# EFFECT OF STRUCTURAL STEEL PARAMETER AND QUALITY OF CONSTRUCTION UNCERTAINTIES ON SEISMIC PERFORMANC OF A SPACIAL MOMENT RESISTING FRAME

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## ABSTRACT

Seismic excitations are one of the most hazardous loadings encountered during the life time of structures. Seismic evaluation of Steel Moment Frames, which are used often as lateral seismic system subjected to earthquake must account for the structural steel parameter and workmanship uncertainties, is of high importance.

In this study, the uncertainties ,which involve the quality of workmanship (quality of construction and weld fabrication) that is affected in the behavior of the beam-to-column connectionsas well as mechanical properties such as Young modulus and yield-strength, are parameters for considering those associated with structural steel framing parameters. Incremental dynamic analysis is utilized to assess the structural dynamic behavior of the frames and to generate the required data for performance based evaluations.

A probabilistic framework for seismic assessment of a structural system,which takes into account the uncertainty in the mentioned variables, is used to examine the variation of the probability of exceeding a limit state capacity under seismic excitations. In this study, seismic evaluation of structure has been accomplished in two modes, before construction (the designed structure with no uncertainty) and after construction (the structure with uncertainty). This confidence level is assesable and obtaianble through evaluation of the factored demand-to-capacity, namely DCFD format. SMF at the IO performance level, as affected by uncertainties, shows few changs in DCFD values as well as in confidence level in comparisonwith the structure with no uncertaintywhile, at CP, result shows more changes, increase ofthe DCFD parameter and consequently decrease of confidense levelof the structue affected by unertainrties.

## INTRODUCTION

An earthquake is a natural phenomenon with destructive influences on human life. Many studies have been carried out on its effects which appear in the form of seismic loads in buildings. In recent years, occurrence of severe earthquakes has caused remarkable developments in the field of earthquake engineering, and in fact these earthquakes have been the landmark on the extensive research conducted by various institutions and researchers, resulting in the formation and formulation of regulations and the instructions on this subject.

One important point regarding earthquake is the uncertainties associated with this phenomenon, and now there are efforts to incorporate their effect in seismic design and assessment of human-made structures.

So far, in this respect, some methods have been provided in the form of guidelines and regulations. In previous researches, the uncertainties caused by the earthquake were categorized in two categories; Aleatory and Epistemic. The Aleatory uncertainties are associated with the unpredictability and variability inherent in the earthquake phenomenon, while epistemic uncertainties result from our lack of understanding and knowledge, so epistemic uncertainty could be reduced by gaining better and more information.

In performance-based evaluation, it is necessary to examine seismic demand and capacity by considering the uncertainties. Due to the complexity of evaluating the performance of structures in their nonlinear behavior area, there should be considered methods for modeling their real behavior. In recent years, Cornell et al, have proposed a method based on the demand to capacity ratio which is capable of assessing the structure in a probabilistic framework and thereby determining the reliability of structure at a given performance level. Among the studies in this field one could refer to Vamvatsikos & Cornell (2000), and Jalayer & Cornell (2003), and finally, the results of these studies have been employed to develop guidelines including Fema350. In the current study, researchers have also benefited from the mentioned method for evaluating the performance of structures.

Seismic evaluation of steel moment frames, which are used often as lateral seismic system in Iran, is of high importance. Steel moment frame systems are divided into three categories: Special, with high ductility, Intermediate, with average ductility and Ordinary with low ductility. This research aims at studying the performance of special moment frames. As mentioned earlier, this type of frames has high ductility, and this means that the mentioned frame has less resistance force under the seismic loads in the area of nonlinear behavior, but it tolerates great plastic deformation in the plastic hinges formed in the beams. In fact, the energy loss caused by earthquake damage to structures results from large plastic deformations.

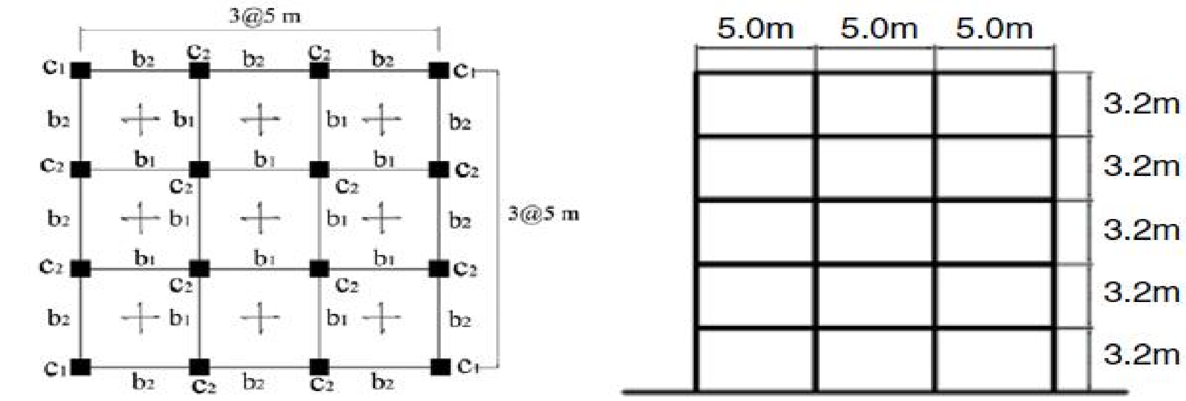
So far, much research has been done on the performance assessment of moment frames, for example, one could refer to the studies of Cornell et al. Jalayer & Cornell proposed a probabilistic framework for determining the seismic reliability of moment frame at a given level of performance (Jalayer & Cornell, 1998). Also Asgarian et al. compared the seismic performance of three types of moment frames discussed previously (Asgarian et al., 2010). It should be noted that, in this study, they used a probabilistic framework provided by Jalayer & Cornell to evaluate the performance. In the present study, the authors are going to make a comparative evaluation of the seismic performance of special moment frames in two modes, one before and one after building construction. In the first case, the structure is designed without considering the uncertainties. In the latter case, the building structure is considered with uncertainties. The considered uncertainties include the material properties and workmanship. In the following, the uncertainties are discussed in details.

## MODELDESCRIPTION

In this section, a designed structure that is required for nonlinear analysis is introduced. The structure has 5 floors with special steel moment frame as lateral load resisting system. It is square in plan and has three bays with equal dimension in each direction. The height of each story is the same and is 3.2 m and the length of each bay is 5 m (Fig. 1). The structure has designed for high seismic risk based on Iranian seismic code (Standard no. 2800, 2004). It is assumed that location of the structure is on a soil type where the shear wave velocity is about 360-750 m/s to the depth of 30m in which according to the mentioned seismic code is named soil type B. The considered response modification factor assignedto special moment frame in Iranian seismic code is 10. Next, the pattern and values of gravity load that consists of live and dead loads are determined based on the Iranian guideline’s roles.

In this study, to develop an accurate modeling and consequently to reach a more precise behavior in nonlinear range, a panel zone model is used to connection modeling. According to the proposed model by Foutch and Yun (2002), a rotational spring is considered as scissor to model the panel zonebased on the proposed model that is shown in Fig. 2a. A nonlinear load deformation backbone curve, that is presented in the mentioned reference for panel zone behavior is assigned to the rotational scissor spring. As mentioned in previous section, one of the goals of this study is considering the quality of workmanship.

According to the previous studies, the quality of workmanship accounts for the structure by the behavior of the beam-to-column connections (Li and Ellsingwood, 2008). Gross has developed an analytical model in the form of a nonlinear hysteretic behavior backbone based on the laboratory experiments that is capable of considering the effect of the weld fracture of the lower beam flange to the column flange. In this

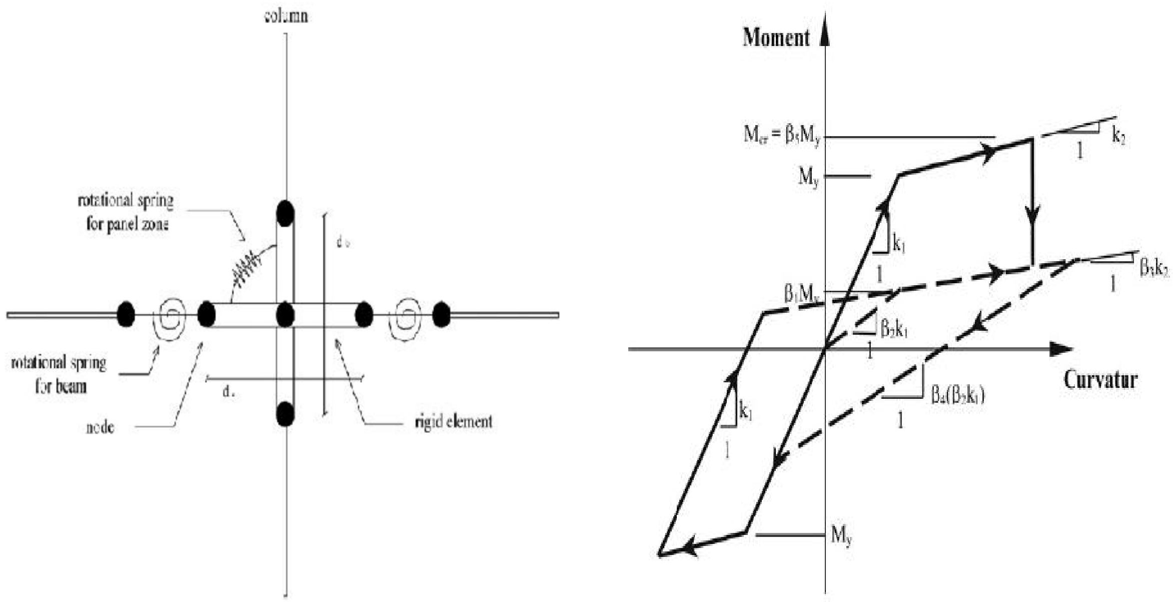



a) Plan view of building b)Elevations and spansrefrenced frame

Figure 1. Plan view and the refrenced interior frameof buildings

analytical model, there are two backbone curves. The first one is bilinear envelop with a yield capacity specified by My that stands for the connection behavior before the weld fracture. After the weld fracture (the bottom beam flange and column flange are effectively disconnected) the analytical model enters the second backbone curve in a way that activates with the onset of the weld fracture specified by Mcr. Then, the former envelop is replaced by the new degraded bilinear envelop that is shown in Fig. 2b. In the second envelop, the new parameter is represented by a factor times the related parameter in the former envelop. In the negative portion of the envelop, we assume the same behavior as the first bilinear model.

These introduced panel zone and connection behavior were modeled by Opensees (Mazzoni et al., 2007). Here, the main designed structure is regular both in the plan view and in the height of structure. Because of this, one of the interior frames shown in Fig. 1a is modeled as representing all the structure. After modeling and analysis, the fundamental period of the reference frame is T1 = 1.7 sec.




a) Panel zone modeling b)Hysteretic model for connection(Li and Ellingwood, (Li and Ellingwood. 2008)

Figure 2. Panel zone model and aHysteretic model of damaged welded connection

## GROUND MOTION

A set of ten ground motion records, that their magnitudes are between 6.5_7.2,and belong to the far field records, islisted in table 1.They are recorded on soil type B (Average shear wave velocity to a depth of 30 m: 360_750 m/s).

Table 1.The Suite of Fifteen Ground Motion Records.

| No | Event | Station | Soil | M | R(Km) | PGA |
|---|---|---|---|---|---|---|
| 1 | NORTHRIDGE | ROLLING HILLS EST-RANCHO VISTA | B | 6.7 | 46 | 0.12g |
| 2 | LOMA PRIETA | COYOTE LAKE DAM DOWNST | B | 6.9 | 22.3 | 0.18g |
| 3 | CAPE MENDOCINO | FORTUNA - FORTUNA BLVD | B | 7.1 | 23.6 | 0.1g |
| 4 | LOMA PRIETA | ANDERSON DAM DOWNSTREAM | B | 6.9 | 21.4 | 0.24g |
| 5 | NORTHRIDGE | STONE CANYON | B | 6.7 | 22.2 | 0.37g |
| 6 | LOMA PRIETA | GILROY ARRAY | B | 6.9 | 19.2 | 0.17g |
| 7 | NORTHRIDGE | LA - CHALON RD | B | 6.7 | 23.7 | 0.21g |
| 8 | NORTHRIDGE | LA - N WESTMORELAND | B | 6.7 | 29 | 0.33g |
| 9 | PALM SPRINGS | SAN JACINTO SOBOBA | B | 6 | 32 | 0.25g |
| 10 | Friuli | FORGARIA CORNINO | B | 6.5 | 24.3 | 0.11g |

## UNCERTAINTYDEFINITION

The features of the mentioned hysteresis backbone, a behavior that is developed based on experiments, are determined and influenced by the weld quality. To create high quality welded joints, which show an acceptable behavior under dynamic actions, a well-trained welder is required while the connections produced by a poorly-trained welder have lower strength and show greater variability in weld fracture resistance. Here, in modeling this behavior, parameters $\beta 1$ and $\beta 5$ are used as random variables which demonstrate the uncertainties of workmanship. The other hysteresis parameters are set at $\alpha$ = 0.03 and $\beta 2$=0.2, $\beta 3$=0.03, $\beta 4$=1. In this study, uniform distribution is assumed for parameter $\beta 1$ and $\beta 5$ (Li and Ellingwood, 2008). For an appropriate workmanship quality, the means, coefficients of variation and the considered statistical distribution are presented in Table 2.Here the mentioned statistical distribution is used because of the lack of practical information about these parameters.

Table 2. Statiscal Distribution for $\beta_1$ and $\beta_5$

| Parameter | Mean(N/m$^2$) | COV | CDF |
|---|---|---|---|
| BETA1 | 0.4My | 0.4 | Uniform |
| BETA5 | 1.1My | 0.2 | Uniform |

Mechanical properties such as Young modulus and yield-strength are parameters for considering uncerainties associated with structural steel framing parameters. In Iran the Yield-stregth of steel material is vary and Young moduls that introduced in Iran's guidelines is different from the other guidelines in which they have measured and presentYoung modulus parameter. In Table 3, the conciered statstical information for mechanical parameter is presented.

Table 3. Statiscal Distribution for Fy and E

| Parameter | Mean(N/m$^2$) | COV | CDF |
|---|---|---|---|
| Fy | 2.35e8 | 0.12 | LogNormal |
| E | 2.1e11 | 0.06 | Uniform |

In this section, fifty random numbers were generated based on thementioned statistical distribution. Here, it's noticeable to state the author's reason for the mentioned generation. This study covers the real range for the yielding strength of steel material that observed in Iran as well as for weld fracture strength and other mentioned parameter, therefore fifty cases are consideredin which, four numbersare simultaneously considered for each uncertainty parameter per conduced IDA. Then the performace assesment has bee done.

## PERFORMANCE EVALUATION

In recent years, incremental dynamic analysis has proved to be an influential method for analyzing the responses of structures because it provides an inclusive and precise evaluation of seismic performance of structure. In this method, because of the large numbers of the conducted nonlinear dynamic analysis, the researchers could evaluate a wide range of the structure's behavior from elastic to nonlinear inelastic, and then it continues to the global dynamic instability (Vamvatsikos and Cornell, 2002). In incremental dynamic analysis method, a suit of records is required, then each record is scaled by several multipliers in a way that they multiply to all of the record time steps and finally, they increase the record intensity. A nonlinear dynamic analysis per each scaled record is performed. Here, the10 ground motions presented in table 1 are used. To display and characterize the incremental dynamic analysis curves we need to scale them in a way that one of them is used to represent the scaling factor of the record (IM) and the other one represents the Engineering Demand Parameter (EDP) as structural response. Based on the performed study by researchers about the efficiency of the mentioned parameters, one of the appropriate parameter for the intensity measure is the 5%-damped first-mode spectral acceleration Sa (T1, 5%), while the maximum interstory drift $\theta_{max}$ of the structure is a good candidate for the EDP. Different patterns or algorithms are presented to scale the records. In this study, the hunt-and-fill algorithm is applied because of the techniques used in this method. The mentioned method is able to make the continuous IDA curves. IDA curves are summarized to the other curves that are named 16%, median and 84% IDA curves. Here,the median IDA curves are used to examine the performance assessment.

## INTRODUCIONOFLIMITSTATEON IDA CRVES

In this study, IDA is the instrument that is used to produce the required data for seismic performance assessment of the structure, and then the probabilistic frame that was proposed by Jalayer and Cornell is used for reliability evaluation. According to the mentioned procedure, limit states should be defined,therefore immediate occupancy and collapse prevention are considered for the structure. These limit states are defined at FEMA guidelines (FEMA, 2001), in which for special moment frames, IO is determined at $\theta_{max}$ = 2% and for CP there are two criteria. At CP, the first point, in which the local slope of the IDA curve is less than 20% of the primary slope, is determined for this limit state. The second one is that, if the former condition doesn't observed,$\theta_{max}$ = 10% is determined for CP.

## PROBABILITY-BSED DEMANANDCAPACITYFACTORDESIGN

In this section, seismic performance is assessed by the DCFD format namely the probabilistic demand and capacity factors based on the presented definition by Jalayer and Cornell (Jalayer and Cornell. 1998). In fact, DCFD is a probabilistic design framework and its concept is generated by a notion based on the annual frequency of exceeding the limit state after some alterations.

$$H_{LS}=v.P_{LS}=v.P[D\geq C] \tag{1}$$

Here $H_{LS}$ is the annual frequency of exceeding the limit state, P[D≥C] is limit state probability and v represents the hazard curve that is predicted for the site by earthquake risk analysis(Jalayer and Cornell, 1998). If it isassumed that the $H_{LS}$ is equal to the probability of exceeding the seismic drift demand of the drift capacity for specific limit state, finally it is possible to extract a drift demand hazard curve for calculating $H_{LS}$.

$$H_D(C)=v.P_{LS} \tag{2}$$

By use of the aforementioned discussion, the functional DCFD format in a closed-form is gained, which is capable of considering the uncertainties in the seismic assessment of structure. Here is a brief note on the derivation of DCFD format represented by Jalayer and Cornell:

$$\eta_{D|P_{LS_{Sa}}}.\, e^{\frac{1k}{2b}\beta^2{}_{D|Sa}}=\eta_C .\, e^{\frac{1k}{2b}\beta^2{}_c} \tag{3}$$

InEq. (3), all of the main components of DCFD format are represented in which some of them stand for both of the demand and capacity's median values and others stand for considering the deviation related to the mentioned parameters. Where $\eta_{D|P_{LS_{Sa}}}$ is the median drift demand for a given spectral acceleration, $P_{LS_{Sa}}$, corresponds to hazard level in the proximity of an acceptable limit state probability,$P_{LS}$. $\eta_C$is the median drift capacity, $\exp(\frac{1}{2}\frac{k}{b}\beta^2 D|Sa)$ is the Demand Factor, and $\exp(\frac{1}{2}\frac{k}{b}\beta^2 C)$ is the Capacity Factor (Jalayer and Cornell). Finally, to reach a simply formed equation, instead of the represented components in Eq. (3), some parameter are used; D stands for $\eta_{D|P_{LS_{Sa}}}$, C for $\eta_C$, $\gamma$ for $\exp(\frac{1}{2}\frac{k}{b}\beta^2 D|Sa)$ and Ø for $(\frac{1}{2}\frac{k}{b}\beta^2 C)$ (Jalayer and Cornell, 2003),

$$D.\ \gamma \leq C.Ø \tag{4}$$

In the above equation, both sides are the factored demand and capacity in which C and D are the mean values, andγ and φ are representative of the dispersion of their corresponding mean values. Finally, here, to investigate and discuss the effects of the considered uncertainties λ parameter is applied.

$$a = \frac{D.\ \gamma}{C.Ø} \tag{5}$$

Analyses are conducted according to the mentioned method andresults are divided in two parts; Prior to the construction and after the construction. Because of the excess of the outcomes related to the after construction results, they are displayed by two histograms (indicating the cumulative frequency of the DCFD parameters) for every performance levels:

1) Collapse prevention against 2/50 hazard level
2) Immediate occupancy against 50/50 hazard level

Prior to the construction of structure and with no uncertainties arising from the material properties or construction of structure, the value of λ is:

Table 4. λValues with no Uncertainties

| | | | |
|---|---|---|---|
| DCFD parameter | CP | Landa CP | 0.619338 |
| | IO | Landa IO | 0.950555 |

The given histograms are shown in Fig. 3&4 present the distribution of λ parameter for the after construction stage of the structure as affectedby the mentioned uncertainties atthe assumed performance level.

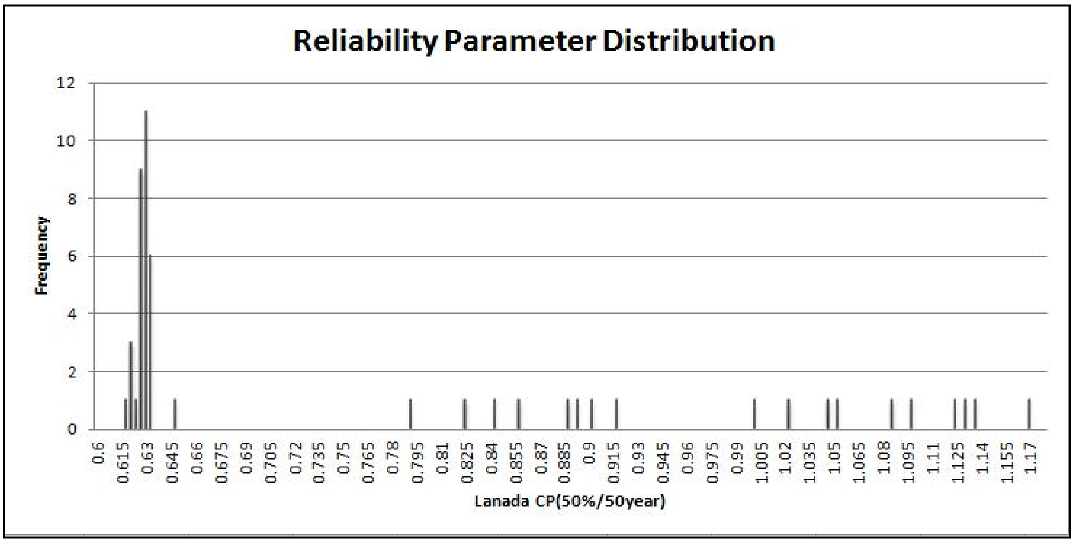



Figure 3. The λhistogram forthecollapse prevention against 2/50 hazard level

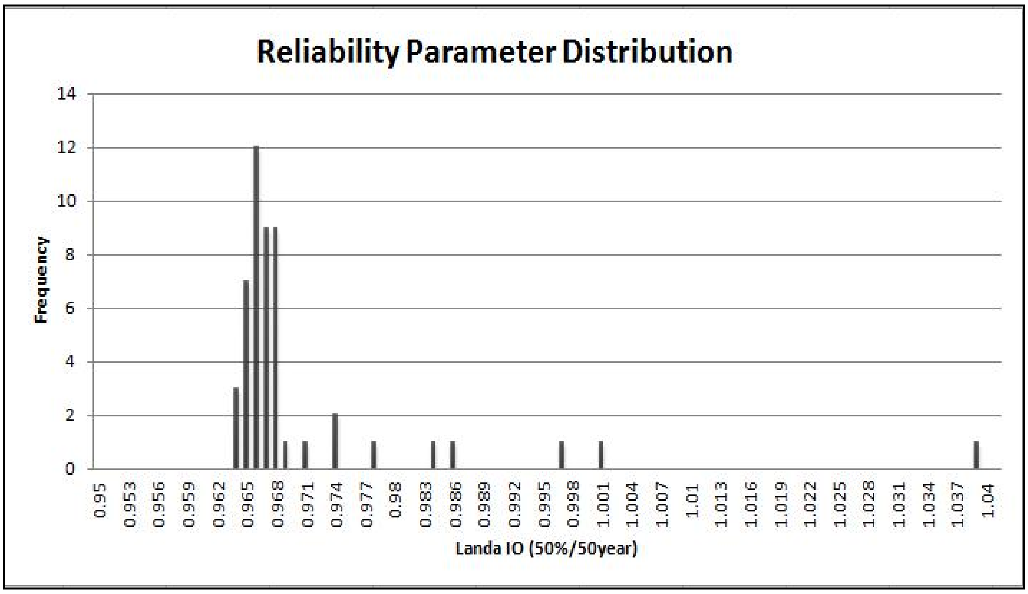



Figure 4. The λhistogram for the immediate occupancy against 50/50 hazard level

## CONCLUSION

In this study, the uncertainty, which involve the quality of workmanship (quality of construction and weld fabrication) that is affected in the behavior of the beam-to-column connectionsas well as mechanical properties such as Young modulus and yield-strength, is the parameter for considering those associated with structural steel framing parameters. Fifty random numbers were generated based on the mentioned statistical distribution.

Incremental dynamic analysis is utilized to assess the structural dynamic behavior of the frames and to generate the required data for performance based evaluations. Based on the IDA result and DCFD parameter, the building performanceis evaluated by examiningthe level of confidence in the building's ability to meet any desired performance objectives.This confidence level is assesable and obtaianble through evaluation of the factored demand-to-capacity, namely DCFD formt.

DCFD parameter for the IO performance level, as affected by uncertainties, shows few changes in comparison to the structure with no uncertainty. In IO performance level, structure is located within the linear behaviorand effects of the introduced uncertainties have been insignificant, so the DCFD showsfew changes. The DCFD of CP performance level, as affected by different uncertainties, shows lots of changes in comparison to the structure with no uncertainty. Finally it is shown that by using the above mentioned procedure for performance based evaluation, the DCFD parameter of special moment frames with uncertainties for the mentioned parameter in some cases, will be increased to 40%, which demonstrates the difference between the structure before and after the construction based on the accomplished seismic assessment. According to the relation between DCFD and confidence level(The annual frequency of exceeding the limit state),by increasing the DCFD value confidence level of the structure is reduced therefore ,the confidence level for CP is very diverse and has large reduction in comparison to the structure with no uncertainty. In IO limit state the confidence level has light difference.

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