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RELIABILITY ANALYSIS OF EMBEDDED BASE CONNECTIONS

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Abstract

Column base connections are critical components of Steel Moment Frames (SMFs) since these connections transfer the loads (e.g., gravity, seismic) from the superstructure to the concrete foundation, being an interface between them. Because of it, these connections have been extensively investigated in the last decade. Notable large-scale experimental programs have been conducted, and numerical and analytical investigations have complemented these programs. Traditionally, column-base connections are divided into exposed base plates and embedded bases. The former type of connection is mainly used for low and mid-rise buildings, while the latter detail is the norm for tall buildings.
The behavior of embedded base connections has been studied in recent experimental programs. As a result, design methodologies and models to simulate their hysteretic characteristics have been proposed. However, surprisingly a reliability analysis of these connections has not been conducted so far. Motivated by this issue, this study presents a reliability analysis of the embedded base connections with the aim of providing design recommendations. For this purpose, nonlinear mathematical models of archetype frames are developed and subjected to nonlinear dynamic analysis to obtain the column base connection demands. These results are then used to conduct statistical simulations and recommend preliminary resistance factors for the design of embedded base connections. Limitations of the investigation are discussed, and future lines of investigation are outlined.

Keywords: Embedded Base Connections, Reliability Analysis, Earthquake Engineering, Steel Moment Frames

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1 INTRODUCTION

In the context of earthquake-resistant design, column-base connections are perhaps the most critical component of Steel Moment Frames (SMFs). These connections transfer the loads (e.g., dead load, live load, or earthquake load) from the superstructure to the concrete foundation, being an interface between them. In relation to US engineering practice, column-base connections are traditionally divided into two groups: exposed base plates (EBPs) and embedded base connections (EBCs). The former detail is used in low and mid-rise buildings, while the second detail is the norm for tall buildings, where the applied loads make the exposed base plate detail impractical. Figure 1 illustrates both types of column base connections.

The importance of column base connections in the seismic performance of SMFs has led to the development of numerous research programs. Early studies on the topic started with experimental programs aimed at identifying modes of failure, understanding the performance, and establishing strength methodologies for EBPs. (e.g., [1–4]). These studies resulted in notable analytical models to predict the strength of EBPs (e.g., [5]). In the US, the Steel Design Guide 1 (DG1) [6] provides a design methodology for EBPs, while the Seismic Design Manual (Example 9 [7]) details practical examples regarding their design procedure. These documents constitute the basis of the modern design of EBPs. Subsequent studies on EBPs included experimental programs to refine the design methodologies described earlier (e.g., [8]). Other studies presented methodologies to estimate their rotational stiffness [9] and models to simulate their hysteretic behavior [10].

In contrast to EBPs, embedded base connections received attention more recently. Grilli and Kanvinde [11] conducted a large-scale experimental program to study the behavior of embedded base connections and suggested a strength method based on the tests. This method assumes an internal stress distribution responsible for counteracting the applied loads (Axial Load and Flexural Moment). This assumption is illustrated in Figure 2. As per this internal stress assumption, the applied loads are resisted by the interactions between the embedded column and the surrounding concrete foundation. Thus, the method postulates that the applied moment is resisted by the force-couple developed by the horizontal and vertical stresses (Figure 2), and the fraction of moment taken by each mechanism is computed with an empirical factor that depends on the embedded length.

Later studies on EBCs focus on methods to estimate their rotational stiffness and simulate their hysteretic behavior. For instance, Torres-Rodas et al. [12] developed a methodology to compute the secant stiffness of these connections by aggregating the deformations within their components (i.e., embedded column, surrounding media). Similarly, Torres-Rodas et al. [13] presented a hysteretic model to simulate the cyclic behavior of EBCs. This phenomenological model consists of a two-parallel springs model. Each spring captures the assumed load-carrying mechanism, i.e., horizontal bearing stresses and vertical bearing stresses. The model includes hysteretic rules appropriate for these connections as well as rules to predict cyclic deterioration observed in the experimental tests.

More recently, efforts have focused on determining appropriate load combinations for their design. Specifically, Torres-Rodas [14] recommended design loads for these connections based on nonlinear time history analyses (NTHA) of archetype steel moment frames (SMFs). Other studies investigated their behavior using sophisticated finite element models (e.g., [15–17]) determining internal stress distributions and patterns of deformation. Apart from that, the influence of the behavior of EBCs on the seismic performance of SMFs has been studied with the use of NTHA conducted on archetype frames [18–21]. These studies conclude that the EBCs pose good hysteretic characteristics that can be incorporated as part of the energy dissipative mechanisms of SMFs.

Although EBCs have been studied extensively in the last decade, reliability analyses of these components are absent in the literature. In contrast to EBCs, reliability analyses have been conducted on exposed base plates [22–24], but at least, to our best knowledge, are absent for EBCs. Motivated by this issue, this paper presents the first approximation to address the reliability analysis problem of EBCs. Specifically, this study presents preliminary strength resistance factors appropriate for the seismic design of these connections. Results from NTHA conducted on an 8-story archetype frame are used as seismic demands.

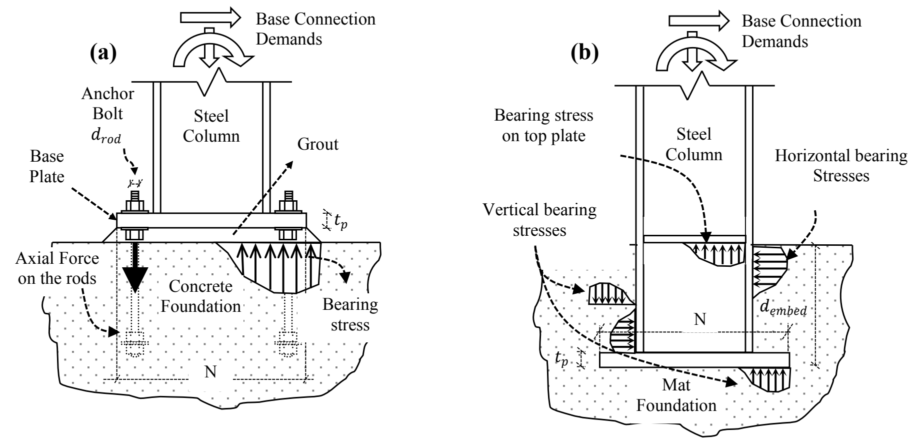

Figure 1: a) Exposed Base Plate, b) Embedded Base Connections [14].

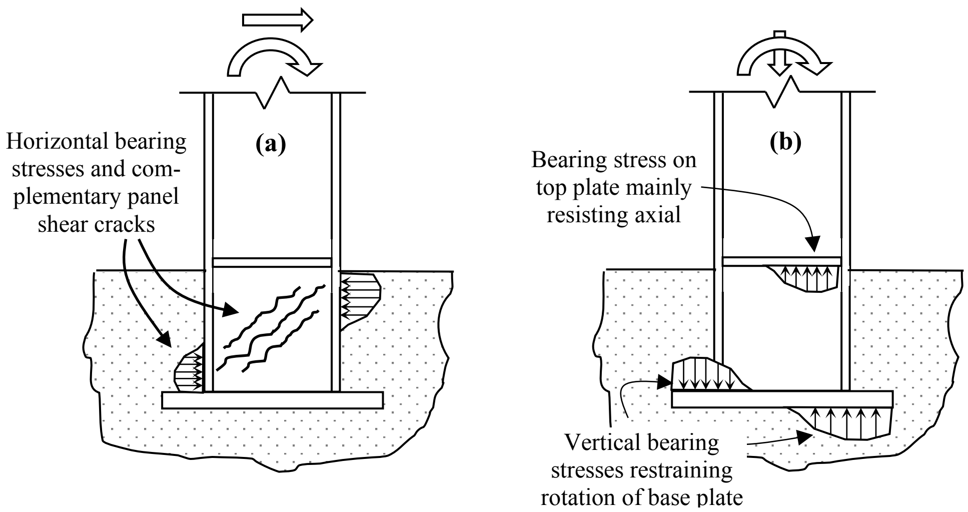

Figure 2: a) Horizontal bearing stresses, b) Vertical bearing stresses [12].

2 METHODOLOGY

This study conducted a reliability analysis of the design methodology of EBCs developed by Grilli and Kanvinde [11] using a single scenario, including dead, live, and earthquake loads. The earthquake loads are considered from NTHA carried out on an 8-story archetype SMF. Details of the nonlinear models and the ground motions used in the analyses are explained in the following subsections. The reliability analysis is conducted by means of Monte Carlo simulations, in which all the uncertainties are sampled from their respective assumed probability density functions (PDF) and the number of failure cases is counted to compute the probability of failure.

2.1 Archetype Frame

For this study, an archetype SMF is used to conduct NTHA and monitor the demands at the column base connection. Specifically, an 8-story frame representative of US construction practice is used. Fig. 3a illustrates the plan view, while Fig. 3b shows the mathematical model representative of the SMF. As per these figures, the SMF is located at the perimeter of the building, while the other components are part of the gravity framing system. The building was designed as per ASCE 7-16 [25]. Thus, a dead load of 4.78 kN/m^2 and an unreduced live load of 2.38 kN/m^2 were applied to all floors except the roof, where the live load applied was 0.95 kN/m^2. Apart from these loads, a perimeter load of 1.20 kN/m^2 was considered to simulate the cladding. For the seismic design of the archetype frame, it was assumed the seismic design category D_{max} and site class "D," which is consistent with the far-field conditions of the Los Angeles basin. The SMF is a three-bay frame with a width of 6.10m and a typical floor height of 3.90m, while the first-story floor is 4.50m.

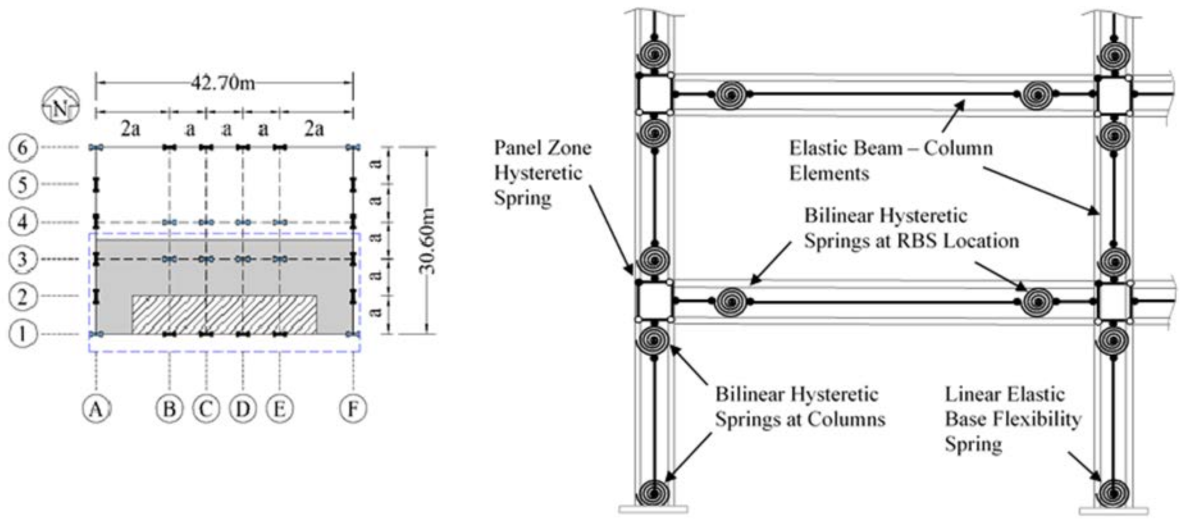

Figure 3: a) Typical plan view of all frames, b) Mathematical models of the SMFs

Referring to Figure 3b, concentrated plasticity models are used for the NTHA. This assumption is justified for the purpose of this research since several parametric studies on SMFs in the past have adopted this procedure (e.g., [18,19,21,26–29]). Thus, beams and columns are idealized as linear-elastic elements with nonlinear springs at their ends. These springs are simulated with the well-known IMK bilinear model [30]. This model consists of a trilinear backbone curve with rules to capture cyclic and in-cyclic strength and stiffness deterioration. As with all uniaxial plasticity models, the IMK model cannot capture the interaction between the axial load with the bending moment (P-M). This limitation is important in column members where P-M interaction is present and is addressed in an approximate manner. First, the gravity loads (dead load and live load) are assumed as an average value present during the NTHA. Next, the moment capacity of the columns is reduced with the interactive equations from AISC 360-16 [31], and this reduced moment is used in all the column springs. This procedure has been adopted in the past in several studies on SMFs (e.g., [20,21,29]) and has been validated experimentally in shaking tables [32].

The panel zones are idealized as a hinged parallelogram assembly by rigid elements with one single rotational spring in a corner to simulate shear distortion. A leaning column loaded with gravity loads corresponding to half of the area in the plan-building (Figure 3a) is used to capture the P-Δ effects through large displacement geometric nonlinear. Finally, the column base hysteretic behavior is simulated with the mathematical model developed by Torres-Rodas et al., 2018 [13] for embedded base connections.

2.2 Ground Motions

In this paper, the seismic demands in the EBCs are monitored through NTHA. The above-mentioned SMF is subjected to a total of 120 ground motion records. These records are obtained from the NGA-West 2 [33] database and consist of 4 sets of 30 records from different source conditions, including 1) near-fault ground motions from the reverse-fault mechanism, 2) near-fault ground motions from the strike-slipe mechanism, 3) far-fault ground motions from the reverse mechanism, 4) far-fault ground motions from the strike-slipe mechanism. The resulting set of 120 ground motions is scaled to the design basis earthquake intensity level (i.e., return period of 475 years) corresponding to the life safety performance limit state worldwide use for design purposes.

2.3 Column Base Connection Design

For the reliability assessment, a column base connection, representative of an 8-story SMF, is designed as per the strength method presented by Grilli and Kanvinde [11]. For this reason, the method is briefly summarized. First, the authors postulated a mechanism for internal force distribution within the connection. This mechanism consists of horizontal bearing stresses developed by the contact between the column flanges and the surrounding media, a concrete panel zone shear, and vertical bearing stresses of the embedded base plate. Thus, an indeterminate system results from this mechanism, which is solved by the use of an empirical coefficient that distributes the applied moment to each bearing mechanism (i.e., horizontal and vertical bearing stresses). This coefficient mainly relies on the embedded depth and is based on the Winkler beam theory on elastic media. Figure 2 idealizes the internal force distribution.

Following this design procedure, the CBCs of the 8-story SMF represented in Figure 3 are designed. For this purpose, a single-load combination scenario is assumed. Specifically, the load combination used herein to compute the axial load, shear, and applied moment is: $1.2D + 0.5L + \Omega_0 E$, which is consistent with published documents on the topic (e.g., [14,20,21]) where D and L are the Dead and Live Loads respectively, and E the earthquake load. The first to loads come from gravity analysis while the latter is computed from a pushover analysis. Table 1 summarizes the dimensions of the CBCs obtained.

Column type	d_{emb}(mm)	b_f (mm)	B (mm)	N (mm)	Mn (kN.m)
Exterior	406	328	558	660	1167
Interior	914	508	762	762	2309

Table 1: Column base connection dimensions.

2.4 Reliability Assessment

The reliability analysis is divided into two components. First, from the NTHA, the reactions in the connections are computed as described previously. From each analysis, the envelopes of the total shear V, axial P, and moment M in the connection are obtained. From these results, it is clear that these quantities are uncertain, and their probability density function (PDFs) needs to be estimated. From the numerical data, a kernel density estimation is computed for each quantity, which is assumed as a good estimation for the reliability analysis.

The second component consists of Monte Carlo simulations to estimate the probability of failure. The uncertain demands are given by V, P, and M in the connection, which is sampled from the estimated PDF in the first component. It is needed the inverse cumulative density function (CDF) for each of the random variables so that the variables are given by the inverse CDF of a random variable sampled uniformly in the interval [0,1]; the CDF is obtained directly, and a simple interpolation is used to obtain other values of the inverse CDF. These random variables are assumed as independent for this preliminary analysis. For the geometry and material properties, the following table provides the summary of the random variables considered, which are based on values in the study by Song et al. [24]. In this study, the dead and live loads are assumed deterministic as their values are small compared to the earthquake load, and the envelopes from the NTHA already include their effect.

Variable	Bias	CoV (%)	Distribution
Base plate length	1.0	2.5	Normal
Base plate width	1.0	4	Normal
Base plate thickness	1.0	3	Normal
Embedment depth	1.0	4	Normal
Column properties	1.0	2	Normal
Concrete compressive strength	1.235	14.5	Normal
Yield strength of base plate steel	1.16	7	Normal

Table 2: Summary of random variables for Geometry and Material properties.

The failure is assumed as demand exceeds capacity, given by the following equation:

$$f = M_n / \phi - M \leq 0 \tag{1}$$

where M is the total demand that comes from the PDF obtained in the first component, M_n is the capacity that is a function of the random variables in Table 2 and the uncertain reactions in the connection, and ϕ is a global reduction factor for the EBP. In the previous equation, as only one design is considered, the real capacity is approximated as the ratio of the capacity by the strength reduction factor. The probability of failure is estimated as the total cases where f is smaller than 0 over the total samples. The goal is this study is to obtain a global reduction factor ϕ for a target probability of failure.

3 ANALYSIS OF RESULTS

From the NHTA of the building with different earthquakes, the envelopes of the total shear V, axial P, and moment M in the connection are obtained. Figure 4 shows the histograms for this data for each quantity and exterior and interior columns. It also shows the kernel density estimates of the PDF for each quantity, which are used for the Monte Carlo simulations.

Next, the Monte Carlo simulations are performed using 10^6 samples to obtain a global reduction factor ϕ for a target reliability index β of 3.6, which is standard for connections subjected to gravity and seismic loads [24].

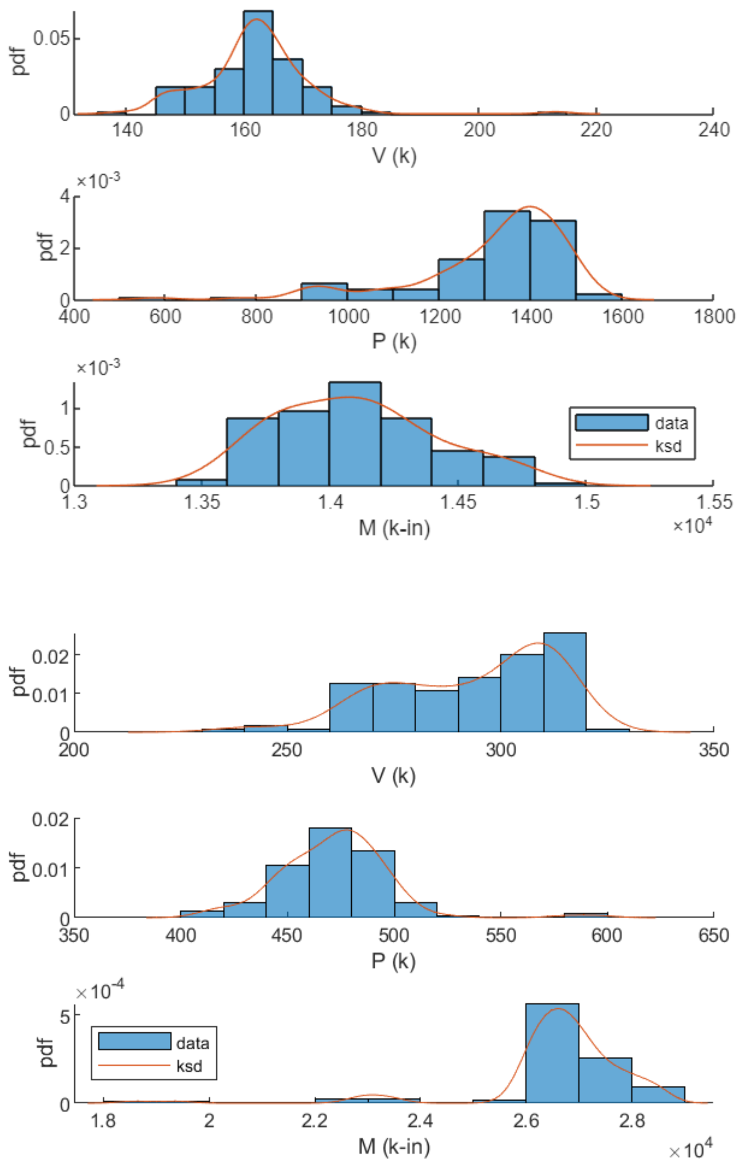

Figure 4: Histograms from the envelopes of V, P, and M in the connection from the NTHA and kernel density estimation PDFs for the a) exterior column and b) interior column

Figure 5 shows the curves of the reliability index vs. strength reduction factor for both types of columns. A median value is estimated from both cases, and as a result, a global reduction factor of 0.4 is recommended in this case. Note that obtained reduction factors are somewhat small due to the fact that the nominal capacities shown in Table 1 are smaller than the results obtained in the NTHA shown in Figure 4; consequently, this uncertainty in the connection load reduces the strength reduction factor.

Further studies are required to incorporate other designs as well as other building characteristics. Also, the obtained reduction factor is a global value, and it does not take into account the different types of failures; therefore, further studies are required to obtain a reduction factor for different contributions, as recommended in the study by Song et al. [24].

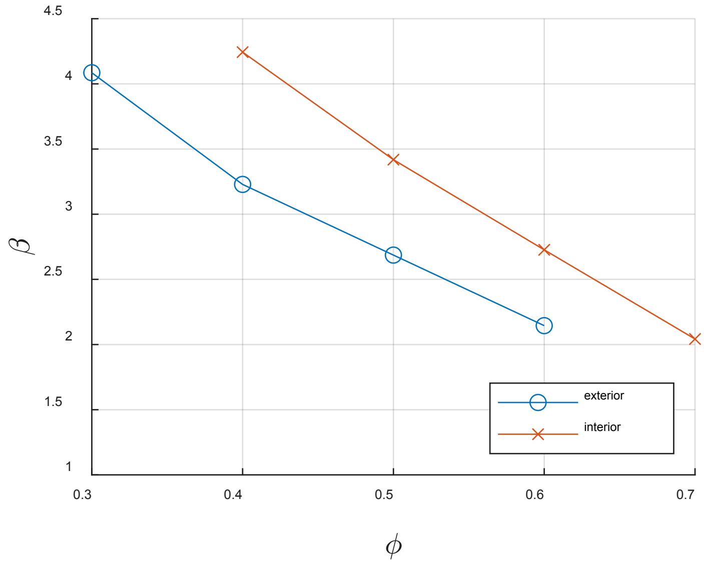

Figure 5: Reliability index vs. strength reduction factor

4 CONCLUSIONS

Column base connections are critical components of Steel Moment Frames (SMFs) since these connections transfer the loads (e.g., gravity, seismic) from the superstructure to the concrete foundation, being an interface between them. Notable large-scale experimental programs have been conducted, and numerical and analytical investigations have complemented these programs. Traditionally, column-base connections are divided into exposed base plates and embedded bases. The former type of connection is mainly used for low and mid-rise buildings, while the latter detail is the norm for tall buildings.

The behavior of embedded base connections has been studied in recent experimental programs. However, surprisingly a reliability analysis of these connections has not been conducted so far. Motivated by this issue, this study presents a reliability analysis of the embedded base connections with the aim of providing design recommendations. For this purpose, nonlinear mathematical models of archetype frames are developed and subjected to nonlinear dynamic analysis to obtain the column base connection demands. From the results of the NTHA, the probability distribution of the loads in the connection is obtained. Then, a Monte Carlo analysis is performed, including these uncertainties as well as material and geometry uncertainties in the connection. A global reduction factor of 0.4 is recommended to achieve a target reliability of 3.60, which is a typical reliability index worldwide accepted for connections.

This study represents the first approximation of the reliability assessment problem. Thus, it must be interpreted with caution once a designer wants to incorporate its results into a design office. First, in this study, only one building was utilized in the NTHA, which limits its scope. More building configurations need to be considered in a further study. Furthermore, in this preliminary study, a global strength reduction factor was proposed herein. Other studies can explore the effect of using different strength reduction factors for each limit state assumed in the design of the connection.

REFERENCES

[1] DeWolf J, Sarislay E. Column Base Plates with Axial Loads and Moments. J Struct Div 1980;106:2167–84.

[2] Astaneh A, Bergsma G, Shen JH. Behavior and design of base plates for gravity, wind and seismic loads. Proc. Natl. steel Constr. Conf., AISC Chicago, IL; 1992, p. 209–14.

[3] Burda JJ. Studies of seismic behavior of steel base plates. University of Nevada, Reno; 1999.

[4] Fahmy M, Stojadinovic B, Goel SC. Analytical and experimental studies on the seismic response of steel column bases. Proc. 8th Can. Conf. Earthq. Eng., 1999.

[5] Drake RM, Elkin SJ. Beam-column base plate design-LRFD method. Eng Journal-American Inst Steel Constr 1999;36:29–38.

[6] Fisher; James M. Base Plate and Anchor Rod Design. AISC Des Ser 2006:73.

[7] Grilli DA, Kanvinde AM. Special moment frame base connection: Design example 8. 2012 IBC SEAOC Struct Des Manual, Examples Steel-Frame Build 2013;4:255–80.

[8] Gomez I, Kanvinde A, Deierlein GG. Exposed Column Base Connections Subjected to Axial Compression and Flexure. AISC, Chicago 2010:257.

[9] Kanvinde AM, Grilli DA, Zareian F. Rotational Stiffness of Exposed Column Base Connections: Experiments and Analytical Models. J Struct Eng 2012;138:549–60. https://doi.org/10.1061/(asce)st.1943-541x.0000495.

[10] Rodas PT, Zareian F, Kanvinde A. Hysteretic Model for Exposed Column–Base Connections. J Struct Eng 2016;142:04016137. https://doi.org/10.1061/(asce)st.1943-541x.0001602.

[11] Grilli DA, Kanvinde AM. Embedded column base connections subjected to seismic loads: Strength model. J Constr Steel Res 2017;129:240–9. https://doi.org/10.1016/j.jcsr.2016.11.014.

[12] Rodas PT, Zareian F, Kanvinde A. Rotational Stiffness of Deeply Embedded Column–Base Connections. J Struct Eng 2017;143:04017064. https://doi.org/10.1061/(asce)st.1943-541x.0001789.

[13] Torres-Rodas P, Zareian F, Kanvinde A. A hysteretic model for the rotational response of embedded column base connections. Soil Dyn Earthq Eng 2018;115. https://doi.org/10.1016/j.soildyn.2018.08.015.

[14] Torres-Rodas P, Zareian F, Kanvinde A. Seismic demands in column base connections of steel moment frames. Earthq Spectra 2018;34:1383–403.

[15] Mora-Bowen N, Torres-Rodas P. Nonlinear Finite Element Models for Embedded Base Connections. In: Eccomas Proceedia, editor. 8th ECCOMAS Themat. Conf. Comput. Methods Struct. Dyn. Earthq. Eng., 8th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering; 2021, p. 4331–40. https://doi.org/10.7712/120121.8789.18586.

[16] Inamasu H, Kanvinde AM, Lignos DG. Seismic design of non-dissipative embedded column base connections. J Constr Steel Res 2021;177.

https://doi.org/10.1016/j.jcsr.2020.106417.

[17] Inamasu H, Lignos DG. Finite element modeling and behavior of dissipative embedded column base connections under cyclic loading. J Constr Steel Res 2022;189:107063.

[18] Torres-Rodas P, Flores F, Zareian F. Seismic response of steel moment frame considering gravity system and column base flexibility. 11th National Conference Earthquake Engineering 2018, NCEE 2018 Integration Science Engineering Policy 2018;12:7383–93.

[19] Torres-Rodas P, Flores F, Astudillo B, Pozo S. Sensitivity of Special Steel Moment Frames to the influence of column-base hysteretic behavior including gravity framing system. Proc. Int. Conf. Struct. Dyn. EURODYN 2 3629-3642, 2020, p. 3629–42. https://doi.org/10.47964/1120.9298.19279.

[20] Falborski T, Torres-Rodas P, Zareian F, Kanvinde A. Effect of Base-Connection Strength and Ductility on the Seismic Performance of Steel Moment-Resisting Frames. J Struct Eng 2020;146:04020054. https://doi.org/10.1061/(asce)st.1943-541x.0002544.

[21] Torres-Rodas P, Flores F, Pozo S, Astudillo BX. Seismic performance of steel moment frames considering the effects of column-base hysteretic behavior and gravity framing system. Soil Dyn Earthq Eng 2021;144:106654. https://doi.org/10.1016/j.soildyn.2021.106654.

[22] Aviram A, Stojadinovic B, Kiureghian A Der. Performance and reliability of exposed column base plate connections for steel moment-resisting frames 2010:190.

[23] Torres-Rodas P, Fayaz J, Zareian F. Strength resistance factors for seismic design of exposed based plate connections in special steel moment resisting frames. Earthq Spectra 2020;36:537–53. https://doi.org/10.1177/8755293019891714.

[24] Song B, Galasso C, Kanvinde A. Reliability Analysis and Design Considerations for Exposed Column Base Plate Connections Subjected to Flexure and Axial Compression. J Struct Eng 2021;147:04020328. https://doi.org/10.1061/(asce)st.1943-541x.0002903.

[25] American Society of Civil Engineers. Minimum design loads for buildings and other structures, American Society of Civil Engineers (ASCE). Reston, VA. 2016.

[26] Campos, Liseth; Torres-Rodas, Pablo;Quinde, Pablo; Yepez F. Influence of earthquake mechanisms to the seismic response of steel moment frames. In: Proceedings C, editor. 8th Int. Conf. Comput. Methods Struct. Dyn. Earthq. Eng. COMPDYN 2021, Athens: 8th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering; 2021. https://doi.org/10.7712/120121.8853.18546.

[27] Miranda, E., Eads, L., Davalos, H., & Lignos D. Assessment of the Probability of Collapse of Structures Under Earthquakes. Proc. 16th World Conf. Earthq. Eng., 2017.

[28] Jalayer F, Cornell CA, Chakroborty S, Roy R, Miranda E, Eads L, et al. Assessment of the probability of collapse of structures during earthquakes. Earthq Eng Struct Dyn 2016;128:122. https://doi.org/10.1115/1.4034725.

[29] Fayaz J, Torres-Rodas P, Medalla M, Naeim F. Assessment of ground motion amplitude scaling using interpretable Gaussian process regression: Application to steel moment frames. Earthq Eng Struct Dyn 2023.

[30] Ibarra LF, Medina RA, Krawinkler H. Hysteretic models that incorporate strength and

stiffness deterioration. Earthq Eng Struct Dyn 2005;34:1489–511. https://doi.org/10.1002/eqe.495.

[31] American Institute of Steel Construction. Specification for Structural Steel Buildings. ANSI/AISC 360-16. Chicago 2016.

[32] Kanvinde AM. Micromechanical simulation of earthquake-induced fracture in steel structures. Stanford University; 2004.

[33] Ancheta TD, Darragh RB, Stewart JP, Seyhan E, Silva WJ, Chiou BS-J, et al. NGA-West2 database. Earthq Spectra 2014;30:989–1005.