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RELIABILITY OF EXPOSED COLUMN-BASE PLATE CONNECTION IN SPECIAL MOMENT-RESISTING FRAMES

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ABSTRACT :

Many steel moment-resisting frame buildings suffered failure at their column base connections during the 1995 Kobe, 1994 Northridge and 1989 Loma Prieta earthquakes. System reliability analysis of an exposed moment-resisting base plate connection designed for a low-rise steel special moment resisting frame is carried out using a structural reliability analysis software. Modes of failure of the column base are defined using a limit-state formulation based on the AISC Design Guide No. 1-2005. The predominant failure modes of the exposed column base include: yielding of the base plate on the compression side, crushing of concrete, and shear failure due to sliding of the base plate and bearing failure of the shear lugs against the adjacent concrete. Sensitivity analysis is carried out to determine the influences of limit-state and distribution parameters on the reliability of the system. On the demand side, the cantilever length of the base plate extending beyond the column cross section and the bending moment at the column base are found to be the main parameters influencing the failure of the column base connection. On the capacity side, the thickness of the base plate and the strength of steel are the main parameters influencing the reliability of the connection. Fragility curves are developed for each failure mode of the column base plate as well as for the connection as a system. These are expressed as a function of the spectral acceleration at the first mode period of the building.

KEYWORDS: steel base plate connection, reliability analysis, moment resisting frame

1. INTRODUCTION

A typical column-base connection between the column of a steel moment-resisting frame (MRF) and its concrete foundation, commonly used in US steel construction today, consists of an exposed steel base plate supported on unreinforced grout and secured to the concrete foundation using steel anchor bolts. This moment-resisting connection is generally subjected to a combination of high bending moments, axial and shear forces. A number of steel buildings, particularly low-rise moment resisting frame systems, developed failure at the column-base plate connection during the 1995 Kobe, 1994 Northridge and 1989 Loma Prieta earthquakes. It was found (Bertero et.al, 1994; Youssef et.al, 1995) that the rotational stiffness and strength of the base plate assemblages affected the damage these structures suffered not only directly in the column bases, but also in other regions of their lateral load resisting frames.

A number of methodologies for the design of column-base plate connections under various load conditions are found in the literature. The most recent method presented in the AISC Design Guide No. 1-2005 (Fisher and Kloiber, 2005) is already widely implemented in current US engineering practice.

Reliability analysis of a column base connection in a MRF, obtained using the AISC Design Guide No. 1-2005 procedure, has not been carried out to date. Yet, such reliability analysis is needed to assess the safety of this important structural component with respect to its diverse failure modes and to evaluate the adequacy of the design method and limit-state formulation. A sensitivity analysis of the different components of the column base connection is needed to identify the critical parameters in the design process. These issues are the focus of the present paper.

2. RELIABILITY ANALYSIS

Seismic design of an exposed column base connection in a typical low-rise moment resisting frame is carried out in the US following the AISC Design Guide No.1-2005 procedure. In this paper, the column base connection of an exterior column of the ATC-58 3 story-3 bay MRF office building, which is located on the University of California at Berkeley campus (Yang et.al, 2006), is used as an example. This connection is shown in Figure 1.

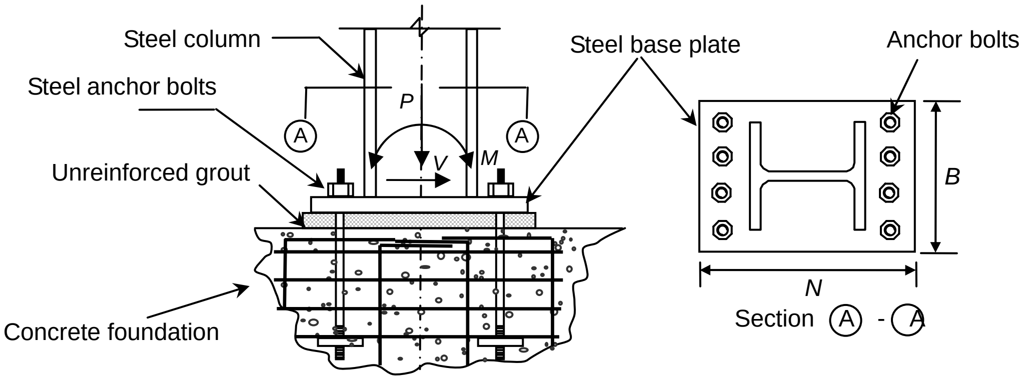

Figure 1 Configuration of a typical exposed base plate connection

The loads used for the design of the connection are obtained from a series of nonlinear time history analysis (NL THA) of the MRF model with fixed column bases. The median values of the joint reactions obtained from a suite of 7 ground motions corresponding to the design earthquake hazard level (10% in 50 years probability of exceedance (PE)) are used to design the connection. Several load combinations from each NL THA are considered to find the critical load combination.

1.1. Random Variables

The following table summarizes the random variables (RVs) used in the reliability analysis of this column base connection. These RVs and their distributions represent different column base components and parameters that influence the behavior of the selected column base connection at different hazard levels defined for a site in Berkeley, California

Table 1 Summary of column base connection random variables

RV	Description	Distribution	μ-Mean	Units	c.o.v.	Reference/Source
Dimensions						
d_c	Column depth	Normal	26.02	in	0.01	ASTM A6-05
b_f	Column flange width	Normal	13.11	in	0.01	ASTM A6-05
N	Base plate length	Normal	38.0	in	0.025	ASTM A6-05
B	Base plate width	Normal	25.0	in	0.040	ASTM A6-05
t_{PL}	Base plate thickness	Normal	3.75	in	0.03	ASTM A6-05
l_{sl}	Shear lug depth	Beta	3.5	in	0.15	ASTM A6-05
b_{sl}	Shear lug length	Normal	25.0	in	0.025	ASTM A6-05
d_b	Anchor bolt diameter	Normal	2.0	in	0.05	ASTM F1554-04
d_{edge}	Edge distance from bolt centerline	Normal	3.0	in	0.085	AISC-Code Standard Practice, 2000
t_g	Grout thickness	Beta	2.0	in	0.25	AISC-Code Standard Practice, 2000

Table 1 Continued…

RV	Description	Distribution	μ-Mean	Units	c.o.v.	Reference/Source
Material Strength						
$F_{y,PL}$	Base plate steel yield stress, Gr. 36	Lognormal	50	ksi	0.07	ASTM A992-04; Liu, 2003
F_{ub}	Anchor bolt ultimate stress, Gr. 105	Lognormal	137.5	ksi	0.10	ASTM F1554-04
f'_c	Concrete compressive strength (4 ksi)	Lognormal	4.8	ksi	0.15	MacGregor, 2005
Coefficients						
μ	Friction coefficient	Beta	0.80	-	0.30	Fisher and Kloiber, 2005
Loads						
High hazard level- Collapse Prevention (2% in 50 yr PE): Case M_{max}						
P_m	Seismic axial load (including gravity)	Lognormal	432.6	kips	0.07	NL THA
V_m	Seismic shear force	Lognormal	241.6	kips	0.13	NL THA
M_{max}	Seismic bending moment	Gumbel	35664.2	kip-in	0.08	NL THA
High hazard level- Life Safety (5% in 50 yr PE) : Case M_{max}						
P_m	Seismic axial load (including gravity)	Lognormal	430.6	kips	0.01	NL THA
V_m	Seismic shear force	Gumbel	218.4	kips	0.09	NL THA
M_{max}	Seismic bending moment	Lognormal	32466.8	kip-in	0.03	NL THA
Moderate hazard level- Immediate Occupancy (10% in 50 yr PE): Case M_{max}						
P_m	Seismic axial load (including gravity)	Lognormal	428.8	kips	0.01	NL THA
V_m	Seismic shear force	Gumbel	206.5	kips	0.08	NL THA
M_{max}	Seismic bending moment	Lognormal	30568.7	kip-in	0.09	NL THA
Low hazard level- Operational (50% in 50 yr PE): Case M_{max}						
P_m	Seismic axial load (including gravity)	Lognormal	354.4	kips	0.08	NL THA
V_m	Seismic shear force	Gumbel	121.4	kips	0.23	NL THA
M_{max}	Seismic bending moment	Lognormal	16997.3	kip-in	0.21	NL THA

The friction coefficient is taken to have a relatively high mean value of 0.80 because net tension rarely occurs in this specific column base connection, even under severe ground motions corresponding to the highest seismic hazard level. The load case corresponding to the maximum bending moment (M_{max}) and the corresponding shear (V_m) and axial (P_m) loads occurring at the same time instant is found to be the most critical for the connection, resulting in the highest failure probabilities. Additional load cases are also considered in the analysis but are not presented in this paper for brevity. The correlation coefficients between the seismic shear, bending moment and axial loads are determined based on 7 records for each hazard level. The Nataf joint distribution model (Liu and Der Kiureghian, 1986) is assumed for the loads.

1.2. Limit-State Formulation and Failure Mode Hierarchy

The limit-state for each failure mode of the base-plate connection is formulated based on the AISC Design Guide No.1-2005 procedure. This Guide assumes a rectangular stress distribution in the supporting concrete foundation, consistent with the LRFD method for design of reinforced concrete structures used in the US. According to the LRFD methodology, different components of the connection are considered to be at their plastic or ultimate capacities and their relative stiffnesses are disregarded for determination of internal forces. The flexibility of the base plate is neglected for calculating the bearing stress. The dimensions of the plate and the anchor bolts required to achieve the desired strength are obtained from global vertical and moment equilibrium equations. The yield-line theory is used to model the bending behavior of the base plate. The resulting base plate design is also checked for shear-friction resistance and anchor bolt shear. If the shear capacity is insufficient, bearing action to resist shear can be developed by adding shear lugs under the base plate. Shear checks are performed assuming no interaction between the shear and moment resistances.

The limit-state functions $g(x)$ for all failure modes used for the component and system reliability analysis are defined as the difference between the corresponding capacity and demand values: $g(x)$ = Capacity – Demand (see Table 2). Failure is defined as the event where demand exceeds capacity, i.e. $g(x)<0$, and does not necessarily correspond to a physical collapse of the connection. For the ductile failure modes, a redistribution of forces among the components of the connection is expected to occur. Such behavior is disregarded in this formulation.

Table 2 Limit-state functions

Component	Description	g_i (X)- Limit-state function
1	Concrete crushing	$g_1(\underline{X}) = 0.85kf'_c - \left(\frac{P}{NB} + \frac{M}{(1/6)BN^2}\right)$
2	Yielding of the base plate due to cantilever bending on the compression side	$g_2(\underline{X}) = \frac{F_{y,pl}(t_p)^2}{4} - \left(\frac{P}{NB} + \frac{M}{(1/6)BN^2}\right) \cdot \frac{1}{2}\left(\frac{B-0.80b_f}{2}\right)^2$
3	Yielding of the base plate due to cantilever bending on the tension side	$g_3(\underline{X}) = \frac{F_{y,pl}B(t_p)^2}{4} - (0.85kf'_c B \cdot A - P)\frac{(N-d_c-2d_{edge})}{2}$ $A = (N-d_{edge}) - \sqrt{(N-d_{edge})^2 - 2\frac{P(M/P+N/2-d_{edge})}{0.85kf'_c B}}$
4	Tensile yielding of anchor bolts	$g_4(\underline{X}) = \frac{n}{2}(C_{ub1})F_{ub}\frac{\pi d_b^2}{4} - (0.85kf'_c B \cdot A - P)$ C_{ub1}=0.75 is a coefficient for ultimate stress of the anchor bolts in tension.
5	Friction failure or sliding of plate	$g_5(\underline{X}) = \mu P - V$
6	Shear failure of anchor bolts	$g_6(\underline{X}) = C_{ub2}F_{ub}\frac{\pi d_b^2}{4} - \frac{V}{2}$ C_{ub2}=0.50 is a coefficient for ultimate stress of the anchor bolts in shear. Only two bolts are assumed to be effective in resisting shear in the connection.
7	Bearing failure of shear lugs against the adjacent concrete	$g_7(\underline{X}) = C_{brg} f'_c n_{sl} b_{sl}(l_{sl} - t_{grout}) - V$ C_{brg}=0.80 is a concrete bearing coefficient

Selection of base plate dimensions and material strengths following the AISC Design Guide 1-2005 method resulted in some highly unlikely failure modes (i.e. these failure modes have high safety factors). They are: concrete edge breakout, anchor bolt pull-out failure, bearing failure of the base plate, bending failure of the shear lugs, and column-to-base-plate weld failure. These failure modes are therefore ignored in this paper. A hierarchy of column base connection failure modes used in the reliability analysis is shown in Figure 2.

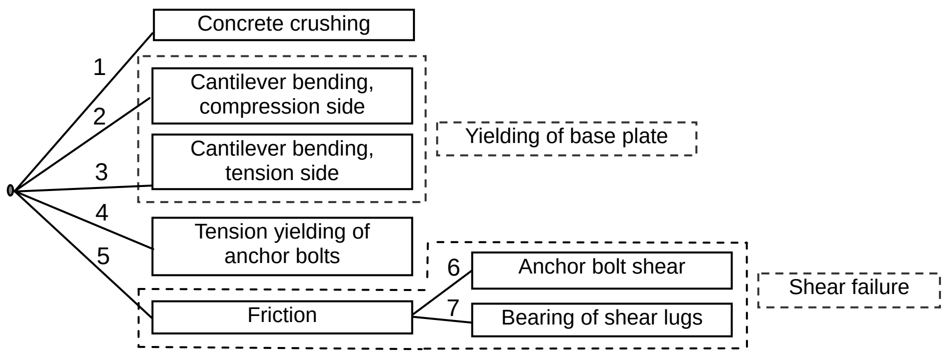

Figure 2 Hierarchy of column base connection failure modes

The minimum cut-set formulation of the system is: C_{min}={(1)(2)(3)(4)(5,6)(5,7)}, i.e., the failure of the system can occur due to 6 failure modes. The component and system reliability analyses are carried out using the first-order reliability method (FORM) for the four seismic hazard levels shown in Table 1. Computations are carried out using the CalREL software (Der Kiureghian, Haukaas and Fujimura, 2006).

3. DISCUSSION OF RESULTS

3.1. System Reliability Analysis Results

Using the minimum cut-set formulation for the column base connection system, component reliability analysis results are combined to obtain the conditional system failure probability of the connection for the four seismic hazard levels considered. The design of the connection remains unmodified throughout. The failure probabilities for each failure mode and for the system are presented in Table 3.

Table 3 Conditional failure probabilities computed for different hazard levels

P_{f1}- Failure probability		**Hazard level (PE in 50 yr)**			
Failure Mode	**Description**	**2%**	**5%**	**10%**	**50%**
1	Concrete crushing	5.818×10^{-2}	1.069×10^{-2}	9.524×10^{-3}	1.467×10^{-4}
2	Yielding of base plate (compression side)	2.956×10^{-1}	1.171×10^{-1}	7.551×10^{-2}	1.061×10^{-3}
3	Yielding of base plate (tension side)	5.554×10^{-4}	1.490×10^{-6}	1.898×10^{-8}	4.723×10^{-9}
4	Tensile yielding of bolts	1.695×10^{-2}	1.378×10^{-5}	5.155×10^{-6}	7.011×10^{-7}
5	Friction & bolt shear	2.370×10^{-11}	1.431×10^{-11}	1.307×10^{-11}	8.405×10^{-12}
6	Friction & bearing of shear lugs	2.129×10^{-2}	1.311×10^{-2}	1.046×10^{-2}	5.268×10^{-4}
System- Failure probability P_{f1}		**3.431×10^{-1}**	**1.381×10^{-1}**	**9.170×10^{-2}**	**1.700×10^{-3}**
System- Reliability index β		**0.404**	**1.089**	**1.330**	**2.935**

The total failure probability for each failure mode and for the system are obtained by combining the corresponding conditional failure probabilities at each hazard level presented above weighted by the corresponding probabilities of the hazard levels. Table 4 presents the total failure probabilities for 1 year and for the expected 50 year lifespan of the structure. Also shown in Table 4 is the corresponding system reliability index.

Table 4 System failure probability P_{f1} and reliability index β

Failure Mode	**Description**	**$P_{f1,t=50\ yr}$ (%)**	**$P_{f1,t=1\ yr}$ (%)**
1	Concrete crushing	3.234×10^{-3}	1.993×10^{-4}
2	Yielding of base plate (compression side)	2.026×10^{-2}	1.405×10^{-3}
3	Yielding of base plate (tension side)	1.983×10^{-5}	2.879×10^{-7}
4	Tensile yielding of bolts	6.054×10^{-4}	9.340×10^{-6}
5	Friction & bolt shear	9.469×10^{-12}	8.425×10^{-12}
6	Friction & bearing of shear lugs	2.373×10^{-3}	5.623×10^{-4}
System- Failure probability P_{f1}		**2.431×10^{-2}**	**2.107×10^{-3}**
System- Reliability index β		**1.972**	**2.862**

The largest contribution to the system failure probability is due to yielding of the base plate on the compression side, which is a ductile and desirable failure mode. The other dominant failure modes are undesirable brittle failures including concrete crushing and shear failure due to sliding of the base plate and bearing failure of the shear lugs against the adjacent concrete. Tension yielding of the anchor bolts also has an important contribution. The remaining failure modes have negligible contribution to the system's failure for this connection design.

The resulting system reliability index β of 1.972 and failure probability P_{f1} of 2.43% computed for the expected 50 year lifespan of the structure may be considered relatively low and high, respectively. The design, carried out for the 10% in 50 year PE hazard level, also results in relatively high conditional failure probability of 9.17% for an earthquake of relatively moderate intensity. For the highest seismic hazard level of 2% in 50 year PE, the failure probability of 34.31% is also relatively high. Based on the results of this reliability analysis, the AISC Design Guide No. 1-2005 column base plate connection design procedure should be modified by reducing resistance factors to increase connection reliability. It is also important to incorporate a capacity design approach to promote the occurrence of ductile failure modes over brittle failure modes.

The failure probability estimates presented above, which employ the well known PEER formula, entail an error due to the presence of non-ergodic variables (Der Kiureghian, 2005). The original PEER formula was intended to compute the mean annual rate of a performance measure exceeding a specified threshold. Approximation of the exceedance probability using this formula may result in as much as 20% error for probabilities around 0.05 and 30% error for probabilities around 0.10. For failure probabilities less than 0.01 the approximation has a negligible error. In the present project the total failure probability of the connection computed for one year is less than 0.01.

Therefore this approximation of the failure probability has a negligible error. For the lifespan of the structure of 50 years, the error in the failure probability of 2.43% may be as much as 20%. However, since the error is on the conservative side (Der Kiureghian, 2005), this approximation of the connection reliability is found to be acceptable.

Fragility curves are obtained relating the conditional failure probabilities of occurrence of each failure mode and occurrence of system failure to an earthquake intensity measure (IM). In this study, IM is the spectral acceleration at the first mode period ($S_{a,T1}$) of the MRF. This measure was computed at each hazard level using the hazard data for a location in Berkeley, California. The fragility curves are obtained using a lognormal fit to the data presented in Table 3 and a least-square approximation of the error (see Figures 3 and 4).

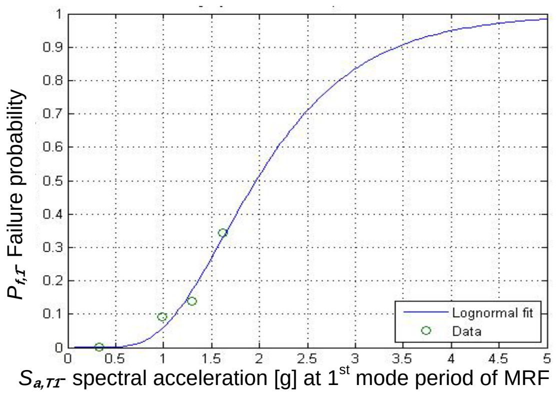

Figure 3 Lognormal fit to failure probabilities of base plate connection

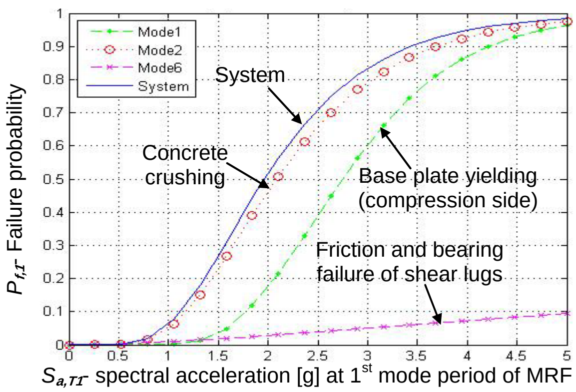

Figure 4 Fragility curves for the system and predominant failure modes

3.2. Relative Importance of Random Variables

Determining the order of importance of random variables for column base connection behavior is performed by perturbing the mean values of these variables (on the order of 10%), determining the resulting variations in connection reliability, and computing the importance value $\delta=(\partial\beta/\partial\mu)\sigma$ for each variable. The characterization as a capacity or demand variable is established for each RV according to the sign of the corresponding element in vector δ. The results computed for the 2% in 50 year PE hazard level are presented in Table 5.

Table 5 Importance vector δ obtained from system reliability analysis

RV	$\delta=(\partial\beta/\partial\mu)\sigma$	Order of Importance	Approximate Classification
d_c	2.40×10^{-5}	17	Capacity
b_f	8.65×10^{-2}	7	Capacity
N	2.72×10^{-1}	4	Capacity
B	-5.23×10^{-1}	1	Demand
t_{PL}	2.20×10^{-1}	5	Capacity
l_{sl}	2.86×10^{-3}	14	Capacity
b_{sl}	3.46×10^{-3}	12	Capacity
d_b	3.00×10^{-3}	13	Capacity
d_{edge}	-4.17×10^{-4}	16	Demand

RV	$\delta=(\partial\beta/\partial\mu)\sigma$	Order of Importance	Approximate Classification
t_g	-7.25×10^{-2}	9	Demand
$F_{y,PL}$	3.20×10^{-1}	3	Capacity
F_{ub}	4.09×10^{-3}	11	Capacity
f'_c	1.02×10^{-1}	6	Capacity
μ	7.80×10^{-2}	8	Capacity
P	-2.80×10^{-3}	15	Demand
V	-5.72×10^{-2}	10	Demand
M	-4.90×10^{-1}	2	Demand

On the demand side, the base plate dimension affecting directly the cantilever length and bending moment of the column base plate has the highest influence on the failure of the connection. On the capacity side, the base plate thickness and base plate steel strength represent important components of the system resistance. The strength of the concrete foundation and the friction coefficient between the base plate and grout are the next important aspects affecting the reliability of the column base connection.

3.3. Sensitivity Analysis of Limit-State Parameters

The sensitivity of the failure probability P_{f1} of the system with respect to the limit-state parameters, denoted θ_g, are obtained through a small perturbation in the parameter values (on the order of 5%) and the computation of the resulting variations in P_{f1}. The results for the 2% in 50 year PE hazard level are presented in Table 6. The confinement coefficient k, which for the present design is equal to 2.0 based on the assumption of adequate transverse reinforcement and large cross section of the concrete pedestal, has a sensitivity $\nabla_{\theta g} P_f\ (\theta_g/P_f)$ of 0.89. For example, this corresponds to an increase of 0.089 (or 8.9%) in the system failure probability P_{f1} for the 2% in 50 year hazard level if the value of parameter k is reduced by 10% due to inadequate confinement. The remaining limit-state parameters have small to negligible effect on the failure probabilities of the connection, even for the highest hazard level.

Table 6 Sensitivities of P_{f1} of the system to variation in limit-state parameters θ_g

Parameter	Description	θ_g- Value	$\nabla_{\theta g} P_f\ (\theta_g/P_f)$-Sensitivity
k	Concrete confinement coefficient for maximum stress	2.0	0.89
C_{ub1}	Coefficient for tension in anchor bolts	0.75	0.10
C_{ub2}	Coefficient for shear in anchor bolts	0.50	0.00
C_{brg}	Coefficient for bearing of shear lugs	0.80	0.14

4. CONCLUSIONS

The reliability of an exposed column base designed according to the AISC Design guide No. 1-2005 provisions was evaluated in this paper. A sample column base was designed for the moment resisting frame of the ATC-58 building located in a high seismic zone. Random variables describing this design were characterized using ASTM and AISC design code data. The demands on the column base were characterized using data from suites of non-linear time history earthquake response analyses of the ATC-58 building frame conducted at four different seismic hazard levels (2, 5, 10 and 50% in 50 year probabilities of exceedance). A hierarchy of column base connection failure modes was established. Reliability analysis was carried out using the FORM approximation to compute the reliability index of the connection. Fragility curves for each failure mode and the system were developed using a lognormal fit.

The most likely failure modes of the connection were yielding of the base plate on the compression side (ductile), concrete compression crushing (brittle), and shear failure due to base plate sliding and bearing failure of shear lugs (brittle). In order to promote ductile failure modes and a desirable failure mode sequence, resistance factors in AISC Design Guide No. 1-2005 should be decreased and a capacity design approach should be rigorously adhered to. Although relatively high failure probabilities were computed for the column base connection at the high hazard level (34.31%), a lower probability of failure equal to 2.43% (β=1.97) was obtained as the combined total failure probability of the connection in 50 years. This failure probability can still be considered as relatively high for a critical structural connection. The column base connection design procedure proposed in AISC Design Guide No.1 can therefore be considered unconservative.

A sensitivity analysis was carried out to evaluate the importance of the random variables, as well as the sensitivity of the failure probabilities to variations in the limit-state function parameters. On the demand side, the largest cantilever length of the base plate has the highest importance for the connection’s failure. This dimension should be minimized during the design process to minimize the bending demand on the plate. The thickness of the plate and its yield strength are also important variables controlling failure. Finally, a reduction in the confinement of the foundation can lead to an important increase of the failure probability of the connection due to crushing of concrete. Steel frame and foundation designers should interact to ensure adequate confinement of the foundation, particularly for column bases of the perimeter columns.

Resistance to sliding of the column base should be studied further to better understand the interaction between the different shear resistance mechanisms and their sequence. Different column base configurations should also be investigated because other failure modes may be triggered. Data from laboratory tests is needed to perform additional component and model uncertainty analyses, and to calibrate the resistance factors in the design equations.

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