations must be interpreted as: i) the first attempt to capture overall connection response, identifying component deformation modes within the connection, bearing stress distribution between base plate and foundation, anchor bolt forces, and hysteretic behavior; ii) in the absence of a full-scale experimental program, these simulations represent the testbed to validate the strength method proposed herein. Fig. 3a illustrates a representative FE model developed in this research (case 1). Fig. 3b shows the deformed shape of the connection under lateral displacements in the presence of axial compressive force. Fig. 4a presents the hysteretic curve (moment vs. rotation) obtained in the FE simulations conducted on case 1 (Table 1) whereas Fig. 4b shows the hysteretic response of one of the specimens tested by Gomez et al. [8]. The intention of presenting these two figures (i.e. Figs. 4a and 4b) is to compare the response of the traditional detail (TD) (Fig. 4b) [8] and the proposed configuration with extended bolts (ND) (Fig. 4a). The criteria utilized to make this comparison was to design the ND with the same loads as the ones reported in Gomez et al. [8] for the TD. The main observations are discussed next.

Referring to Fig. 4b, the hysteretic curve of the connection is defined by two main characteristics: the pinching behavior and the hysteretic flag-shape. The first phenomenon may be attributed to contact and gapping in the interface between the base plate and the foundation. This explains the significant change in the stiffness during the history of deformations, which entails the pinching behavior characteristic of base connections. On the other hand, the flag hysteretic shape is due to the presence of axial load which leads up to the re-centering effect. A similar hysteretic response has been reported for the traditional exposed base plate detail. For further details on the topic, refer to [13]. A closer inspection of Fig. 4b reveals that this new detail, compared with the traditional one (Fig. 4a) presents significantly less cyclic deterioration. Specifically, Fig. 4b reveals an improvement in the capacity to maintain the *peak strength* of the connection stable. Virtually, no peak strength deterioration is observed up to 0.08 rad. The strength deterioration of the intermedium plateau is considerably less as well. This may be attributed to the concept of concentrating all the plasticity in the extended length of the anchor bolts. In contrast, the traditional detail may yield in either i) base plate in the tension side, ii) base plate in the compression side, or iii) in the anchor bolts. The overall hysteretic response



is improved. The quality of the bolt materials could potentially affect the hysteretic response of the connection. In this study, only ASTM F1554 GR 36 anchor bolts were used. However, the authors in [20] and [21] have shown the effect of this parameter in the connection response. Consequently, this important topic deserves further scrutiny in a future study.

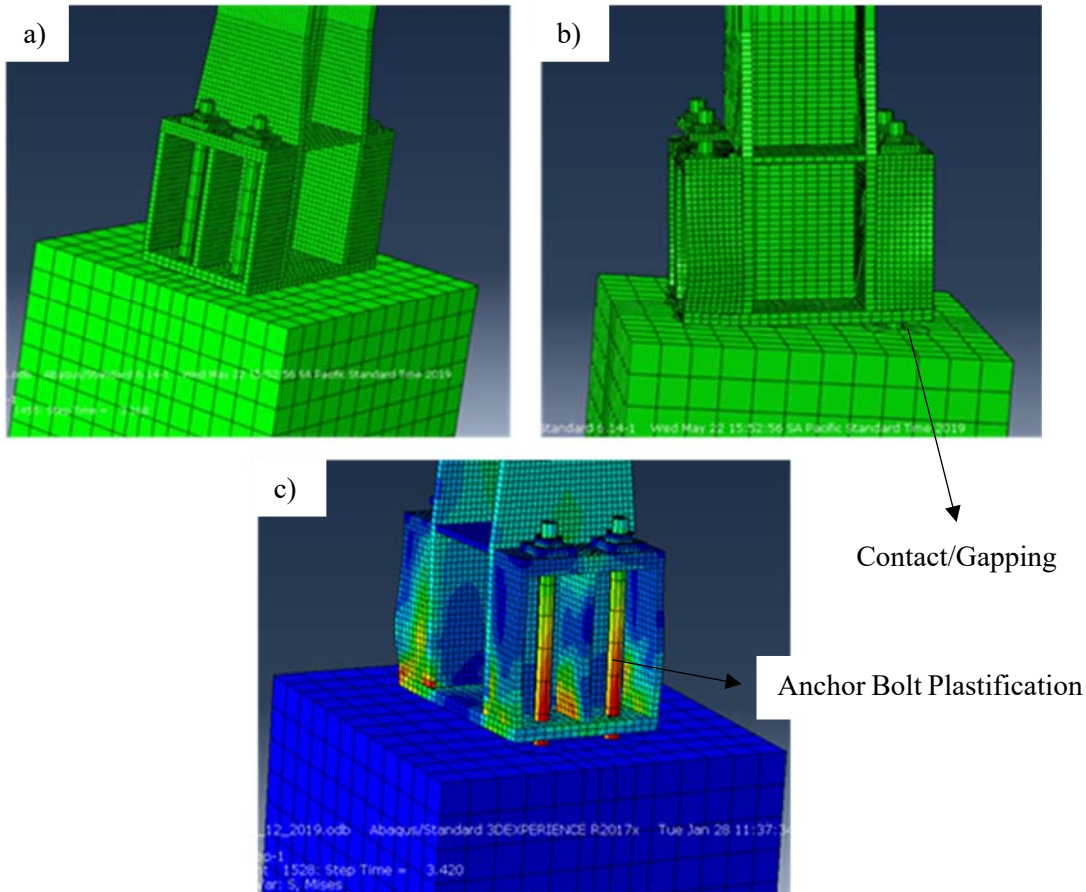


Fig. 3: FE simulation models: a) schematic illustration of the model; b) patterns of deformation; c) stress distribution

3. Strength Method

Building on the insights gained from the FE simulations, this section describes a method to calculate the strength of a ductile type of exposed base plates with extended anchor bolts. Recommendations for the design of all the components of the connection are provided. This method utilized the main aspects of Steel Design Guide 1 [7] which will be detailed herein. First, the method assumes a predetermined form of bearing stress distribution in the interface base plate-foundation. Specifically, a uniform rectangular stress block is considered. Studies such as Kanvinde et al. [15] demonstrated drawbacks from this assumption. However, up to the date, the rectangular bearing stress block is used worldwide in the modern design of exposed base connections. As per this assumption, seismic demands (i.e., M_u and P_u) are taken by a force-couple developed by the resultant force of the bearing stresses in the compressive side and the tensile forces in the anchor bolts. The magnitude of the bearing stresses is given by Eq. (1):

$$f_{max} = 0.85 f'_c \left(\sqrt{\frac{A_2}{A_1}} \right) \leq 1.7 f'_c \quad (1)$$



Next, the anchor rod forces, T , and the length of the rectangular stress block, Y , (Fig. 1) are obtained by the equilibrium of forces within the connection [5]:

$$Y = (N - g) - \sqrt{(N - g)^2 - \frac{2 [M_u + P_u (\frac{N}{2} - g)]}{f_{max} B}} \quad (2)$$

$$T = f_{max} B \left\{ (N - g) - \sqrt{(N - g)^2 - \frac{2 [M_u + P_u (\frac{N}{2} - g)]}{f_{max} B}} \right\} - P_u \quad (3)$$

In Eq. (1) A_1 is the area of the base plate, and A_2 represents the maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area. In Eqs. (2) and (3), N is the length of the base plate; M_u and P_u are the design moment and compressive axial load respectively; B is the width of the base plate; g is the distance from the holes to base plate edges, and f_{max} is the rectangular stress block derived using Eq. (1). The design strength of the connection corresponds to the attainment of anchor bolts yielding limit state, and the corresponding moment is defined as the moment at first yield M_y .

The method proposed herein aims at concentrating all the expected plasticity in the extended region of the anchor bolts (Fig. 2). As per this criterion, the rest of the connection components (i.e. base plate, concrete foundation, a steel chair and stiffeners) are detailed by Capacity design criteria to resist the anchor bolts fully plastified and strained hardened capacity. Because of this, the seismic demands suggested for the connection design proposed in this paper (i.e., M_u , P_u) must be calculated for reduced seismic loads. Traditionally, CBCs are designed for $M_u = 1.1 R_y M_p$ of the attached column representing a “strong connection – weak column” criterion. However, AISC 341-16 (section D2.6c) permits a reduction in the seismic demands if a ductile limit state in the column base controls the design. A complete study on seismic demands on CBCs was conducted by Torres-Rodas et al. [27]. Falborski et al. [18] carried out a parametric study (being the number of stories the main parameter investigated) to explore the correlations between CBC strength, stiffness, and rotation capacity with the probability of collapse of SMFs. The results of this study indicate that connection demands can be reduced if they are detailed to hold rotations of at least 0.05 rad. The FE simulations run in this research indicate that these connections must pose a rotation capacity of at least 0.06 rad. These findings are not surprising considering the evidence presented in notable experimental programs such as [1], [2], [3], [4], [8]. These arguments strengthen the motivation of exploring ductile base connection details as part of the dissipative mechanism of SMFs.

Once the forces within the connection components are calculated, the next step is to determine the number and diameter of anchor bolts. According to LRFD procedure: $T \leq R_y F_y A_{bolt}$ and $R_y F_y A_{bolt} \geq 0.75 F_u A_{bolt}$. Appropriate bolt materials must be employed (e.g. ASTM F1554 Gr36). Trautner et al. ([20], [21]) highlighted the impact of anchor bolt material on the rotation capacity of base connections. As mentioned in the previous section, this topic deserves further scrutiny. Both base plate and steel chair (Fig. 2) has been designed based on Yield line theory considering boundary condition as recommended next (Fig. 6). It is suggested to assign yield lines only in continue plate areas and not in welded joints. Therefore, the steel chair and base plate thicknesses are calculated from Eq. (4) to Eq. (9), respectively:

Steel Chair thickness:

$$\phi P_n \leq P_u = T \quad (4)$$

$$\phi P_n = \phi M_n k_{pc} \quad (5)$$

$$\phi M_n = 0.9 \frac{e_c^2}{4} F_y \quad (6)$$



Base plate thickness:

$$\phi P_n \leq P_u = q_u a b \quad (7)$$

$$\phi P_n = \phi M_n k_{pb} \quad (8)$$

$$\phi M_n = 0.9 \frac{e_p^2}{4} F_y \quad (9)$$

In these equations, k_{pb} and k_{ps} are the yield line factors for the base plate and steel chair, respectively, they depend on the geometric configuration of the plates; e_p and e_c are the base plate and steel chair thicknesses, respectively; q_u is the concrete bearing stress, “a” and “b” are the longitudinal and transverse dimensions of the analyzed panel of the base plate. The stiffeners (Fig. 2) must be designed to hold a compressive force, coming from the greatest between anchor bolt capacity and the maximum reaction in the interface base plate-foundation. Finally, the extended anchor bolt length (Fig. 2), one of the most essential details in this base connection configuration, is defined based on a target rotation capacity.

4. Results

This section presents the results obtained by applying the proposed design methodology on nine connections representative of mid-rise buildings. The intention is to validate the goodness of the method through the FE mentioned above simulations. All models were developed in the ABAQUS finite element platform [22]. The metrics recorded in the simulations were the ones described in section 3 (i.e., top column displacements, base reactions and bolt forces). In this study, the models were grouped in order to parametrically evaluate the impact of two metrics (i.e., target rotation of the connection and level of axial compressive force in the response). Target rotation is an essential parameter in this configuration since it defines the extended length of the bolts. The level of axial load plays a key role in the behavior of exposed base plates as it has been extensively reported [8], [13], [27]. Specifically, in this study two target rotations were evaluated: 0.02 rad and 0.04 rad. Moreover, three levels of axial load were considered: 10%, 20% and 50% of the axial capacity of the column (i.e., $R_y F_y A_g$). Table 1 summarizes the details of the models developed herein showing the design demands (P_u , M_u), base plate size (BxN), extended length of the anchor bolts over the base plate (L_{ext}), base plate thickness (t_p), anchor bolt diameter (D_{bolt}) and the total number of anchor bolts. Table 2 presents the results obtained in this study for the proposed method validated through the FE simulations. Specifically, Table 2 presents the connection capacity calculated by the analytical method suggested herein and the connection capacity obtained from the FE simulations. The error is defined as the ratio between the connection capacity from the simulations divided by connection capacity from the method where a value of 1 indicates that the method does not deviate for FE simulations. Table 2 summarizes the error for each case analyzed. As per Table 2, all the errors are close to one indicating that the method provides a reasonable estimate of connection capacity. The average error among all the cases is 1.07 with a COV=0.06. These results indicate that the method calculates the connection capacity with accuracy.

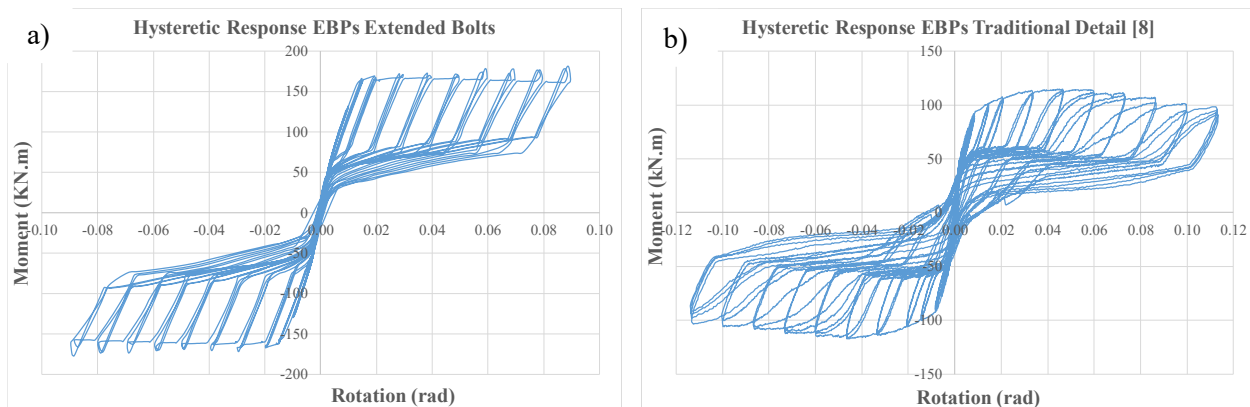
Fig. 5 illustrates the response of four connections varying the level of axial load (i.e. 10 and 20% of $R_y F_y A_g$) and the target connection rotation (i.e. 0.02 and 0.04 rad). Similar behavior is observed when the target rotation is changed for the same level of axial load (Figs. 5a and 5b; Figs. 5c and 5d). However, results indicate that the level of axial load affects the hysteretic shape of the connection by reducing the pinching behavior. By comparing Figs. 5c and 5d with 5a and 5b are evident that higher axial loads tend to reduce the pinched effect. This may be attributed to the fact that a greater axial compressive force would delay base plate uplift leading to a reduction of the gapping between the concrete foundation and the base plate which is the main cause of pinching behavior in base connections.

**Table 1 – Base Plate Design Parameters**

Case	P_u (kN)	M_u (kN m)	B (m)	N (m)	L_{ext} (m)	t_p (mm)	D_{bolt} (mm)	Number of bolts
1	411	98	0.25	0.40	0.27	20	25	4
2	670	1730	0.75	1.10	0.30	35	64	6
3	3340	1730	0.70	1.05	0.30	30	57	6
4	670	1730	0.75	1.10	0.50	35	64	6
5	3340	1730	0.70	1.05	0.50	30	57	6
6	670	520	0.40	0.85	0.20	20	32	6
7	670	520	0.40	0.85	0.37	20	32	6
8	1338	520	0.40	0.85	0.20	20	25	6
9	1338	520	0.40	0.85	0.37	20	25	6

Table 2 – Analytical model results

Case	Steel Column	Target Rotation (rad)	$\frac{P_u}{F_y \cdot A_g}$	$M_y^{simulations}$ (kN m)	M_y^{method} (kN m)	$\frac{M_y^{simulations}}{M_y^{method}}$
1	W8x48	0.04	0.18	159	151	1.05
2	W24x103	0.02	0.10	2200	2222	0.99
3	W24x103	0.02	0.50	2408	2574	0.94
4	W24x103	0.04	0.10	2470	2222	1.11
5	W24x103	0.04	0.50	2750	2574	1.07
6	W24x103	0.02	0.10	710	630	1.12
7	W24x103	0.04	0.10	720	630	1.14
8	W24x103	0.02	0.20	760	670	1.13
9	W24x103	0.04	0.20	760	670	1.13

**Fig. 4:** Hysteretic Response: a) EBPs with extended bolts; b) Exposed Base Plate [8]

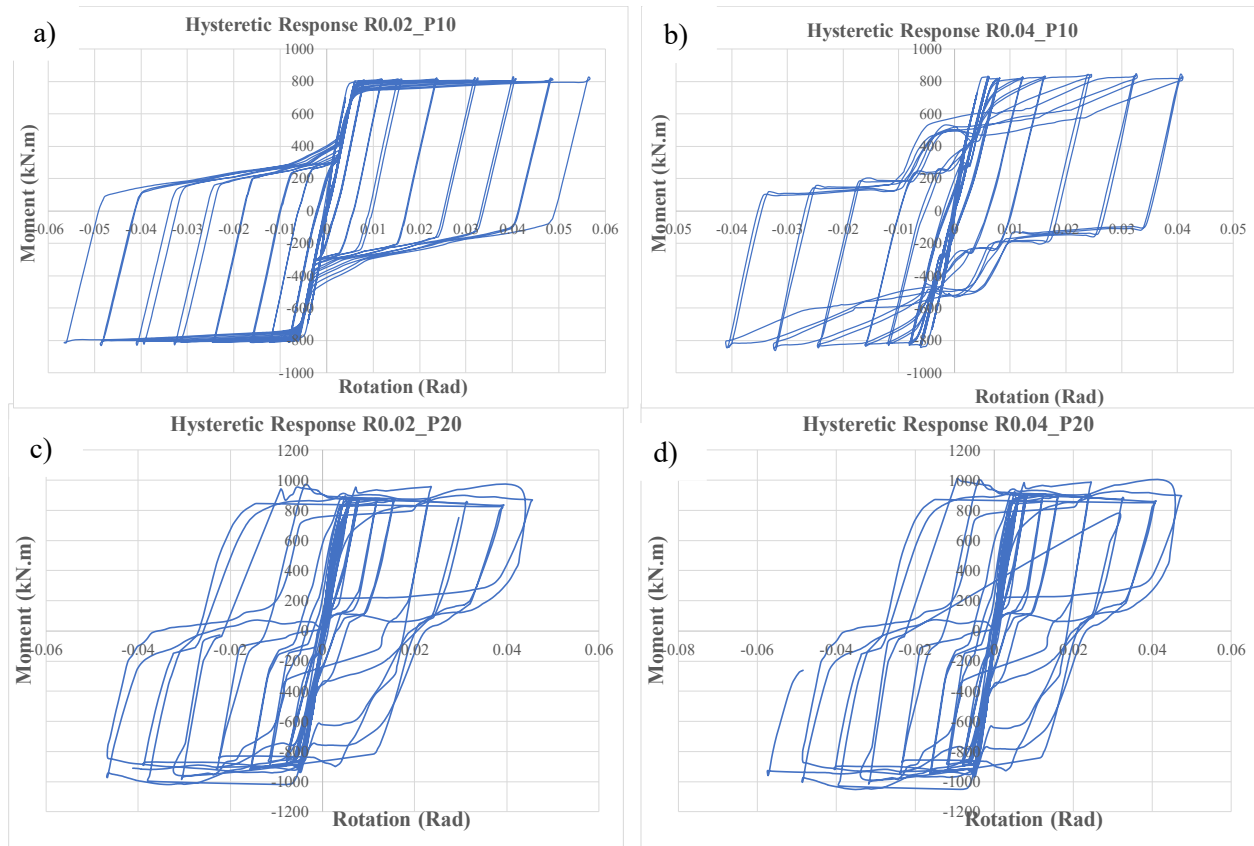


Fig. 5: Hysteretic Response: a) Case 6: Rot0.02_P10; b) Case 7: Rot0.04_P10; c) Case 8: Rot0.02_P20; d) Case 9: Rot0.04_P20

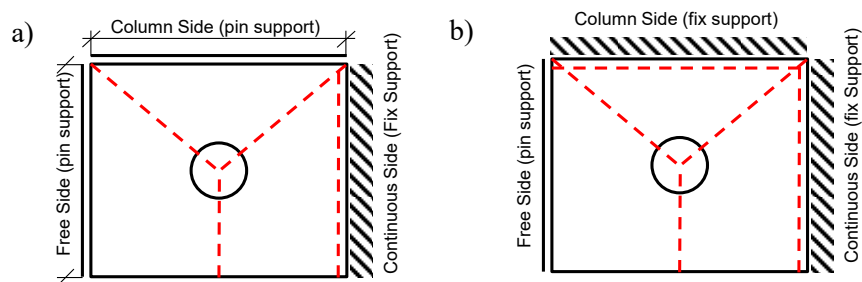


Fig. 6: Plate yielding line model adopted: a) Left: Steel Chair; b) Right: Base or bottom base plate

5. Summary, Conclusions and Future Work

This paper describes the hysteretic response of exposed base plates with extended anchor bolts based on FE simulations and presents a method aimed at designing this configuration. This detail consists of a steel column welded to a base plate anchored to the concrete foundation. Usually, a grout pad is set between the foundation and the base plate. The anchor bolts are extended over the base plate. Steel chairs and stiffeners assemble the connection. The intention of this design criterion is to concentrate all plasticity in the extended region of the anchor bolts. Henceforth, the rest of the connection components (i.e., base plate, a steel chair, and stiffeners) must be designed to resist the full capacity of the bolts. The material selected for the anchor bolts must be appropriate in order to develop ductile behavior.



Finite element simulations were conducted to understand the behavior of these connections. The simulations indicate that significant rotation capacity can be achieved under the investigated detail. The hysteretic curves obtained show little strength deterioration with excellent deformation capacity. This is not surprising, since several experimental programs show that exposed base plates pose rotation capacities in the range of 0.05-0.10 rad. However, a closer inspection when comparing the hysteretic curves obtained in this study with the ones reported in studies such as [3], [4], and [8] reveals a better hysteretic response in this base plate with extended anchor bolts detail. For instance, peak strength deterioration is virtually negligible up to rotations in the order of 0.08 rad. Also, unloading stiffness deterioration is lower compared with the traditional detail. In the authors' analysis, the overall hysteretic response is more stable.

The strength method proposed herein relies on fundamental assumptions suggested by the Steel Design Guide 1. Specifically, a uniform rectangular stress block is assumed, to simulate the bearing stresses in the compression side of the connection. Equilibrium equations are set to compute the forces in the anchor bolts. These forces define the type and diameter of the anchor bolts. Using capacity design criteria, the rest of the connection components are detailed. This configuration has been extensively used in industrial facilities in Chile. Studies such as Soules et al. [23] have indicated that these connections performed well in the 2010 Maule earthquake, which motivates them to explore this topic further.

Although this study indicates that the exposed base plates with extended anchor bolts have excellent deformation capacity and stable hysteretic response, it has several limitations that must be addressed. First, no experimental program was conducted. FE simulations are the only test-bed to validate the design method described herein. A full-scale experimental program is recommended to validate further the insights detailed in this paper. Second, the method is aimed at calculating the connection strength only under uni-axial bending. Third, a limited number of models were developed. Moreover, only one type of anchor bolt was used in the models. More simulations with different types of anchor bolts are recommended for future investigation considering the profound impact of this parameter in the performance of the connections (especially in their rotation capacity). Finally, the method assumes a rectangular stress block on the compression side. Studies (e.g. [14], [15]) have pointed out the drawbacks of this assumption but this is the methodology used up to date in modern design. It is recommended to study this issue through appropriate computational models further.

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