RELIABILITY ANALYSIS OF EXPOSED COLUMN BASE PLATE CONNECTIONS SUBJECTED TO SEISMIC LOAD COMBINATIONS

B. Song(1), C. Galasso(2), A. Kanvinde(3)

(1) *PhD Candidate, Dept. of Civil, Environmental & Geomatic Engineering, University College London, UK, biao.song.14@ucl.ac.uk*
(2) *Associate Professor, Dept. of Civil, Environmental & Geomatic Engineering, University College London UK, and Scuola Universitaria Superiore (IUSS) Pavia, Italy, c.galasso@ucl.ac.uk*
(3) *Professor, Dept. of Civil and Environmental Engineering, University of California, Davis, USA, kanvinde@ucdavis.edu*

Abstract

Exposed Column Base Plate (ECBP) connections are commonly used in low- to mid-rise steel moment resisting frames in seismic regions. These connections withstand combinations of applied forces and moments through bearing between the footing and the base plate, in conjunction with axial tension in anchor rods. Current practice for designing ECBP connections in the United States relies on the AISC (American Institute of Steel Construction) *Design Guide One* (DG1). This design approach relies on the load and resistance factor design (LRFD) approach and strength reduction (ϕ-) factors, requiring all components in the connection (i.e., concrete footing, base plate and anchor rods) to be designed against their force/stress demands, separately. This study investigates the implied structural safety of ECBP connections designed as per the DG1 approach, with the objectives of (1) identifying deficiencies of current design method; and (2) proposing insights on possible improvements of the current design approach. A set of 16 ECBP connections is designed (as per the DG1 method) in support of these objectives. The reliability of these design cases is evaluated by first characterizing all the sources of uncertainty (including geometric, material, loading, and modeling uncertainties), formulating the limit-state functions for the specific problem, and then performing Monte Carlo simulations to compute the values of the reliability index (β) with respect to four failure modes. These modes include (1) bearing failure in the footing – *BF*; (2) flexural yielding of base plate on the compression side – *PC*; (3) flexural yielding of base plate on the tension side – *PT*; and (4) anchor rod yielding – *AT*. Finally, some future perspectives for the design of ECBP connections are discussed based on the results of the proposed reliability analysis.

Keywords: exposed column base plate connection; reliability analysis; AISC Design Guide One

1. Introduction and Motivation

Exposed Column Base Plate (ECBP) connections are widely used in low- to mid-rise Steel Moment Resisting Frames (SMRFs) to transfer forces from the entire structure, through the first-story column, into the concrete footing. Fig. 1(a) schematically illustrates an ECBP connection detail commonly used in the United States, and featured in various design guidelines, including the American Institute of Steel Construction (AISC)'s *Design Guide One* [1], as well as the AISC Specification [2], and Seismic Provisions [3]. Referring to Fig. 1, the axial force (P) and moment (M) are transferred through combination of upward bearing stresses (in the grout or supporting concrete foundation) on the compression side of the connection, and downward tensile forces (in the anchor rods) on the tension side of the connection. Shear may be transferred either through friction (if sufficient compression is present), through the anchor rods, or through a shear key, if provided [4]. In the United States, the *Design Guide One* (abbreviated DG1 henceforth) is the primary document guiding the design of ECBP connections, under combinations of axial compression, flexure, and shear. The DG1 relies heavily on the internal stress distributions proposed by Drake and Elkin [5]. Connections that utilize similar details and force transfer mechanisms are used in other regions as well; consequently, they have been studied extensively in various contexts. For instance, Ermopoulos and Stamatopoulos [6] developed closed form analytical solutions to characterize internal force distribution, and work by Gomez et al. [7] and Kanvinde et al. [8] has examined the efficacy of the DG1 method through experiments and finite element simulations respectively. Gomez [9] provides a comprehensive review of literature in this area.

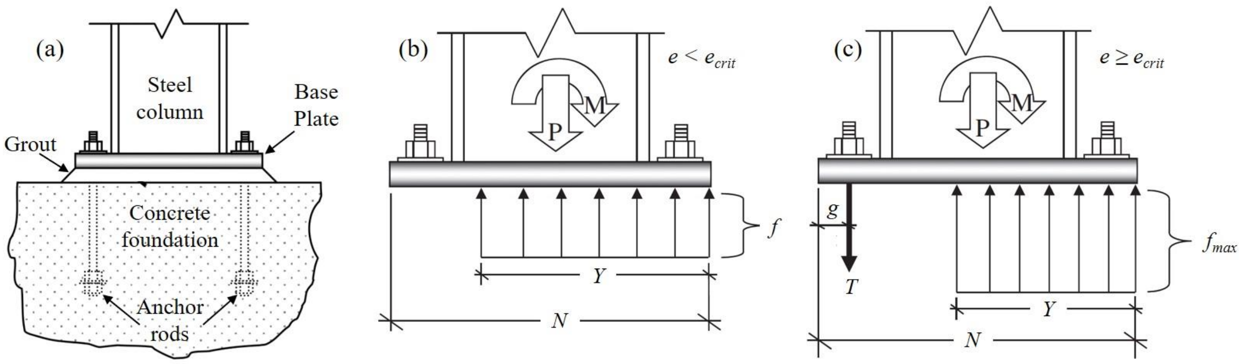

Fig. 1 – (a) Schematic of an exposed column base plate (ECBP) connection and its assumed stress distributions for base connection under (b) low eccentricity and (c) high eccentricity conditions.

These studies largely concur that the Drake and Elkin [5] approach (which underlies the *de facto* design method in the United States through codification in DG1) is effective from a mechanistic standpoint, i.e., it is able to satisfactorily characterize the internal force distribution within ECBP connections in a deterministic sense (Gomez et al., [7]; Kanvinde et al. [8]). However, a closer examination of the method (and associated literature) from a probabilistic standpoint reveal inconsistencies and knowledge gaps that must be addressed to ensure that ECBP connections meet target reliability (i.e., provide acceptable probabilities of failure/safety level). These issues emerge because the approach treats the ECBP connection as a collection of components, each of them designed separately, without considering the interactions between them, or their collective effect on connection failure. Specifically, the approach determines an internal force distribution (i.e., forces in the anchor rods, and bending moment in the base plate on the tension and compression) based on an assumed bearing stress in the concrete/grout (as shown in Figs. 1(b) and (c)), and then applies design checks independently to each of these components (i.e., the anchor rods and base plate) by comparing these estimated forces/moments to their capacities, modified by capacity (i.e., ϕ-) factors. This is problematic for numerous reasons:

- Connection failure is controlled by interactions of these components. Research by Gomez et al. [7] has indicated that flexural yielding of the base plate on the compression side of the connection does not result in connection failure, unless also accompanied by either yielding of the anchor rods, or flexural yielding of the base plate on the tension side. Applying design checks independently to these components entirely disregards this effect, resulting in undue conservatisms.
- In a probabilistic context, applying the design checks independently is inappropriate, because the probability of failing one design check may not necessarily correspond to failure of the entire connection, from a perspective of system reliability.
- The assumed bearing stress in the concrete (used for determination of the internal force distribution) includes a ϕ-factor, to incorporate the uncertainty in this stress. While this may be suitable for design of the concrete footing itself (to provide a conservative estimate of bearing stress), it cannot be justified for design of the other components (i.e., base plate and anchor rods). This is because the bearing stress effectively acts as a "demand" on these other components through overall equilibrium of the connection, such as that a lower estimate of bearing stress may, in fact, be unconservative.
- Finally, the ϕ-factors in the independent design checks for the anchor rods and base plates are borrowed in an *ad hoc* manner from other similar components and are not based on reliability analysis.

Specifically, the design checks consider only the uncertainty in capacity of the components and disregard both the uncertainty as well as bias in the estimated forces and moments in these components.

In summary, it is observed that: (1) while well-intentioned, the DG1 approach fails to effectively incorporate system reliability as well as overall connection response; and (2) given the complex and sometimes counteracting nature of the effects noted above, a consistent connection reliability cannot be ensured.

In response to these issues, this study conducts a detailed reliability analysis of the current DG1 approach for the design of ECBP connections. However, this conference paper only focuses on the first step of the whole study, and the specific objective is to examine the level of connection reliability (quantified by the reliability index β) provided by ECBP connections designed (for seismic load combinations) as per the DG1 approach. The reliability results from this paper may identify deficiencies in the DG1 approach and help to develop possible enhancements. These enhancements will be examined with respect to structural reliability in a manner similar to that used for the current approach.

The paper begins by providing background of the DG1 approach; this is followed by a summary of the methodology used for reliability analysis. A set of 16 design scenarios (SMRF columns for which ECBP connections must be designed) that represent seismic load combinations are then described. For each of these scenarios, ECBP connections are designed as per DG1 approach, and reliability analysis is conducted using Monte Carlo simulations modelling several sources of uncertainty. The paper concludes by providing commentary regarding the current DG1 approach and pointing out the next steps of the whole study.

2. Background of the DG1 Method

Figs. 1(b) and (c) illustrate the key assumptions of the DG1 method. Since this method is well documented in the Design Guide itself and numerous other documents (e.g., Gomez et al. [7]), it is only briefly summarized here. Referring to Figs. 1(b) and (c), the axial compression (P) and moment (M) combination is resisted by: (1) a compression stress block of constant magnitude (f), if the axial force is high relative to moment, i.e., a "low eccentricity" condition; or (2) a compression stress block (with magnitude of $f_{\max}$) supplemented by tension (T) that develops in the anchor rods as the base plate tries to uplift when the axial compression is low compared to the moment, i.e., a "high-eccentricity" condition. The critical value of this load eccentricity (e_{crit}) is determined as:

$$e_{\text{crit}} = \frac{N}{2} - \frac{P}{2 \cdot B \cdot f_{\max}^{\text{DG1}}} \tag{1}$$

where, the terms P, B and N denote the applied axial load, length and width of base plate. The above equation assumes that the bearing side of the connection develops a rectangular stress block with a constant magnitude $f_{\max}^{\text{DG1}}$ (the superscript indicates the design method used), determined as $\phi_{\text{bearing}} \times \min(f_{\text{grout}}, f_{\text{concrete}})$, where ϕ_{bearing}-factor is taken as 0.65, f_{grout} is the crushing strength of the grout, and f_{concrete} may be determined as below, accounting also for the effects of concrete confinement (if the concrete foundation is larger than the size of base plate):

$$f_{\text{concrete}} = 0.85 \times f_c' \times \sqrt{\frac{A_2}{A_1}} \le 1.7 \times f_c' \tag{2}$$

In the above equation, f_c' is the compressive strength of the concrete, A_1 is the bearing area of the plate, and A_2 is the effective area of the concrete (typically the plan area of the footing). The grout pad is typically not confined similarly, since it is above the concrete surface. Thus, a similar adjustment is not required for the grout strength.

If the loading condition is determined to be low eccentricity, i.e., the design load eccentricity e (= M/P) < e_{crit}, the magnitude of the upward bearing stresses f, as well as the stress block length Y^{DG1} (Fig. 1(b)) may be easily calculated through force and moment equilibrium with respect to the applied forces. If a suitable solution cannot be found with $f < f_{max}^{DG1}$ and $Y^{DG1} < N$, then the base plate plan dimensions must be resized. The concrete/grout bearing failure check is applied implicitly in this manner by assuming that bearing failure occurs if an equilibrium solution cannot be found with $f < f_{max}^{DG1}$ and $Y^{DG1} < N$. This design check is denoted *BF* (representing the Bearing Failure limit state) to facilitate subsequent discussion of the reliability analysis. For the low-eccentricity condition, the only other mode of failure is flexural yielding of the base plate on the compression side due to bearing stresses; this is usually done by assuming that the toe of the base plate bends upwards as a cantilever flap, with a yield line parallel to the edge of the column flange. This design check is denoted *PC* (Plate failure on the Compression side). Specifically, failure is assumed to occur if the cantilever moment over the yield line exceeds the reliable capacity of the base plate $\phi_{plate} \times M_p^{plate}$ $(= F_{y,pl} \cdot t_p^2/4$, i.e., plastic moment capacity of the base plate, $F_{y,pl}$ is the yield strength of base plate steel and t_p is the thickness of base plate), where $\phi_{plate} = 0.9$. Referring to the introductory discussion, it is pertinent to note here that the bearing stress (which acts as a "load" on the cantilever flap) includes the $\phi_{bearing}$-factor, thereby reducing flexural demands on the base plate for the design check.

If $e \geq e_{crit}$, i.e. the "high eccentricity" condition (Fig. 1(c)), then the stress in the bearing zone reaches its maximum value (i.e., f_{max}^{DG1}), such that the two remaining unknowns, i.e., the stress block length Y^{DG1} as well as the tension forces in the anchor rods T^{DG1} may again be calculated from force and moment equilibrium; the resulting equations are below:

$$Y^{DG1} = (N-g) - \sqrt{(N-g)^2 - \frac{2\cdot\left[M + P\cdot\left(\frac{N}{2} - g\right)\right]}{f_{max}^{DG1}\cdot B}} \tag{3}$$

$$T^{DG1} = f_{max}^{DG1}\cdot B\cdot\left((N-g) - \sqrt{(N-g)^2 - \frac{2\cdot\left[M + P\cdot\left(\frac{N}{2} - g\right)\right]}{f_{max}^{DG1}\cdot B}}\right) - P \tag{4}$$

This results in four possible limit states, and associated design checks. Similar to the low-eccentricity case, the *BF* design check is applied implicitly, such that failure is assumed to occur if $Y^{DG1} > N\text{-}g$ (g is the distance between the center of the anchor rods to the edge of the base plate), which indicates that the bearing zone extends into the tension anchor rods (which is impossible from a compatibility standpoint). For the base plate, two limit states are possible: (1) the *PC* limit state due to upward bearing on the compression side, and (2) flexural yielding of the base plate on the tension side due to downward tension forces in the anchor rods; this is denoted *PT*, and evaluated by comparing the moment in the plate due to the anchor forces T^{DG1} and the reliable capacity $\phi_{plate} \times M_p^{plate}$. For the *PT* limit state, the controlling mechanism may involve either a yield line parallel to the column flange or inclined to the plate edge, depending on the location of the anchor rods. The final limit state is the yielding of the anchor rods themselves, which is determined to occur if $T^{DG1}/n_{rod} > \phi_{rod} \times 0.75 \times F_u^{rod} \times A_{rod}$ (where n_{rod} is the number of anchor rods in a line, F_u^{rod} is the ultimate strength of the rod, and A_{rod} is the unthreaded area of anchor rod) – this is denoted *AT*. Other anchor limit states may include pullout of the rods or blowout of the concrete. These depend on the footing configuration and reinforcement, and are outside the scope of this article; ACI 318 [10] provides greater detail.

It is relevant to note that each of the design checks outlined above include $f_{\text{max}}^{\text{DG1}}$, and consequently ϕ_{bearing}, which is used to estimate it. For the *PC* check, the non-conservatism is readily apparent because ϕ_{bearing} reduces the moment demand on the plate. The effect of ϕ_{bearing} on the other limit states is not as straightforward (e.g., see Eqs. (3) and (4)). Nonetheless, it is evident that for the same reasons as for the *PC* check, incorporating ϕ_{bearing} within the design checks is not appropriate, and is likely to result in biased or inaccurate characterizations of reliability. Finally, as discussed earlier, the ϕ_{plate} and ϕ_{rod} do not consider either the accuracy of the demand estimation or the variability within it – which is also inappropriate from the perspective of estimating reliability.

The next section outlines a process for estimating the reliability of ECBP connections that addresses these various issues, before applying it to the current DG1 design approach.

3. Methodology and Case Study Definition

This section is organized to present the methodology and its various steps, which are used to evaluate the structural reliability (i.e., probability of failure or safety level) of ECBP connections for which the nominal configurations (i.e., geometry, material properties), and the loadings are known. Specifically, the main steps involved in this assessment (illustrated in Fig. 2) are:

- Developing a set of representative design cases designed as per the DG1 method, in terms of various dimensions and configurational aspects of ECBP connections. This step requires to determine a range of applied moment (M) and axial forces (P) combinations at column bases where ECBP connections are to be designed;
- Identifying sources of uncertainty (i.e., random variables, RVs) in each designed configuration and characterizing the statistics of them;
- Formulating limit-state functions associated with each failure mode, i.e., *BF*, *PC*, *PT*, and *AT*, for each designed configuration;
- Performing Monte-Carlo simulation that utilizes the statistical distributions of input RVs to assess the probability of failure (P_f) and reliability indices (β) of each design configuration.

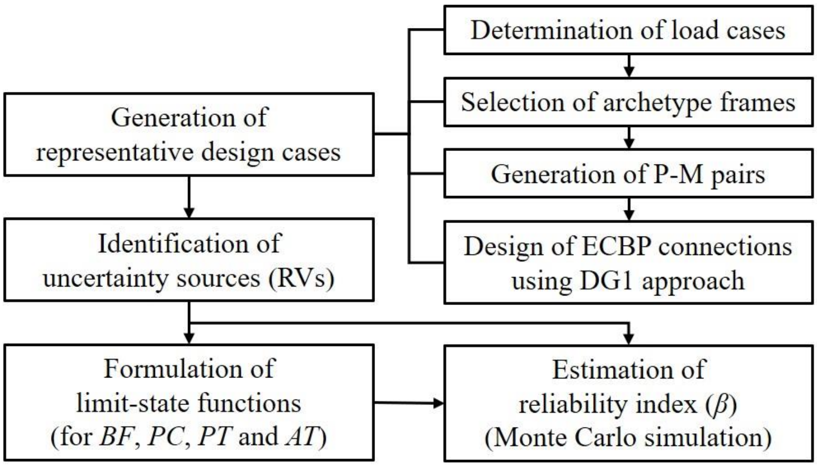

Fig. 2 – Flowchart of the methodology for reliability assessment of ECBP connections.

3.1 Generation of case-study design cases

The design condition for ECBP connections is defined by a combination of moment (*M*) and axial force (*P*). To ensure realism in these *P-M* load pairs, these are not arbitrary generated, but rather derived from four archetype steel moment frames (consisting of four stories and three bays). These designs are selected from an archetype set of special steel moment frames developed by NEHRP [11]; only key details are provided here. Fig. 3 illustrates the dimensions and floorplans, whereas Table 1 summarizes the member properties. The key differences between the frames are the level of seismicity they are designed for (in accordance with Seismic Design Category, SDC, i.e., either SDC D_{max} or SDC D_{min}) and the method used to design them (the Response Spectrum Analysis, RSA; or the Equivalent Lateral Force, ELF). Four-story frames are selected for the representative load case for because taller frames usually warrant embedded base connections, whereas 1-2 story frames often assume ECBP connections to be pinned. The *P-M* pairs are derived from these frames based on the following considerations.

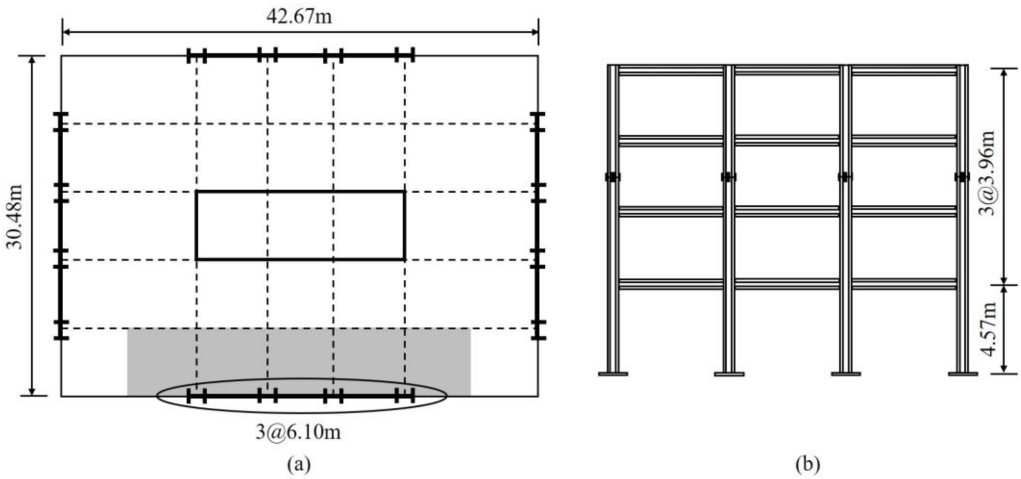

Fig. 3 – Schematic illustration of 4-story archetype frames: (a) plan view; (b) elevation.

For each frame, Dead (*D*), Live (*L*), and Earthquake (*E*) loads are determined from the applicable codes, i.e., ASCE 7-16 [12]. The corresponding *P* and *M* values at each of the column base locations in each building are recovered (to recover the earthquake loads, a non-near fault location in the Los Angeles area is assumed); these were subsequently used to generate *P-M* pairs based on the load combinations indicated in Table 2. These load combinations include those prescribed by ASCE 7-16 [12], as well as others that are informed by recent research and other standard practices. For example, Torres-Rodas et al. [13] indicate that the minimum compressive axial force (rather than maximum) may control the design of some column bases, since a lower compression increases tension in the rods. The load factor $-\Omega_0 E$ (in which Ω_0 represents the "overstrength" seismic load) reflects the overturning effect that minimizes axial compression. The factor of $1.1R_yM_p$ in the load cases reflects a capacity design (AISC 341-16 [3]) which is often specified in high-seismic zones to induce plastic deformations in the attached column. Referring to Table 2, the exterior and interior base connections within each frame are considered for design separately. This results in the generation of 16 *P-M* pairs (considering only seismic load combinations) for which ECBP connections must be designed. Table 2 summarizes the design results of these ECBP connections (as per DG1 approach). It is worth noting that 15 (out of 16) cases are designed with high-eccentricity conditions due to relatively high moment (comparing to their axial forces), and the only exception is case 6, which is designed for low-eccentricity condition.

Table 1 – Member sizes of 4-story archetype frames.

Story	RSA SDC-D$_{max}$ frame			RSA SDC-D$_{min}$ frame			ELF SDC-D$_{max}$ frame			ELF SDC-D$_{min}$ frame		
	Beam	Exterior column	Interior column	Beam	Exterior column	Interior column	Beam	Exterior column	Interior column	Beam	Exterior column	Interior column
1	W21×73	W24×103	W24×103	W16×57	W14×74	W14×82	W24×103	W24×103	W24×131	W18×71	W18×86	W18×97
2	W21×73	W24×103	W24×103	W18×60	W14×74	W14×82	W24×103	W24×103	W24×131	W18×86	W18×86	W18×97
3	W21×57	W24×62	W24×62	W18×60	W14×48	W14×74	W24×76	W24×76	W24×84	W18×71	W18×65	W18×86
4	W21×57	W24×62	W24×62	W16×57	W14×48	W14×74	W24×76	W24×76	W24×84	W18×71	W18×65	W18×86

Table 2 – Design configurations and reliability results of representative ECBP connections designed as per DG1 method.

Design cases	Frame	First-story column section	Load combination[a]	Design configurations						Reliability index			
				N (mm)	B (mm)	t_p (mm)	Anchor rod grade[b]	n_{rod}	d_{rod} (mm)	β_{BF}	β_{PC}	β_{PT}	β_{AT}
1	RSA SDC-D$_{max}$	Exterior (W24×103)	$-E$	965	558	83	105	5	38	5.3	1.7	1.9	2.6
2			$+E$			89	55	4	32	4.4	1.2	4.1	1.7
3		Interior (W24×103)	$-E$	965	558	89	105	5	32	5.0	1.7	4.2	2.2
4			$+E$			89	105	5	32	4.9	1.6	4.3	2.4
5	RSA SDC-D$_{min}$	Exterior (W14×74)	$-E$	711	610	70	105	5	32	5.6	1.9	2.7	2.8
6			$+E$			83	55	2	19	4.4	1.6	5.4	2.2
7		Interior (W14×82)	$-E$	711	610	83	105	2	38	5.1	1.7	5.5	2.1
8			$+E$			89	55	3	38	5.0	2.3	5.6	2.1
9	ELF SDC-D$_{max}$	Exterior (W24×103)	$-E$	965	558	83	105	5	38	5.3	1.8	1.8	2.5
10			$+E$			89	55	4	32	4.3	1.2	4.1	1.7
11		Interior (W24×131)	$-E$	965	660	89	105	5	38	4.8	1.2	3.3	2.3
12			$+E$			89	105	5	38	4.9	1.2	3.3	2.3
13	ELF SDC-D$_{min}$	Exterior (W18×86)	$-E$	813	610	76	105	4	38	5.6	2.0	2.7	2.4
14			$+E$			89	105	2	19	4.4	1.4	5.1	0.9
15		Interior (W18×97)	$-E$	813	610	89	105	3	38	4.9	2.0	4.9	2.1
16			$+E$			89	105	4	32	4.9	2.0	4.9	1.9

[a]"-E" load combinations: $P = 1.2D + 0.5L - \Omega_0 E$ & $M = 1.1R_yM_p$; "+E" load combinations: $P = 1.2D + 0.5L + \Omega_0 E$ & $M = 1.1R_yM_p$;
[b]The anchor rod material is selected from two available grades of ASTM F1554 steel: Grade 55 and Grade 105.

3.2 Characterization of Uncertainty

Reliability analysis of each designed ECBP connection requires the characterization of uncertainty that arises from four main sources: (1) geometry of each component; (2) material properties of each component; (3) applied loads on the connection; and (4) mechanical models used to characterize the demand and capacity of each component. Table 3 summarizes a list of all the uncertainties (from four groups discussed above) used for Monte Carlo simulation (discussed later). They are defined as RVs with their statistics, i.e., statistical distributions, bias coefficient (i.e., the ratio between the mean value of each RV to its nominal value as specified in the design cases), and the Coefficient of variation (CoV, defined as the ratio between the standard deviation of each RV to its mean value). All RVs are considered as stochastically independent.

Table 3 – Summary of random variables for reliability analysis of ECBP connection.

Category	Variable	Bias	CoV (%)	Distribution	Reference
Geometry	Overall depth of base column section, d	0.999	0.2	Normal	Schmidt and Bartlett [14]
	Flange width of base column section, b_f	0.998	0.4	Normal	
	Flange thickness of base column section, t_f	1.04	2.5	Normal	
	Web thickness of base column section, t_w	1.04	2.5	Normal	
	Base plate length, N	1	2.5	Normal	Aviram et al. [15]
	Base plate width, B	1	4	Normal	
	Base plate thickness, t_p	1	3	Normal	
	Anchor rod diameter, d_{rod}	1	8.5	Normal	
	Edge distance, g	1	5	Normal	
Material	Concrete compressive strength, f_c'	1.235	14.5	Normal	Nowak and Szerszen [16]
	Ratio of expected to specified minimum yield strength of W-shaped column steel (ASTM A992), R_y (with a nominal value = 1.1)	1.0	5	Normal	Liu et al. [17]
	Yield strength of base plate steel (ASTM A572 Grade 50), $F_{y,pl}$	1.16	7	Normal	
	Tensile (ultimate) strength of anchor rod steel, F_u				
	ASTM F1554 Grade 55	1.13	11.76	Lognormal	Aviram et al. [15]
	ASTM F1554 Grade 105	1.1	9.09	Lognormal	
Load	Dead load, D	1.05	10	Normal	Ellingwood et al. [18]
	Live load, L	1	25	Gumbel	
	Wind load, W	0.78	37	Gumbel	
	Earthquake load, E	1	10	Lognormal	Assumed
Model	Ratio of concrete bearing stress to concrete compressive strength, $f_{\max}^{\text{test}} / f_c^{'}$	1.07	15.67	Normal	Hawkins [19]
	Error in characterization of flexural demand of base plate on the compression side, $M_{\text{pl,comp}}^{\text{true}} / M_{\text{pl,comp}}^{\text{DG1}}$	0.88	18.98	Normal	Kanvinde et al. [8]
	Error in characterization of flexural demand of base plate on the tension side, $M_{\text{pl,ten}}^{\text{true}} / M_{\text{pl,ten}}^{\text{DG1}}$	0.99	12.38	Normal	
	Error in Characterization of tension demand in anchor rods, $T_{\text{rods}}^{\text{true}} / T_{\text{rods}}^{\text{DG1}}$	0.99	12.38	Normal	

3.3 Formulation of limit-state functions

As discussed in the previous section, four failure modes of ECBP connections subjected to combined flexural and axial loadings have been identified: (1) bearing failure in the footing – *BF*, (2) flexural yielding of base plate on the compression side – *PC*, (3) flexural yielding of base plate on the tension side – *PT*, and (4) anchor rod yielding – *AT*. For each of these, it is important to define the conditions that lead to failure.

Commonly, this is done in the form of limit-state functions (G) that may be defined as the difference between the capacity (C) and counterpart demand (D):

$$G = C - D \tag{5}$$

Failure of each component occurs when demand exceeds capacity, i.e., $G < 0$. Following this, the limit-state functions of three of the four individual failure modes (PC, PT, AT) may be formulated as below:

$$G_{PC} = C_{PC} - D_{PC} = M_p^{\text{plate}} - M_{PC,\ \text{DG1 model}}^{\text{plate}} \tag{6}$$

$$G_{PT} = C_{PT} - D_{PT} = M_p^{\text{plate}} - M_{PT,\ \text{DG1 model}}^{\text{plate}} \tag{7}$$

$$G_{PT} = C_{PT} - D_{PT} = n_{\text{rod}} \times 0.75 \times F_u^{\text{rod}} \times A_{\text{rod}} - T_{AT,\ \text{DG1 model}}^{\text{rod}} \tag{8}$$

In the above, the subscript "DG1 model" indicates the limit-state functions are developed for the DG1 approach (as discussed in Section 2). For BF, the limit-state function cannot be formulated in one equation because it is assumed to occur when the concrete bearing stress (f) and bearing width (Y^{DG1}) required to resist the applied P-M combination, violates one of the following conditions:

$$f > f_{\max}^{\text{DG1 model}} \tag{9a}$$

$$\text{or} \quad Y^{\text{DG1}} > N - g \tag{9b}$$

The former (Eq. (9a)) enforces the condition that the maximum stress is limited by the bearing capacity of the concrete/grout, whereas the latter (Eq. (9b)) disallows the unphysical development of a zone of compression in the concrete under the tensile anchor rods.

3.4 Monte Carlo simulation and reliability assessment

For each of the design cases, a plain Monte Carlo sampling procedure is used to simulate the demands (D) and capacities (C) of each failure mode described above. This Monte Carlo sampling procedure is based on an *ad hoc* MATLAB program developed by the authors, which is used to estimate the probability of the limit-state functions formulated above being negative, i.e., probability of failure (P_f). A total of 10^8 samples (of the RV sets with statistics listed in Table 3) are randomly generated, and then P_f of each failure mode is estimated through a one-by-one check of G in each simulation:

$$P_f = \frac{\text{Number of } G < 0}{\text{Total Number of Samples } \left(=10^8\right)} \tag{10}$$

Then, a commonly used measure of reliability, known as the reliability index β, is adopted to present the results. If G follows a marginal normal distribution, it is related to P_f via the standard normal cumulative distribution function Φ:

$$P_f = \Phi(-\beta) \Leftrightarrow \beta = -\Phi^{-1}\left(P_f\right) \tag{11}$$

Even though G does not have a normal distribution, the failure probabilities can be still converted to β through Eq. (11), as it is a common way to compare safety levels. The 10^8 samples used in this study allows for a reliable estimation of the β-value up to 5.6. This above reliability assessment process is then applied to the case-study generated by the ECBP connections designed as per the DG1 process.

4. Results and Discussion

The methodology discussed in the previous section is applied to analyze the structural reliability of the 16 ECBP connections designed as per the DG1 method. Table 2 above summarizes the design scenarios and individual design results (i.e., design layout and configuration of each ECBP connection). The reliability indices in terms of four individual failure modes are reported in the same table, and denoted as β_{BF}, β_{PC}, β_{PT}, and β_{AT}, respectively. Fig. 4 also presents the histograms corresponding to the average β-values of all the four considered failure modes (i.e., height of each histogram), error bar (of 95% fractile) about each bar is also displayed.

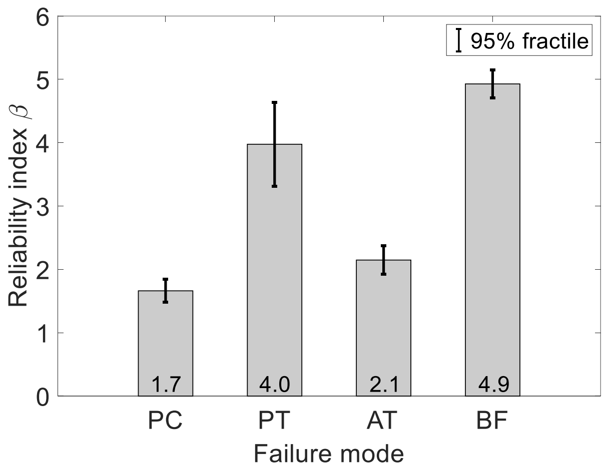

Fig. 4 – Average reliability index (β-) values of four failure modes.

In general, a reliability index β = 4.0 (or sometimes 4,5) is traditionally assumed to reflect an adequate margin of safety for steel connections. According to Table 2 and Fig.4, several commentaries may be made:

- The safety level of ECBP connection against the concrete/grout bearing failure (*BF*) is adequate. The average value of β_{BF} is 4.9 and all the individual β_{BF}-values are no less than 4.3.
- As expected, the mean value of β_{PC} (= 1.7) is considerably low, indicating unacceptably high probability of failure (P_f = 4.8%). This may be mainly attributed to two issues existing in the DG1 method: (1) for high-eccentricity design condition, the use of ϕ_{bearing}-factor (= 0.65) may significantly underestimate the imposed load (demand) under the compression side of the base plate; (2) the value of ϕ_{plate} (= 0.9) assigned in the DG1 is relatively high, leading to insufficient flexural resistance designed for the plate. As per the DG1 method, these two issues may finally result in an unsafe design with respect to the thickness of the base plate.
- The average β_{PT}-value shown in Fig. 4 is 4.0, which reflects an acceptable safety margin of ECBP connection with respect to the flexural yielding failure on the tension side of the base plate. For each individual design case, β_{PT}-value is higher than its β_{PC} counterpart. from the compression side. Because the thickness (t_p) of base plate is designed based upon the larger value computed from both bearing and tension interfaces, and usually this value is obtained from the compression side. This is also the reason

for the relatively significant variation observed in the β_{PT} (see Fig. 4), i.e., the reliability of *PT* failure mode, depending largely on the design check for *PC*, cannot be assessed directly.

- Similar to the results of β_{PC}, the average value of β_{AT} (= 2.1) is fairly low, indicating relatively low safety level. This may suggest a re-examination of current ϕ_{rod}-factor used in the DG1 approach.

5. Conclusions

This preliminary study examines the level of component reliability (in terms of the reliability index, β) guaranteed by ECBP connections designed as per the DG1 approach and seismic loading combinations, which are currently used in the United States. To fulfil this, a total of 16 representative ECBP connections are designed accordingly, their reliabilities with respect to four modes of failure (i.e., bearing failure in the footing – *BF*; flexural yielding of base plate on the compression side – *PC*; flexural yielding of base plate on the tension side – *PT*; and anchor rod yielding – *AT*) are then estimated using methodology proposed in this study and Monte Carlo simulation. Reliability results of this study show that:

- The safety margin of *BF* failure mode is adequate if the ECBP connection design is carried out as per the DG1 method to size the plan dimensions of base plate. It seems that no further investigation on concrete/grout bearing failure is required.
- The reliabilities of *PC* and *AT* failure modes are unacceptably low. It may be attributed to the deficiencies of current design approach, such as the use of ϕ_{bearing}-factor to estimate the moment demand on the plate and the lack of calibration for ϕ_{plate}- and ϕ_{rod}-factors to consider the accuracy of demand estimation (or the variability within it).

As a result, the ongoing work by the authors is aiming at further identifying the deficiencies in the DG1 approach and examine possible enhancements that are based on considering system response. The developed enhancements and the corresponding reliability levels provided by them can be analyzed in manner similar to that used in this study. Finally, prospective design strategies for the safer design of ECBP connections may be suggested.

6. Acknowledgement

The first author would like to acknowledge the financial support from China Scholarship Council (CSC) and University College London (UCL) through a joint research scholarship.

7. References

[1] Fisher JM, Kloiber LA. *Design Guide 1: Base Plate and Anchor Rod Design (Second Edition)*. 2nd ed. Chicago, IL: American Institute of Steel Construction; 2006.

[2] American Institute of Steel Construction (AISC). *Specification for Structural Steel Buildings (ANSI/AISC 360-16)*. Chicago, IL: American Institute of Steel Construction; 2016.

[3] American Institute of Steel Construction (AISC). *Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-16)*. Chicago, IL: American Institute of Steel Construction; 2016.

[4] Gomez IR, Kanvinde AM, Deierlein GG. Experimental Investigation of Shear Transfer in Exposed Column Base Connections. *Engineering Journal* 2011; **48**(4): 245–264.

[5] Drake RM, Elkin SJ. Beam-Column Base Plate Design--LRFD Method. *Engineering Journal* 1999; **36**(1): 29–38.

[6] Ermopoulos JC, Stamatopoulos GN. Mathematical modelling of column base plate connections. *Journal of Constructional Steel Research* 1996; **36**(2): 79–100.

[7] Gomez I, Kanvinde A, Deierlein G. *Exposed Column Base Connections Subjected to Axial Compression and Flexure*. Chicago, IL: 2010.

[8] Kanvinde AM, Jordan SJ, Cooke RJ. Exposed column base plate connections in moment frames — Simulations and behavioral insights. *Journal of Constructional Steel Research* 2013; **84**: 82–93.

[9] Gomez IR. Behavior and Design of Column Base Connections. PhD Thesis. University of California, Davis, 2010.

[10] American Concrete Institute (ACI) Committee 318. *Building Code Requirements for Structural Concrete and Commentary (ACI 318-19)*. Farmington Hills, MI: American Concrete Institute; 2019.

[11] National Earthquake Hazards Reduction Program (NEHRP). *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors, NIST GCR 10-917-8*. NEHRP Consultants Joint Venture; 2010.

[12] American Society of Civil Engineers (ASCE). *Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-16)*. Reston, VA: American Society of Civil Engineers; 2016.

[13] Torres-Rodas P, Zareian F, Kanvinde A. Seismic Demands in Column Base Connections of Steel Moment Frames. *Earthquake Spectra* 2018; **34**(3): 1383–1403.

[14] Schmidt BJ, Bartlett FM. Review of resistance factor for steel: data collection. *Canadian Journal of Civil Engineering* 2002; **29**(1): 98–108.

[15] Aviram A, Stojadinovic B, Kiureghian A Der. *Performance and Reliability of Exposed Column Base Plate Connections for Steel Moment-Resisting Frames. PEER Report 2010/107*. Berkeley, CA: Pacific Earthquake Engineering Research Center; 2010.

[16] Nowak AS, Szerszen MM. Calibration of Design Code for Buildings (ACI 318): Part 1—Statistical Models for Resistance. *ACI Structural Journal* 2003; **100**(3).

[17] Liu J, Sabelli R, Brockenbrough RL, Fraser TP. Expected Yield Stress and Tensile Strength Ratios for Determination of Expected Member Capacity in the 2005 AISC Seismic Provisions. *Engineering Journal* 2007; **44**(1): 15–26.

[18] Ellingwood BR, Galambos T, MacGregor J, Cornell C. *Development of a probability based load criterion for American national standard A58 building code requirements for minimum design loads in buildings and other structures, Special Publication 577*. Washington, D.C.: US Department of Commerce, National Bureau of Standards; 1980.

[19] Hawkins NM. The bearing strength of concrete loaded through rigid plates. *Magazine of Concrete Research* 1968; **20**(62): 31–40.

