RELIABILITY EVALUATION OF ENGINEERING DEMAND PARAMETERS BASED ON THE LENGTH OF SEISMIC LINKS IN ECCENTRICALALLY BRACED FRAMESE

Leila HAJ NAJAFI
PhD. Candidate, Amirkabir University of Technology, Tehran, Iran
lila_najafi@aut.ac.ir

Mohsen TEHRANIZADEH
Professor, Amirkabir University of Technology, Tehran, Iran
dtehz@yahoo.com

Keywords: Reliability, Eccentrically Braced Frames, Link Beam, Engineering Demand Parameter, First Order Second Moment.

ABSTRACT

Reduction in deviation of performance indexes coclude reduction of deviations in decision parameters. One approach for reducing the deviations is adjusting structural design specifications and utilizing appropriate seismic systems. Eccentrically braced frames (EBFs), One of prevalently employed seismic systems are very well-organized and adjustable systems for resisting earthquakes as they combine ductility that is the characteristic of moment frames and stiffness associated with braced frames. This study evaluates the seismic reliability of engineering demand parameters according to the different length of seismic links based on the first order second moment reliability method (FOSM).

The selected EDPs like most of the performance-based assessments are inter-story drift ratios (IDR) and peak floor acceleration (PFA) and two dimensional generic one-bay frames in different stories representative of typical structures with different heights and fundamental periods were employed subjected to two groups of near and far-field records. The median values of reliability index (β), representative of the results' dispersion around the median value, were calculated subjected to the drift and peak floor acceleration for each story of the models.

It has been discovered that, there is an efficient length of link beam that is located in the range of ratios between 0.25 to 0.33 (ratio of the length of link-beam member to the length of braced span) for each of the models with different number of stories conducting to the most reliable EDPs as well as very close compatibility of the efficient length of link beams for both drift and acceleration EDPs. It could also be concluded that the dispersion values subjected to far-field records are almost constant by altering the length of link-beams. Achievements could be straightforwardly utilized for risk assessment or design of EBF systems supporting appropriate length of link beams in countering reliability of EDP parameters.

INTRODUCTION

Performance-based earthquake engineering (PBEE) provides a quantitative basis in assessment of the seismic performance of structures and aims at the design of structures achieving expected acceptable performance levels during probable future earthquakes (FEMA450, 2003). One of very frequently used quantitative performance assessment method is the fully probabilistic methodology of Pacific Earthquake Engineering Research (PEER) Center that is divided into four basic stages accounting for the following: ground motion hazard of the site, structural response of the building, damage of building components and setting up decision variables (DVs) like economic loses, which could be employed by stakeholders to make more informed design decisions (Ramirez and Miranda, 2009). The outcomes of each stage serve as input to the next stage.

The deviation in each stage of performance causes deviation in the decision variables; therefore one of researcher's attempts is to reduce the deviations at each stage. Many approaches have been followed for this purpose like developing new intensity measures which have been employed in the first stage and also dealing with many different engineering demand parameters to encounter as slight deviation in structural responses as it is possible. One approach for reducing deviations in structural responses is adjusting structural design specifications and utilizing appropriate seismic systems. One of prevalently employed seismic systems are eccentrically braced frames (EBF) which are very well-organized structures for resisting earthquakes as they combine ductility that is a characteristic of moment frames and stiffness associated with braced frames (Chao and Goel, 2006).

Great optimization attempts in the previous works devoted to design phase of the EDP systems, like optimizing maximum dissipating energy in the link beams subjected to some design specifications like section of link beams, section of stiffeners in link beams, location of stiffeners and ……(Ohsaki and Nakajima, 2012; Chao and Goel, 2006; Alinia and Dastfan, 2007) that mainly depends on the employed design code or minimizing frame weight based on the geometry of the eccentrically braced frames or other design specifications (Özhendekci, 2008; Gülay and Boduroğlu, 1989; Memari and Madhkhan, 1999); Whereas, the design process of EBFs is not the main concern of view in this paper. By declining the dispersion in the response of systems one could substitute any design pattern based on any design codes.

In this paper, the only specification of the frame that is going to be determined by the ideal amount is the length of link-beam that is a presumed preliminary parameter in design approach of EBF frames and it is not reliant on presumed design provisions. Although according to previous works in this field, the scattering of the structural response is not one of the main indexes considered in design but this paper shows that, the difference between the scattering of geometrically short and long links is striking especially under near-field records. The view considered in this work is noble in researches that helps to select one of preliminary considerations of design not only based on optimizing some response factors like minimizing drift or maximizing energy dissipating, load carrying or link-beam rotational capacity; but also, based on minimizing the dispersion on the responses.

ECCENTRICALLY BRACED FRAMES

Steel braced frame is one of the structural systems used to resist lateral loads in multistoried buildings. Steel bracing is economical, easy to erect, occupies less space and has flexibility to design for meeting the required strength and stiffness whereas they may interfere with architectural features (Ghobarah and Ramadan, 1991).

Concentrically braced frames (CBFs) are prevalent in moderate seismic regions, both because of their high stiffness-to-weight ratio, and because of the ease with which they can be designed and evaluated by the equivalent lateral force method. The concentric bracings increase the lateral stiffness of the frame and usually decrease the lateral drift. However, increase in the stiffness may attract larger inertia forces due to earthquake. Further, while the bracings decrease the bending moments and shear forces in columns, they increase the axial compression in the columns to which they are connected. Eccentrically braced frames (EBFs) can offer the same advantages as CBFs, while also providing significant ductility capacity, and greater flexibility with architectural openings. These systems improve both lateral stiffness of the system and energy dissipation capacity. Moreover, many existing steel buildings need to be retrofitted to overcome the deficiencies to resist the lateral loading and EBF systems are the ideal systems for this purpose in view of the compatibility with the existing architectural plan (Hines, 2009; Hague, 2013; Sabol and Nishi, 2011). They can also be sometimes used to advantage in avoiding large and costly connections which may results solely from the geometric requirements (Popov and Engelhard, 1988).

Links in EBFs act as structural fuses to dissipate the earthquake induced energy in a building in a stable manner. To serve its intended purpose, a link needs to be properly detailed to have adequate strength and stable energy dissipation. All the other structural components (beam segments outside of the link, braces, columns, and connections) are proportioned following capacity design provisions to remain essentially elastic during the design earthquake (Naeim, 2001).

EBF design, like most design problems, is an iterative process. Most of the designers will make a preliminary configuration, choice of bracing arrangement and link length selection based on approximations of the elastic design shears or designers experiences (Becker and Ishler, 1996). If this preliminary configuration in design practice could lead to the most efficient choice of the length of link-beam is the question that is going to be discussed further in this paper.

DESCRIPTION OF STRUCTURAL SYSTEMS USED FOR EVALUATION

On account of the need for generality of the results, and hence, to develop relation between the standard deviation of EDPs and the length of link-beam, the structural frames of the models are not intended to represent a specific structure.

For this purpose, the efficiency of the length of link-beam was considered through conducting nonlinear dynamic analysis of two dimensional generic one-bay frames proposed by Krawinkler and Medina (Krawinkler and Medina, 2004) adapted by EBF structural system. It is worth noting that their study shows that one-bay generic frames are generally adequate to capture the global behavior of multi-bay frames. The generic frames served in this study consist of frames with a number of stories, N, equal to 3, 5, 8, 15 and fundamental periods, T_1, ranges in magnitude from 0.34 (s) to 2.02 (s) representative of typical structures with varying heights and fundamental periods where the height of each story and the length of each span is deemed to be 3m and 6m respectively. The link-beam is assumed to be located in the middle of braced span in all of the models. This EBF type has the advantages of symmetry and since links are not adjacent to the columns, link-to-column connections are avoided. These frames were modeled by means of the open system for earthquake engineering simulation (OpenSees, 2009). IPE and plate girders are utilized for beams whereas BOX and UNP double sections are used for columns and bracings respectively (Tehranizadeh and Haj Najafi, 2008). Design requirements of EDP members are based on AISC2005. This code advises implementation of some stiffeners acquiring enough stiffness and plastic deformation capability for the link members. In this study, the sections of stiffeners and the spaces between them are considered based on the code AISC2005. This paper state application of design patterns based on a very frequently used design specification (AISC, 2005) and accept the deviations based on the diverse link sections which have been chosen based on this design code in its evaluation to adjoin deductions to the real situations.

Except link shear and flexural deformations, all the other deformations are in elastic zone. This motivates the researchers to pick out the EDPs for EBF systems directly associating with the link-beam responses like link-beam rotation in the joint to the out of link-beam instead of general EDPs like inter-story drift ratio or peak floor acceleration. But the point is, by modification in link-beam specifications in most of the design cases the other parts of the system like beams in out of link section, columns and braces have to be adapted and elastic deformation capacity and consequently the final drift capacity of the frame could vary by the modification. Also, in most of the performance assessment procedures, the fragility curves which are supplied to calculate damages in building components are calibrated based on inter-story drifts or peak floor accelerations. In this respect, the selected EDP parameters in this study are inter-story drift ratios (IDR) and peak floor accelerations (PFA), as well as most of the performance based assessments.

Plastification was modeled, using nonlinear material gained from parallel aggregation of some elastoplastic materials for the link-beams. The definition of the incorporated material and nonlinear static analysis were performed according to FEMA273 (1997). Modeling has been performed considering six length of link-beam equivalent to 0.5 m, 1.0 m, 1.5 m, 2.0 m, 2.5 m and 3.0 m respectively equal to 0.083, 0.166, 0.250, 0.333, 0.417, 0.5 ratios of the length of link-beam (e) to the length of braced span (L). Note that e/L ratios of 0.0 and 1.0 correspond to a concentrically braced frame and a moment frame respectively. All the nonlinear dynamic analyses were conducted as Direct Integration Transient time history analyses using Direct Integration in Hilber, Hughes and Taylor's method by consideration of damping ratio for all modes equal to 5% and P-Δ effects.

Respecting that the efficiency of the EDPs should be evaluated at the collapse level as well as the other phases of inelastic performance, the hysteretic model has to incorporate all the significant deterioration sources contributing to demand prediction as the structure approaches collapse. For this purpose, an energy-based deterioration model has been served in this study which reconciles acceptably by the proposed model by Ibara et al. (2005), which permits modeling four major sources of cyclic deterioration, (basic strength, post-capping strength, unloading stiffness and accelerated reloading stiffness). The amounts of each point for cyclic deterioration model were derived from specification of steel A992Fy50 and are exhibited in Fig. 1.

RECORDS

The variability related to record by record variation will be reduced by increasing the number of records, but each percent of reduction expenses too much with respect to nonlinear dynamic analysis. The intent is not to reduce the response dispersion by applying great quantities of records; the intent is to provide an unbiased estimate of the structural response with limited error. A suit of eleven pairs of ground motions is the

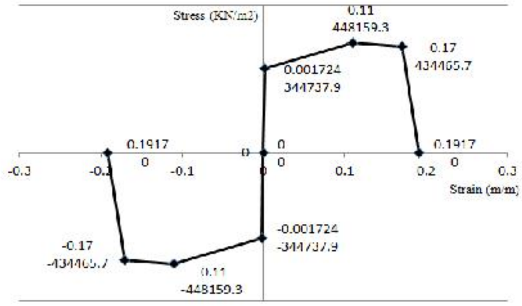

Figure 1. Nonlinear behavior of material used for modeling of link-beams

minimum recommended by the ATC-58. Such a suite will provide a 75% confidence that the predicted median response from will be with +-20% of the true median value of response for an assumed dispersion of 0.5 (ATC-58-1, 2011).

With respect to the considerable effects of pulse motions on dynamic responses of structures, the database in this study comprises eleven near-fault earthquake records identified as containing distinct velocity pulses and enclosing source-to-site distances less than 10 km and all of them were recorded on soil type D (stiff soil, very dense soil and rock) based on NEHRP site classification, equal to Zone 4 of UBC (1997), and soil type II according to Iran Seismic Code (2800 Standard, 2005), or adjusted for this type of soil. (Somerville et al., 1997(a); Somerville et al., 1997(b)).

Table 1: Specifications of ground motions

			Near-field Ground Motions		
Earthquake	Year	Station	Distance (km)	M_W	Duration (sec)
Tabas	1978	Tabas	1.2	7.4	32.84
Bam	2003	Bam	1.0	6.8	66.56
Loma Prieta	1989	Los Gate	3.5	7.0	24.96
Mendocino	1992	Petrolia	8.5	7.1	35.98
Erzincan	1992	Erzincan	2.0	6.7	20.78
Landerz	1992	Lucerne	1.1	7.3	48.12
Northridge	1994	Olive View	6.4	6.7	39.98
Kobe	1995	JMA	0.6	6.9	47.98
Chichi	1995	TCU068	1.1	7.6	90.00
Superstition Hill	1987	Parachute T.S	1.0	6.5	10.5
Coyote lake	1979	Gilroy Array	3.1	5.7	3.4
			Far-field Ground Motions		
Earthquake	Year	Station	Distance (km)	M_W	Duration (sec)
Tabas	1978	Ferdoos	94.4	7.4	40.00
Morgan Hill	2003	Morgan	76.3	6.8	36.00
Landerz	1992	12026 Indio	55.7	7.3	60.00
Whittier Narrows	1987	Downey - Birchdale	56.8	6.0	29.0
Imperial Valley	1979	Victoria	54.1	6.5	86.0
Northridge	1994	Terminal Island	60.0	6.7	40.0
Northridge	1994	Lakewood- Del Amo	59.3	6.7	35.0
Loma Perieta	1989	APEAL 2E	57.4	6.9	25.0
Loma Perieta	1989	Alameda Naval	70.9	6.9	46.0
Chichi	1999	TCU094	54.5	7.6	25.5
Chichi	1999	TCU026	56.1	7.6	22.1

How to choose the records are beyond the scope of this paper and for further details the reader is referred to the paper of Haj Najafi and Tehranizadeh (2014). Records were derived from PEER Strong Motion Database (PEER, 2010) and Iran Strong Motion Network Data Bank (IBHR, 2010). Moreover, eleven far-field records were supplemented to comprehend the comparison. All far-field records have distances above 50 km and do not include any pulse-like wave. The complete specifications of the selected near-field and far-field records have been presented in Table 1.

In this study, Record's two horizontal components were convert into fault parallel and fault normal directions and the effects of horizontal shaking are considered by applying the earthquake shaking components individually along 2D model of the frame without any scaling or modification factors to acquire realistic responses of the frames. The maximum responses associated with these two orthogonal records have been selected as the EDP results.

As six lengths of link-beam were involved for modeling link-beams and utilizing 11 records for near and far-field situations, 132 modeling (66 subjected to near-fields and 66 subjected to far-fields) have been performed to help us concluding in a general state.

STATISTICAL PARAMETERS OF THE STRUCTURAL RESPONSES

In this research, the median and standard deviation of the natural logarithm of EDP parameters were reported as the statistical parameters as well as the probability distribution of EDPs has been assumed lognormal with the parameters gained from the outcomes of nonlinear dynamic analyses.

The median of the EDPs plays a key role in evaluation of structural response and it is the input data to stage three of (PBEE); therefore, its precise estimation conducts to accurate evaluation of the building performance. The median inter-story drift ratios for the models with different number of stories were demonstrated in the case of two length of link-beam in Fig. 2.

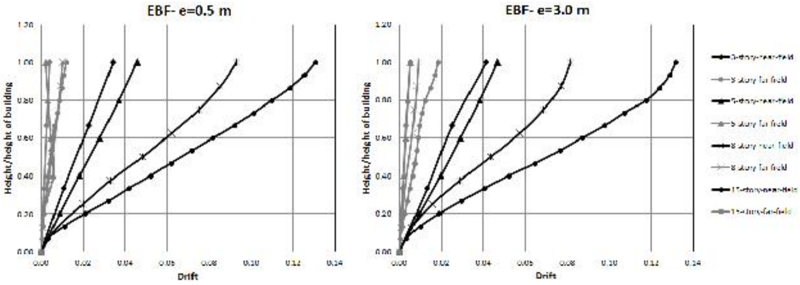

Figure. 2 .The median inter-stories drift ratios subjected to near-field records.

According to near-field ground motions the diagrams illustrate different trends of response for high and mid-rise models rather than short-rise models. Since the standard deviations, not median values have been depicted as the decision making parameter of efficiency, diagrams of median values for the models with the other number of stories have not been moreover pointed out in this paper.

A point estimator is considered more efficient if it leads to a smaller dispersion in comparison to the other point estimators of the same seismic performance parameter. In this study, the standard deviation of natural logarithm of EDP parameters was utilized to compare dispersion around the median values for each EDP parameter associated with the six length of link-beam subjected to a suite of far-field and near-field earthquake records that is going to be discussed comprehensively.

RELIABILITY ASSESSMENT

Probabilistic quantification of vulnerability is attempted by means of an approach relying on First-Order Second-Moment (FOSM) approximation of uncertainty. The basis of FOSM lies in the statement that satisfactory estimates of the parameters of a distribution (which may be unknown) may be given by first-order approximations of Taylor series expansions of second-moment parameters e.g. mean and variance) of a random variable calculated from samples (Uzielli et al., 2006). In operational terms, FOSM analysis requires at least the definition of a central value and a measure of dispersion (Raviprakash et al., 2010). Here, the central value is considered as the median value and the measure of dispersion as the variance of data.If we consider each elementary collapse mechanism by Z_i, each Z_i could be written as a linear combination of M_js contributing to that mechanism:

$$Z_i = \sum_{i=1}^{n} a_{ij} M_j \tag{1}$$

Z_i : Function equation for collapse mechanism in step i and M_j : Moment in hinge number, j.

The safety margin in this equation is the difference of moments between the capacity and demand in the joints contributing to hinges. Considering external forces, Eq. (1) could be expressed as Eq. (2):

$$\sum_{p=1}^{q}\sum_{j=1}^{n} a_{i,j,p} M_{j,p} - \sum_{p=1}^{q}\sum_{k=1}^{n} b_{i,k,p} Q_{k,p} \tag{2}$$

i: Number of step that the equation is written for, $a_{i,j}$ and $b_{i,k}$ are the coefficients that would be attained in step i by considering the equation of equilibrium, and are different for each M_j and Q_k, j : Number of points in the mechanism, regardless of the point that Z_k was written for, M_j : Amount of moments in the points of the mechanism, regardless of the point that Z_i was written for, k : Number of loads contributed in a certain

mechanism, $Q_{k,p}$: Applied external load for span p in the loading step of the linear system by index k, Dead Load : Q_1, Live Load : Q_2, Earthquake Load :Q_3 and p: Number of involved spans in the mechanism.
The equation of g function can be expressed in matrix form as follows:

$$Z_i = [A]\{X\} \tag{3}$$

$$[X] = \begin{bmatrix} M \\ Q \end{bmatrix} \tag{4}$$

$$«_{zi} = [A]\{«_X\} \tag{5}$$

$$\pm^2_{zi} = [A][C_x][A]t \tag{6}$$

where: μ_{zi} : Matrix of average of x and σ_{zi}^2 : Matrix of variance of x.
Therefore, we can define reliability index as:

$$_i = «_{zi} / \pm_{zi} \tag{7}$$

Although calculation of a_i and b_j requires a considerable number of iterations for each mechanism, defining the function equation of the system as above has some advantages. The first is simplification in stiffness matrix production. The second, and more important, is the ability to acquire Z_i as a linear function of M_js and Q_ks; subsequently, the First-order Second Moment method could be precisely applied (Ranganathan, 1990).

As there is one mechanism of failure for a one bay 2D EBF frame in its link beam, the reliability analysis become so simple utilizing Eq. 7. This simplification of collapse mechanism helps the reliability assessment and efficiency assessment lead to the same conclusion. In favor of better understanding about the result dispersion around the mean values, and also to unrestrict the standard deviations from the unite of EDP measuring, the dispersion ratio (DR) was calculated by Eq. (8).

$$DR = \pm / « \tag{7}$$

where: DR: Dispersion ratio and is equal to 1/β, σ: Standard deviation and μ: Mean value.

DR values were calculated subjected to the drift and peak floor acceleration for each story of the models; then, the median of these DR values has been picked out as the representative of the results' dispersion around the median value for each of the models and was exhibited in Tables 2 and 3.

Table 2. The median values of DR for drifts

	1/β Values for Drift Subjected to near-field records					
	e=0.5 m	e=1.0 m	e=1.5 m	e=2.0 m	e=2.5 m	e=3.0 m
3-story	0.38	0.36	0.32	0.36	0.39	0.38
5-story	0.41	0.38	0.34	0.37	0.40	0.42
8-story	0.36	0.35	0.32	0.34	0.35	0.37
15-story	0.44	0.36	0.34	0.40	0.38	0.37
	1/β Values for Drift Subjected to far-field records					
	e=0.5 m	e=1.0 m	e=1.5 m	e=2.0 m	e=2.5 m	e=3.0 m
3-story	0.08	0.07	0.06	0.05	0.05	0.06
5-story	0.21	0.16	0.20	0.19	0.16	0.17
8-story	0.15	0.14	0.16	0.14	0.14	0.15
15-story	0.07	0.06	0.07	0.05	0.08	0.06

Table 3. The median values of DR for peak floor accelerations

	1/β Values for Acceleration Subjected to near-field records					
	e=0.5 m	e=1.0 m	e=1.5 m	e=2.0 m	e=2.5 m	e=3.0 m
3-story	0.24	0.20	0.16	0.18	0.20	0.23
5-story	0.23	0.21	0.17	0.18	0.23	0.32
8-story	0.23	0.21	0.20	0.18	0.20	0.24
15-story	0.24	0.23	0.18	0.20	0.22	0.28
	1/β Values for Acceleration Subjected to far-field records					
	e=0.5 m	e=1.0 m	e=1.5 m	e=2.0 m	e=2.5 m	e=3.0 m
3-story	0.06	0.05	0.05	0.03	0.04	0.04
5-story	0.15	0.13	0.14	0.14	0.12	0.12
8-story	0.14	0.16	0.17	0.12	0.15	0.14
15-story	0.15	0.16	0.16	0.13	0.16	0.15

It could be concluded that the β values subjected to far-field records are almost constant by altering the length of link-beams and the attempts of this study could be focused on the results gained based on the near-field ground motions. The diagrams of the median values of DR for IDR and PFA of the models subjected to near-field ground motions were demonstrated in Fig. 3.

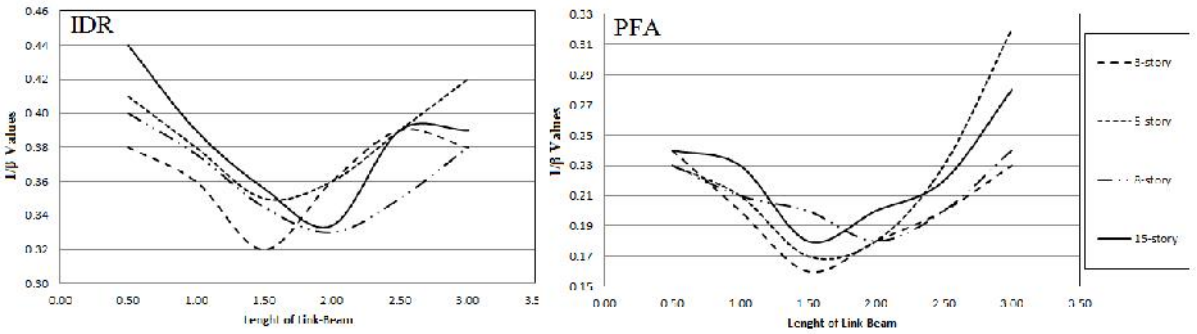

Figure. 3. The median values of DR for drift and acceleration of the models subjected to near-field records.

It could be seen that there is a rational trend in the amounts of DR based on the length of link-beams considering both IDR and PFA. Table 4 provides evaluation of the results gained by modeling. It could be deduced that the efficient length of link-beam is located in the range of magnitude between 1.5 -2.0 (m) (0.25 – 0.33 ratio of the length of link-beam member to the length of braced span) for models with different number of stories and it is adequate for a designer to choose magnitude of the length of link-beams in this range to get slight dispersion in structural responses without dealing with the equations.

Table 4. Efficient length of link-beams for EBF models with different number of stories

Number of stories	Efficient Length of Link beams
3	1.50
5	1.65
8	1.96
15	1.92

CONCLUSIONS

There is very good reconcilement for the efficient length of link-beam achieved based on deterministic analyzing by the help of nonlinear dynamic analysis and probabilistic analysis by the help of β-unzipping method for both drift and acceleration EDPs. The efficient length of link-beam is located in the range of ratios between 0.25 to 0.33 (ratio of the length of link-beam member to the length of braced span) for models with different number of stories. Also, it could be concluded that the dispersion values subjected to far-field records are almost constant by altering the length of link-beams.

REFERENCES

Alinia MM and Dastfan M (2006) Cyclic behavior, deformability and rigidity of stiffened steel shear panels, *Journal of Constructional Steel Research*, 63(4), pp.554–563

Becker R and Ishler M (1996) Seismic design practice for eccentrically braced frames based on the 1994 UBC, Structural Steel educational services, Technical Information and Product Services

Chao S and Goel S (2006) Performance-Based Seismic Design of Eccentrically Braced Frames Using Target Drift and Yield Mechanism as Performance Criteria, *Engineering Journal*, 3(1), pp. 173-200

Ghobarah A and Ramadan T (1991) Seismic analysis of links of various lengths in eccentrically braced frames, *Canadian Journal of Civil Engineering*, 18(1), pp. 140-148

Haj Najafi L and Tehranizadeh M, Ground motion selection and scaling in practice, *Periodica Polytechnica, Civil Engineering (in press)*

Hines E (2009) Eccentric Braced Frame Design for Moderate Seismic Regions, *Structures Congress*, Austin, Texas, United States

Ibara LF, Medina RA, and Krawinkler H (2005) Hysteretic models that incorporate strength and stiffness deterioration", *Earthquak Engineering and Strucural Dynamic*, 34(12), pp.1489–1511.

Krawinkler H and Medina R (2004) Seismic demands for nondeteriorating frame structures and their dependence on ground motions, Report No. PEER 2003/15, Pacific Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, CA (2004)

Naeim F (2001) Seismic Design handbook, springer, 2nd edition, California, United State

Ohsaki M and Nakajima T (2012) Optimization of link member of eccentrically braced frames for maximum energy dissipation , *Journal of Constructional Steel Research*, 75, pp.38-44

Ramirez C and Miranda E (2009) Building-specific loss estimation methods & tools for simplified performance-based earthquake engineering, Report No. 171, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, 2009

Ranganathan R (1990) Reliability analysis and design of structures, Indian Institute of Technology, McGraw- Hill, Bombay, India

Raviprakash AV, Prabu B and Alagumurthi N (2010) Mean value first order second moment analysis of buckling of axially loaded thin plates with random geometrical imperfections, *International Journal of Engineering Science and Technology*, 2, pp. 150-162

Sabol TA and Nishi DE (2011) Application of performance-based design to an eccentrically braced frame structure, *The Structural Design of Tall and Special Buildings*, 20 (S_1), pp. 76–84

Popov EP and Engelhard M (1988) Seismic eccentrically braced frames, *Journal of Constructional Steel research*, 10, pp. 321-354

Somerville P, Smith N, Graves RW and Abrahamson NA (1997a) Modification of empirical strong ground motion attenuation relations to include the amplitude and duration effects of rupture directivity, *Seismological Research Letters*, 68(1), pp.199–222

Somerville P, Smith NS, Punyamurthula S and Sun J (1997 b) Development of Ground Motion Time Histories for Phase 2 of the FEMA/SAC Project, Report No. SAC/BD-97-04, [Online]. Available: http://www.sacsteel.org

Tehranizadeh M and Haj Najafi L (2008) Assessing Seismic Behavior of Eccentrically Braced Frames (EBFs) Due to Near-Field Ground Motions, *Proceeding of the 14th World Conference on Earthquake Engineering*, Beijing, China

Uzielli M, Duzguny S and Vangelstenz BV (2006) A first-order second-moment framework for probabilistic estimation of vulnerability to landslides, International Geotechnical hazard Engineering Conference, Lillehammer, Norway