EVALUATION OF STEEL COLUMN-BASE PLATE CONNECTION IN SPECIAL MOMENT-RESISTING FRAMES AND OPTOMIZED BY ABAQUS

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ABSTRACT: Many steel moment-resisting frame buildings endured failure at their column base connections amid the 1995 Kobe, 1994 Northridge and 1989 Loma Prieta seismic tremors. Framework dependability investigation of an uncovered moment-resisting base plate connection designed for a low-ascent steel extraordinary moment resisting frame is done utilizing a basic unwavering quality examination software. Methods of failure of the column base are characterized utilizing a farthest point state definition based on the AISC Design Guide No. 1-2005. The overwhelming failure methods of the uncovered column base include: yielding of the base plate on the compression side, crushing of concrete, and shear failure because of sliding of the base plate and bearing failure of the shear lugs against the adjacent concrete. Affectability examination is done to decide the impacts of point of confinement state and dispersion parameters on the unwavering quality of the framework. On the demand side, the cantilever length of the base plate reaching out past the column cross segment and the bowing moment at the column base are observed to be the principle parameters impacting the failure of the column base connection. On the limit side, the thickness of the base plate and the quality of steel are the primary parameters affecting the unwavering quality of the connection. Delicacy bends are produced for every failure method of the column base plate and in addition for the connection as a framework. These are communicated as a component of the ghastly quickening at the primary mode time of the building.

KEYWORDS: Steel base plate connection, anchorage bolt, reliability analysis, moment resisting frame, Finite Element, Abaqus.

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1. INTRODUCTION

A run of the mill column-base connection between the column of a steel moment-resisting frame (MRF) and its concrete establishment, ordinarily utilized in US steel development today, comprises of an uncovered steel base plate bolstered on unreinforced grout and anchored to the concrete establishment utilizing steel anchor bolts.

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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.

Various philosophies for the design of column-base plate connections under different load conditions are found in the writing. The latest technique introduced in the AISC Design Guide No. 1-2005 (Fisher and Kloiber, 2005) is as of now generally actualized in current US building practice.

Unwavering quality examination of a column base connection in a MRF, acquired utilizing the AISC Design Guide No. 1-2005 strategy, has not been done to date. However, such unwavering quality investigation is expected to survey the security of this critical basic part as for its assorted failure modes and to assess the ampleness of the design technique and farthest point state detailing. An affectability examination of the diverse segments of the column base connection is expected to distinguish the basic parameters in the design procedure. These issues are the focal point of the present paper.Various philosophies for the design of column-base plate connections under different load conditions are found in the writing. The latest technique introduced in the AISC Design Guide No. 1-2005 (Fisher and Kloiber, 2005) is as of now generally actualized in current US building practice.

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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

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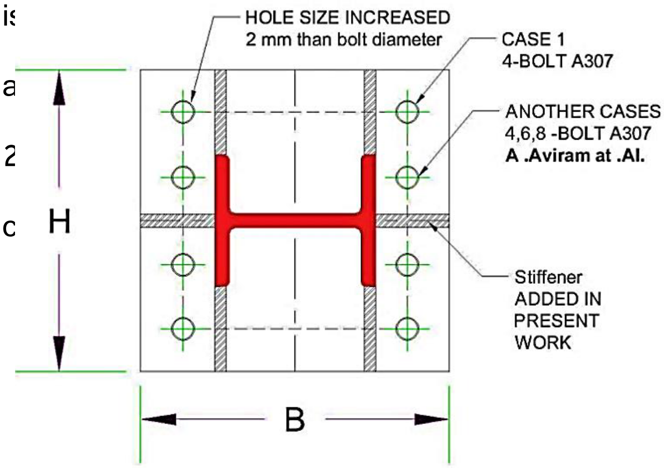

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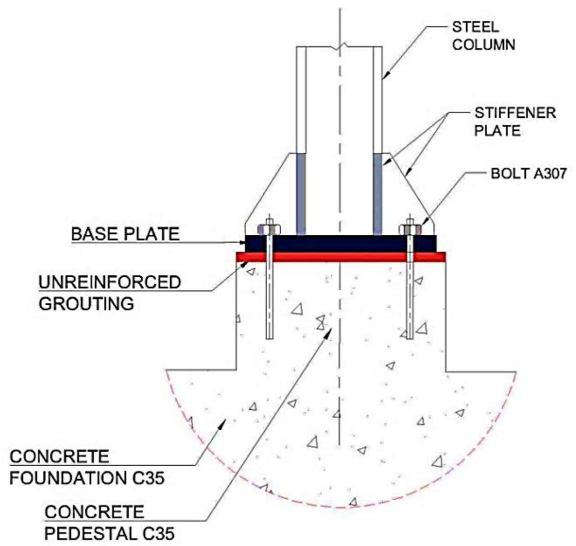

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Figure [1]: the Column – Base Plate connection

The loads used for the design of the connection are obtained from a series of nonlinear time history analysis (NLTHA) 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

ISSN 2229-5518

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 plate

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

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

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

corresponding to the highest seismic hazard level. The load case corresponding to the maximum bending moment (Mmax) and the corresponding shear (Vm) and axial (Pm) 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 stiffness's 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

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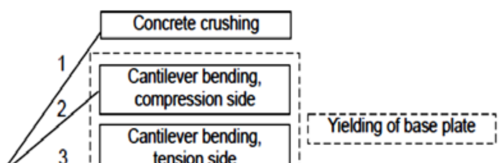
shear and moment resistances.The limit-state functions g(x) for all failure modes used for the component and system reliability analysis are defined

No.	DETAILS	LIMIT STATE FORMULA
1	CRUSING OF CONCRETE MATERIAL	$C1\ (X)=0.85\ K\ f'_C - [(P/NB)+(M/0.166*B.N^2)]$
2	Base plate yielding due to bendin [Compression face]	$C2\ (X)=[(0.25fy_{BPL}\ (t_p)^2)-(P/NB)+(M/0.166*B.N^2)*(0.5(B-0.8B_F)/2)^2]$
3	Base plate yielding due to bendin [TENSION face]	$C3\ (X)=[(0.25fy_{BPL}\ (t_p)^2)-(0.85\ K\ f'_C\ B.\ A-P)((N-d_c-2d_{edge)}/2))]$ $A=(N-d_{edge})-((N-d_{edge})^2-2(P(M/(P+0.5N-d_{edge})/0.85\ K\ f'_C\ B))^{0.5}$
4	BOLTS YIELDING BY TENSILE ACTION	$C4\ (X)=0.5n(C_B\ .\ F_B*0.25*3.14*d_b)-(0.85\ K\ f'_C\ B.A-P)$ C_{B1} = 0.75 is a coefficient of [ultimate stress in tension for anchorage bolt]
5	Sliding of plate- loss friction	$C5\ (X)=\mu.P-V$
6	Anchorage bolt – shear failure	$C6\ (X)=C_{B2}.F_B(0.25*3.14*d_b-0.5V)$ C_{B2} = 0. 5 is a coefficient for shear ultimate stress
7	Bearing failure – shear lugs	$C7\ (X)=C_{B3}\ f'_C\ n.b(l-t)-V$ C_{B3} = 0. 5 is a bearing coefficient of concrete

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.

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.

2. ABAQUS [FINITE ELEMENT MODEL]

In the present FEM, all connected parts including the column, base plates, and anchorage bolts were modeled using the continuum three dimensional eight quadratic [3d stresses] element with reduced integration technique. See figure [3-a]. The structural steel components such as steel column and bolt are modeled as an isotropic elastic-plastic material in both tension and compression. The mechanical properties of steel material for members and anchorage bolts shown in figure [4]. Figure [4] shows the yield stress to plastic strain of both the bolt and another
steel parts of base plate to column connection. The true stress – true strain relationship is then obtained from curves and tabulated for the use in ABAQUS.

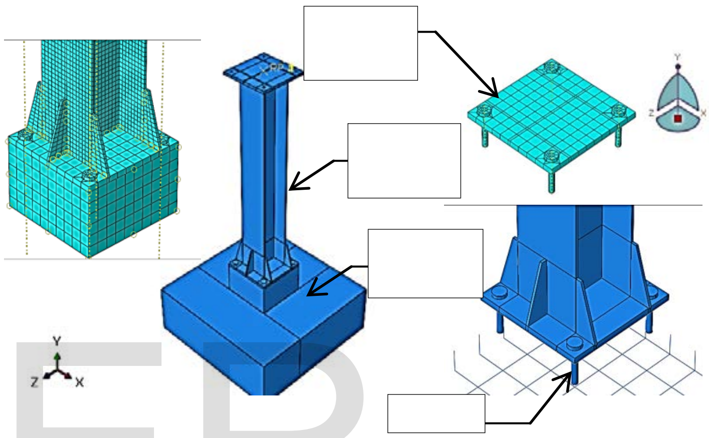


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Figure [3]: Steel Column to RC Footing Connection

2.1 Mechanical Properties of Materials

The mechanical properties of steel members [column section, base plate and stiffeners] and high strength bolts A307 shown in figure below.

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Figure [4] : The yield stress to plastic strain of both the bolt and another steel parts of base plate to column connection.

Figure [5] : Von Mises Stress distribution and deformation of base plate, anchorage bolts , stiffeners, and column [ABAQUS].

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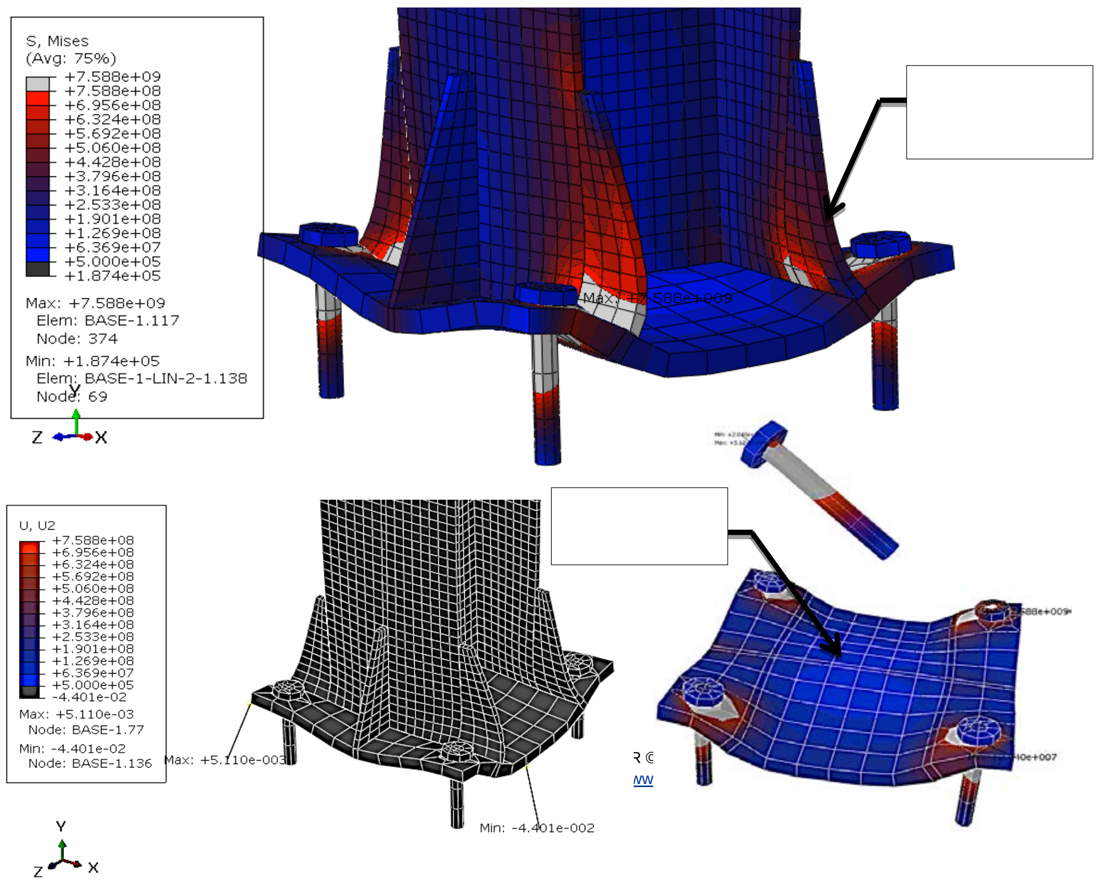

Using the minimum cut-set formulation for the column base connection system, component reliability analysis results are combined to obtain the conditional

P_{f1}- Failure probability		Daingrous level calculate for 50 years			
Failure Mode	Description Numerical and Abaqus	1.5% with stiffeners	(2%) $*10^{-2}$	(10%) $*10^{-2}$	(50%) $*10^{-2}$
1	Concrete of pedestal crushing	$1.889 *10^{-2}$	3.86	9.324	0.01466
2	BPL Yeilding (compression face)	$2.778*10^{-1}$	27.665	7.1203	0.10803
3	BPL Yeilding (tension face)	$4.5670*10^{-4}$	$3.765*10^{-2}$	$1.9802*10^{-6}$	$0.04760*10^{-7}$
4	Bolts yeild by tensile stress	$1.765*10^{-2}$	$1.845*10^{-4}$	$1.403*10^{-9}$	$0.07401*10^{-5}$
5	Bolt shear and friction mechanism	$2.344*10^{-11}$	$1.3989*10^{-9}$	1.1230	$0.08405*10^{-10}$
6	Bearing of shear lugs and Friction	$2.003*10^{-2}$	1.0554	1.0431	0.05245
P_{f1}	**Failure probability** **System**	$3.243*10^{-1}$	9.2043	8.9750	$0.1705*10^{-1}$
β	**Reliability index** **System**	0.412	$1.076*10^{2}$	$1.229*10^{2}$	303.5

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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.

3. DISCUSSION OF RESULTS

3.1. System Reliability Analysis Results

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

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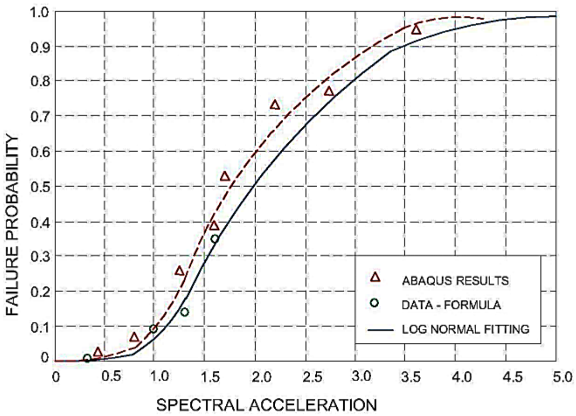
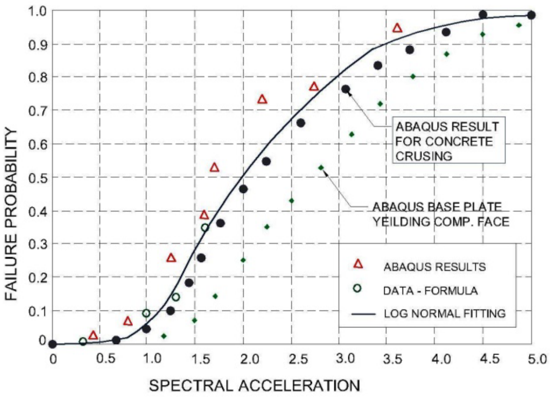
ISSN 2229-5518

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 6 and 7).



Figures 6: Failure Probability of Base plate connection [Log Normal fitting]



Figures 7: Fragility curves for the system and predominant failure modes.

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3.REFERENCES

1-American Institute of Steel Construction (2000). Code of Standard Practice for Steel Buildings and Bridges, American Institute of Steel Construction, Inc., Chicago, Illinois.

2-ATC-58 (in progress). Development of Next-Generation Performance-Based Seismic Design Procedures for New and Existing Buildings, Applied Technology Council, Committee 58, http://www.atcouncil.org/atc-58.html.

ISSN 2229-5518

3-ASTM A992 (2004). Standard Specification for Structural Steel Shapes, American Standards for Testing and Materials, ASTM International, West Conshohocken, Pennsylvania.

4-ASTM A6 (2005). Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling, American Standards for Testing and Materials, ASTM International, West Conshohocken, Pennsylvania.

5-ASTM F1554 (2004). Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength, American Standard for Testing and Materials, ASTM International, West Conshohocken, Pennsylvania.

6-Bertero V.V., Anderson J.C., and Krawinkler, H. (1994). Performance of Steel Building Structures during the Northridge Earthquake, Report No. UCB/EERC-94/09, Earthquake Engineering Research Center, University of California at Berkeley.

7-Der Kiureghian, A. (2005). Non-ergodicity and PEER's Framework Formula. Earthquake Engineering and Structural Dynamics 34, 1643-1652.

8-Der Kiureghian, A., T. Haukaas and K. Fujimura (2006). Structural reliability software at the University of California, Berkeley. Structural Safety, 28(1-2):44-67.

9-Fisher, J.M. and Kloiber, L.A., 2005. AISC Design Guide No.1- Column Base Plates, American Institute of Steel Construction.

10-Liu, P-L., and A. Der Kiureghian (1986). Multivariate distribution models with prescribed marginals and covariances. Probabilistic Engineering Mechanics, 1, 105-112.

11-MacGregor, J. G. and Wight, J. K. (2005). Reinforced Concrete: Mechanics and Design, 4th Edition, Prentice Hall, Upper Saddle River, New Jersey.

12-Yang, T.Y., Moehle, J., Stojadinovic, B., and Der Kiureghian, A. (2006). An Application of PEER Performance-Based Earthquake Engineering Methodology. Proceedings, Eighth National Conference on Earthquake Engineering, San Francisco, California, April 18-22, 2006, CD, Paper #1271.

13-Youssef, N.F.G., Bonowitz, D., and Gross, J.L. (1995). A Survey of Steel Moment-Resisting Frame Buildings Affected by the 1994 Northridge Earthquake, NISTIR 5625. National Institute of Standard and Technology, Gaithersburg, Maryland.

14-Abaqus user's manual 6.14.

15- A. Aviram et. Al. " Reliability Of Exposed Column-Base Plate Connection In Special Moment-Resisting Frames ". World Conference on Earthquake Engineering October 12-17, 2008, Beijing, China.

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