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Preparing the Fragility Curve of Semi-Rigid Beam-to-Steel Column Connection

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Abstract

Background/Objectives: Examination of the connections in a steel structure is of paramount importance and lack of precision in design and implementation of connections in steel structures will cause not only failure in the connection itself but also devastating effects on structural members and thereby entire the structure**. Methods/Statistical Analysis**: In present study, structural safety is assessed through probabilistic methods and the work mainly tries to make a suitable platform for its performance within the useful life of structure. To this end, the experimental setup for testing these connections - which are proper agents of their performance in a building frame (connection place and a half of joint beam and column) - was modeled in the software ABAQUS and acceleration time history of the frame it is embedded in is included under a set of certain records of the desired site. **Results**: Connection in a structure with a special frame and moderate height was studied and initially it was determined to join levels of performance with a suitable criteria based on connection failure. Thereafter, the corresponding fragility curves were drawn and then compared with moment-rotation curve and cyclic loading based on joining models which are influenced by nonlinear static analysis and cyclic loading. **Conclusion/Application**: Pushover and cyclic loading curves of this connection were determined in order to specify level of performance of the connection as per the connection failure.

Keywords: Beam-to-Steel Column Connection, Fragility Curve, Nonlinear Static Analysis, Pushover Loading Curves, Semi-Rigidtion

1. Introduction

Detailed examination of the connections in a steel structure is of particular importance and lack of precision in design and implementation of connections in steel structures will cause not only failure in the connection itself but also devastating effects on structural members and thereby entire the structure. Based on available information, the collapses in steel structures are often reported due to poor performance of the connections[1–5].

Following Northridge earthquake (1994) some wide studies were carried out based on which plenty of regulations and standards were presented for many types of earthquake-resistant structural systems. In order to achieve a structure, the main components (e.g. beams and columns) are necessary to be accurately jointed so that the integrated performance can satisfy the desired productivity. In other words, the connections are in charge of power transfer from one structural member to another member or to a support. Development of design principles for main structural members is possible almost with no complexity and problem by the equations presented by design regulations which are relied on theoretical equations of Mechanics of structures. Therefore, analysis of the connections behavior of a structure often is along with specific complexities which often cannot be readily explained only by theoretical equations and requires experimental tests to ensure their behavioral accuracy[1]. Therefore, the accurate understanding of structural behavior of the connection and proper awareness of the way of power transferring by connection is necessary to design a secure and economic connection. Ensuring

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the correct power transfer and also feasibility and constructability should be taken into consideration in design of a connection in a steel structure[1].

2. Behavior of Connections

The major role of connections in steel structures is to transfer loads imported to stories through the beams to columns. Generally, the transferring forces by connections from beams to columns include axial force, shear force, flexural moment and torsional moment. Since axial and shear deformations are low in most of the connections used in steel frames compared to rotational deformations, thereby such effects are typically eliminated in the study[6]. Therefore, rotational deformations are commonly considered for practical objectives in steel structures. As it can be seen from Figure 1, by applying the moment M for a connection, the rotation Ø is made into the connection. In fact, this rotation indicates changing the angle between the beams and columns relative to the situation before applying moment[6].

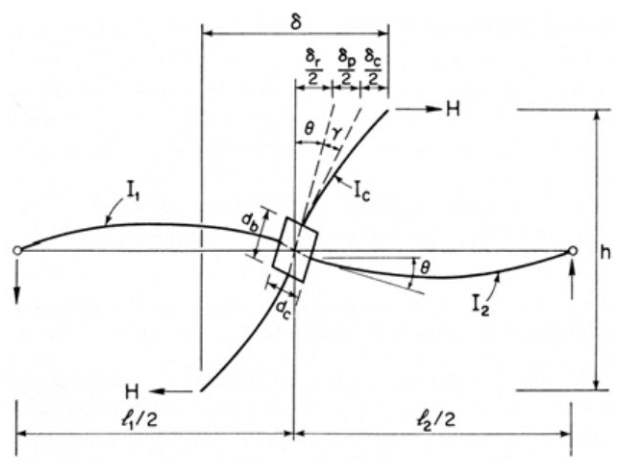

Figure 1. Rotational deformation of the connection caused by the force H in the column[7].

$$\delta_r = \frac{h^2\left(1-\frac{2d_c}{l_1+l_2}\right)}{6E\left(\frac{I_1}{l_1-d_c}+\frac{I_2}{l_2-d_c}\right)}V_{col} \qquad (1)$$

$$\delta_c = \frac{(h-d_b)^3}{12EI_c}V_{col} \qquad (2)$$

ä2)) e>1 into the connection. In fact, this rotation indicate

Vcol = column shear force, kip

h = story height (centerline dimension)

$$\delta_p = \frac{(h-d_b)\left(\frac{h}{d_b}-1\right)}{Gt_pd_c}V_{col} \qquad (3)$$

Ic = column moment of inertia.

db = depth of beam.

dc = depth of column.

tp = thickness of joint panel zone.

3. The Fragility Curve of the Connection

Assume that we are provided with a certain connection whose effective parameters on dynamic behavior is clear and we are asked to determine the seismic intensity which leads this connection to a desired level of performance. In other words, we are asked to show what a level of performance this connection reaches in case of earthquake occurrence with a certain intensity[8].

If this question should be answered carefully, lots of uncertainties should be taken into consideration regarding both stimulation of earthquakes and the seismic response of moment frame. Since the earthquake is caused by the waves which are made by irregular slip along faults, followed by reflections, refractions and multiple random attenuations and then reach the structure after passing different structures of the ground. Such uncertainties are expected in frequency content, intensity, duration and input energy of the earthquake, etc. If instead of one connection, a set of connections of a moment frame is existing, other uncertainties should be considered such as size and mass of the structure and strength of materials used. It seems that the most reasonable way is to address the structural performance in probabilistic form.

In other words, instead of addressing the seismic intensity which leads the structure to a special level of performance, the probability should be addressed for that level of performance per diverse seismic intensities. This difference can be seen in Figure 2, Figure 2(A) and Figure 2(B) address probabilistic and forms, respectively. Figure 3 depicts general form of fragility curve.

If the parameter R indicates structural response and LSi is level of performance or limit state associated with the parameter R and also IM is one of the parameters indicating the seismic intensity and S is the desired

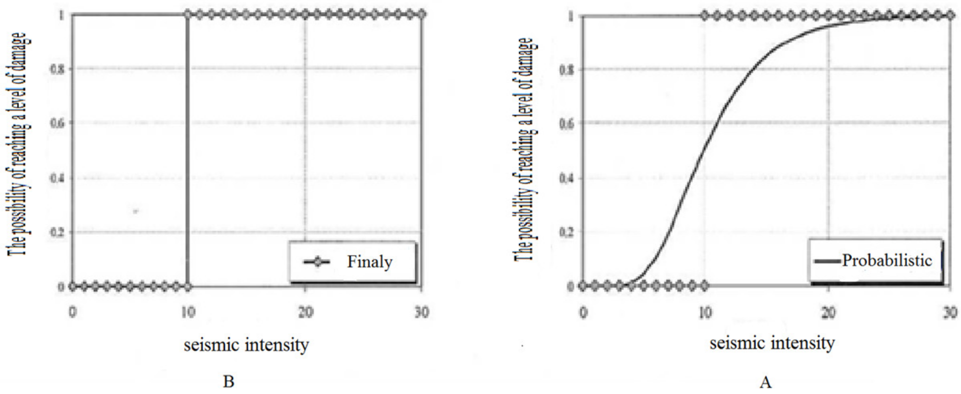

Figure 2. The difference of definitive and probabilistic evaluations for the connection performance; (A) Probabilistic, (B) Definitive.

intensity then the concept of fragility curves can be shown as follows:

$$Fragility = P(R \rangle LS_i \,/\, IM = S) = \int_0^{\infty} f_{RS}\,(r\,/\,s)\,dr \qquad (4)$$

FRS (r/s) is conditional probability density of seismic intensity IM=S[10].

R can be for example either shift or rotation or curvature or yield stress in a member. In this case, LSi will be of that type. IM also can be addressed in terms of Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and Spectral Acceleration (SA) or any parameters which suggest seismic intensity.

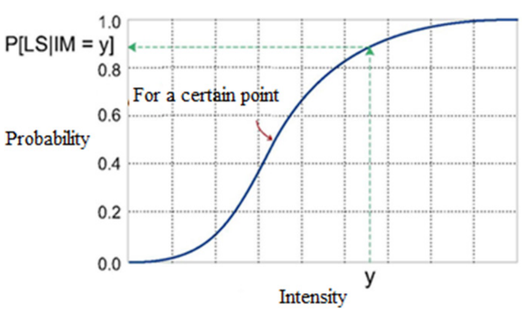

Figure 3. General form of fragility curve[9].

Therefore, three types of parameters influence fragility curve as follows:

- The parameters associated with seismic response.
- Level of performance of limit state or damage associated with the response parameter.
- Seismic intensity.

In present study, three limit states were used to address the connection response and level of damage. Moreover, level of performance of the connection were defined based on regulations and researches of other authors.

General representation of fragility curves and the corresponding interpretation per fixed seismic intensity can be observed in Figure 3. Since with increased seismic intensity, the damage imported on the structure will be increased, this curve continuously is expected to be ascending. Also by more critical limit state, fragility will be decreased. For example, in other words, the curve associated with limit state of initial yield is permanently above the curve associated with full failure.

4. How to Determine Performance Points of the Modeled Connections

In elastic analyses, the assumption to use rigid connection beams and columns and to use axis of beam and column which are connected in the column center has resulted in favorable analyses. Such an assumption can be reasonable for nonlinear analysis of connection prior to yield of panel zone, beam or column and achieve escape angle θ_{SD} (Table 1). In order to calculate yield capacity of beam, column or the panel zone, the desired resistance values ought to be used. Each key escape angle of Figure 4 is defined based on failure amount observation in the connection of beam to column. Table 1 qualitatively represents failures amount for each level of performance as per following observations[11]:

- Initializing the local buckling of beam flange.
- Reduction of bending strength of the complex as much as lower than capacity of nominal bending strength.
- Initialization of failure of bolts, welds or base metal which significantly decreases the resistance of the complex.

- Complete failure of the connection which results in connection's inability to transfer gravity loads.

Table 1. Description of evaluating the performance of a connection set (If a connection has brittle behavior, then θ_{IO} will approach θ_{SD}.)[11]

Escape angle	Level of performance	Description
	IO	The lowest escape angle which causes the 1, 2 or 3.
	CP	The escape angle which causes the behavior 4.
	-	The lowest escape angle which causes the behaviors 2, 3 or 4.

The models which have used beam and column axes, calculate escape angle as a pre-estimated value relative to the models with panel zone. However, although it is possible to not include the models with panel zone in performance evaluation, realistic estimation based models hardly can satisfy performance goal relative to enhancement of confidence level of the structure. The analyses conducted on the frames with diverse connection models, have resulted in a relatively similar escape angle. Nevertheless, they had a considerable impact in recognition of plastic deformations concentration place such as in panel zone or beam or a combination of them[11].

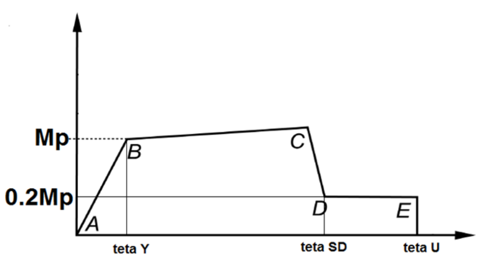

Figure 4. The relationship between moment-escape angle of a quite rigid connection[5].

5. Dynamic Analys is of the Frame for Selected Records

Initially, 3D model of 4-story frame was modeled in the software SAP and the selected records were applied in the scaled form with an incremental step of 0.1. The modeled frame were put under nonlinear time-history analysis for each of these earthquakes and for each earthquake as many as the number of incremental steps up to achievement of quite failure of connection. Following each analysis in each incremental step of PGA for each scaled earthquake, the shear value in the middle of connection beam was taken and then applied to the micro-connection model in the software ABAQUS as a history of the shear versus time. Also in this stage, a history of connection duration versus time was gained so that the maximum connection duration was taken in each stage. Thereafter, by logging this maximum duration in each stage for each certain earthquake, incremental dynamic analysis curves were drawn. Ultimately, these stages were iterated for the remaining records.

6. Selection of Suitable Seismic Record Sets

The first step in fragility curves drawing process is to provide a set of seismic mappings in such a way that the set indicates seismicity of desired area. In fact, if sufficient seismic mappings are logged in the desired area, then that set will be used and otherwise, artificial seismic mappings can be used. These mappings are developed based on the seismic intensity, distance to the center, the route drop and local soil conditions.

Another tip regarding seismic mappings is the higher number of studied earthquakes, the more precise results of fragility curve. Earthquakes accelerograms are employed in dynamic analysis of the structures versus earthquake in IDA method which results in seismic intensity curves (required by structural engineering).

Initially, a set of seismic records should be selected that the dynamic analysis under their impact is done on the structure. If required information is logged for the required, then it can be used; otherwise, records of other areas should be used which are similar in terms of geotechnical factors. In present study, 10 seismic records were used to conduct analyses. These records were used from the record bank which is used by SAC institution in research works. Table 2 shows specifications of these accelerograms.

Spectrum of above earthquakes response was drawn with a damping of 5% by the software Seismo Signal. Spectra of above earthquakes were scaled for PGA=1g in order of number and then were drawn in following figure.

From drawing of earthquakes spectra in Figure 5, it can be concluded that for the structure with a period lower than 1 second, a severe acceleration will be applied on the structure and for the structures with a period between 1 and 2 seconds, a lower acceleration than the

Table 2. Specifications of the accelerograms for selected earthquakes

Record Number	probability of exceedance	Record Name	Points Number	Step (Second)	Duration (Second)	PGA (g)
1	10% in 50 years	LOMA PRIETA 10/18/89 00:05, AGNEWS STATE HOSPITAL	8000	0.005	39.975	0.172
2	10% in 50 years	SUPERSTITION HILLS 11/24/87 13:16, BRW	2221	0.01	22.11	0.155
3	10% in 50 years	SUPERSTITION HILLS 11/24/87 13:16, EL CENTRO IMP CO CENTER	8000	0.005	39.975	0.357
4	10% in 50 years	SUPERSTITION HILLS 11/24/87 13:16, PLC	2223	0.01	22.22	0.185
5	10% in 50 years	SUPERSTITION HILLS 11/24/87 13:16, WESTMORELAND FIRE STATION	8000	0.005	39.995	0.171
6	10% in 50 years	LOMA PRIETA 10/18/89 00:05, CAPITOLA	7991	0.005	39.95	0.443
7	10% in 50 years	NORTHRIDGE EQ 1/17/94, 12:31, LA - CENTINELA	2999	0.01	29.98	0.321
8	10% in 50 years	IMPERIAL VALLEY 10/15/79 2316, CALIPATRIA FIRE STATION	7905	0.005	39.52	0.078
9	10% in 50 years	IMPERIAL VALLEY 10/15/79 2316, EL CENTRO ARRAY #12	7802	0.005	39.005	0.116
10	10% in 50 years	LOMA PRIETA 10/18/89 00:05, GILROY ARRAY #7	7990	0.005	39.005	0.225

former but higher than those with a period greater than 2 seconds will be applied. Now, if structural stiffness can be decreased by some tricks and thereby make the structure more formable, then the period of the structure will be increased due to have inverse ratio with stiffness and thereby the structure will experience lower acceleration.

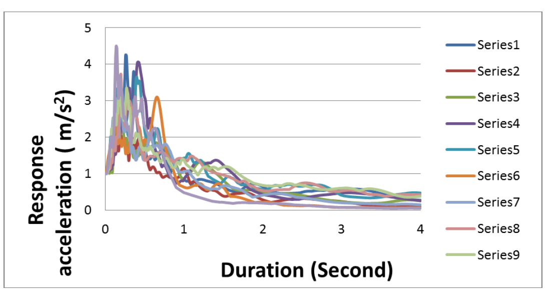

Figure 5. Response spectra of the used earthquakes with a damping of 5% for PGA=1g.

7. Geometry of the Model

Finally, the work focused on the connection behavior but not on the connected members. To this end, sections of beam and column are shown in following figures.

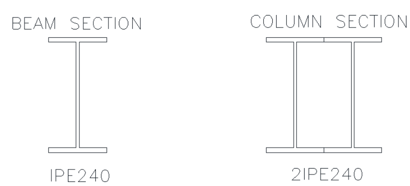

Figure 6. Sections of beam and column.

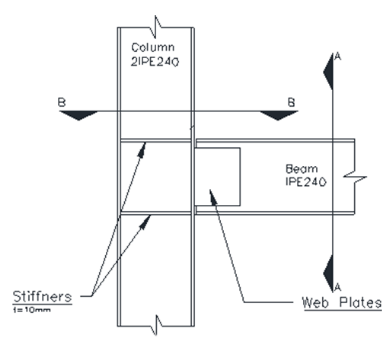

Figure 7. The studied semi-rigid connection in present work.

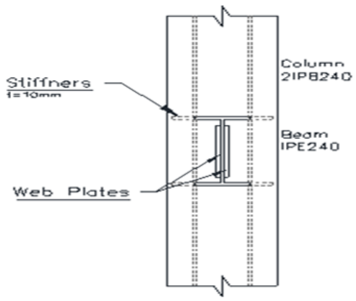

Figure 8. The section A-A of the studied semi-rigid connection.

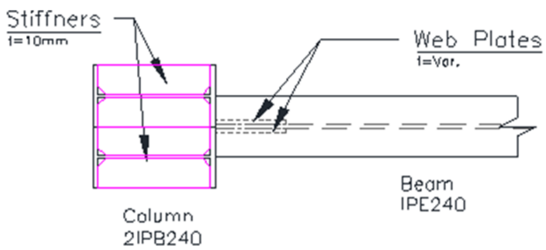

Figure 9. The section B-B of the studied semi-rigid connection.

8. Meshing the Model

8.1 Steel Beams and Columns IPE240

Double-node three-dimensional shell elements C3D8R were used for steel beams, columns and sheets. The length of each element is set to almost 50mm.

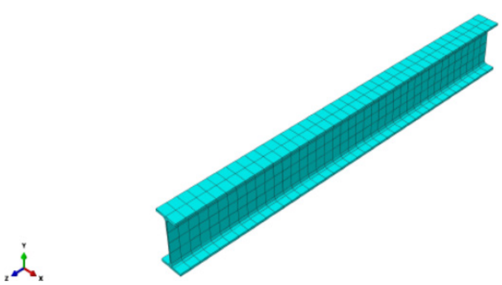

Figure 10. Meshing the element beam.

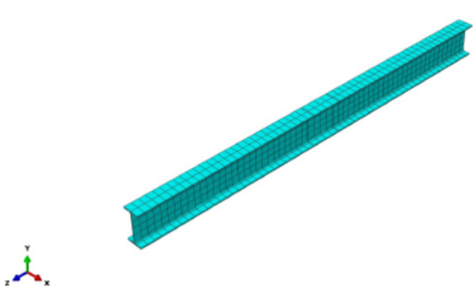

Figure 11. Meshing the element column.

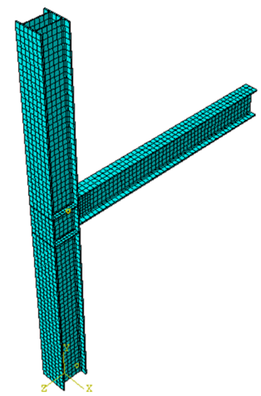

Figure 12. General meshing of the model.

9. Drawing the Pushover Curve

At the next stage, the pushover curve was drawn for the studied connection. This curve is depicted in Figure 13 in such a way that the shift equal to final cycle shift of cyclic loading was applied to end of the beam. This was performed to ensure accuracy of performance points and to compare determined points using cyclic loading curve with the performance points which are obtained through the pushover curve.

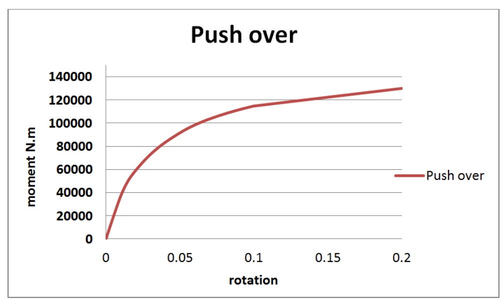

Figure 13. Pushover curve of the connection model.

10. Cyclic Loading Curves and Comparison with Pushover Curve

Initially, by applying the loading pattern SAC for the connection in cyclic form, moment-rotation Hysteresis curves were drawn for the connections. Point of impact of the load application along with the model geometry are shown in Figure 14 in such a form that a half of column length of upper story of the connection and a half of the down story of the connection were modeled with a half of the beam in the model.

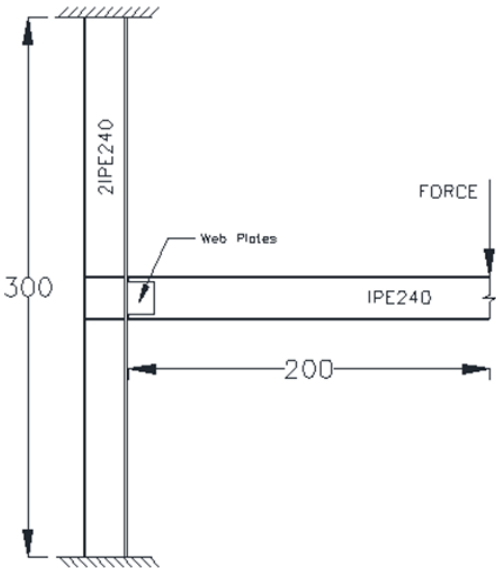

Figure 14. Overview of the connection model and the place to apply shear history.

It can be seen from Figure 15 that pushover curve at elastic part in initial stiffness of the connection is in line with cyclic loading curve and then, in nonlinear part, it is consistent with pushover curve following the increased loading cycles of peak moment in each loading cycle. This indicates that pushover curve is the push of peak moment values of cyclic loading curve and such a consistency leads to a conclusion that cyclic loading can be used to determine performance points and also the software model accuracy is approved due to this consistency.

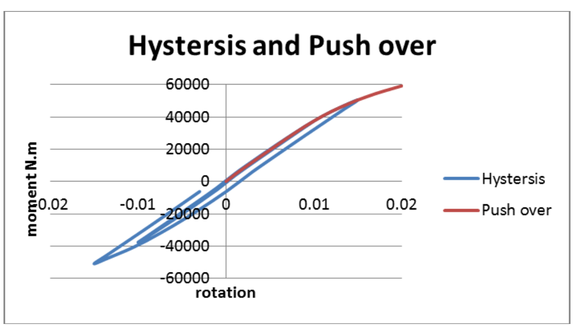

Figure 15. Moment-rotation hysteresis curve and pushover curve of the connection.

11. Drawing IDA Curve of the Connection

After modeling of the connection by a method mentioned in previous sections, dynamic analysis was done one finite element model of the studied connection using each of represented accelerograms. Following each analysis, the maximum rotation in the connection in each step of PGA increase was determined and logged for each earthquake using outputs of the software ABAQUS. Finally, Incremental Dynamic Analysis (IDA) curve of the connection was obtained as follows:

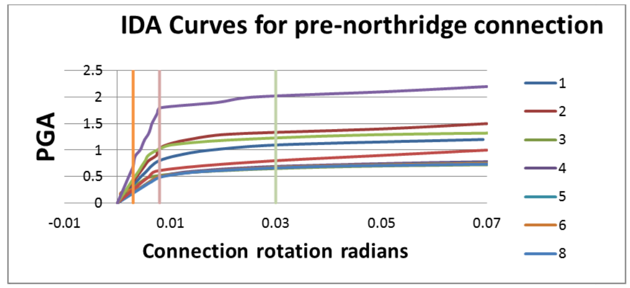

Figure 16. IDA curve of the connection.

12. Drawing Fragility Curves of the Studied Connection

Now, fragility curve can be developed for each of failure modes of the studied connection based on parameters of regression analysis and demand-seismic intensity. As such, fragility curves of the studied connection were drawn and a conclusion was made about its vulnerability. In present work, nonlinear regression analysis and log-normal distribution were used to draw fragility curves.

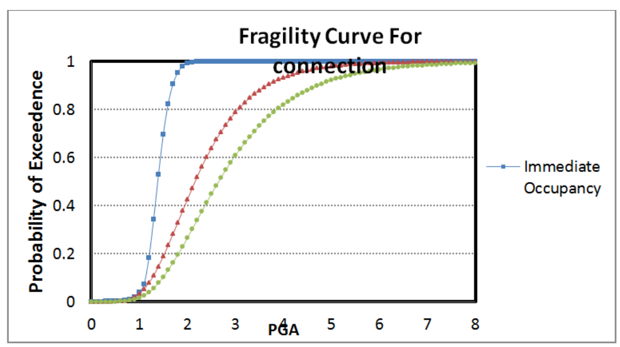

Figure 17. Fragility curve of the connection for diverse performance levels.

From comparison of fragility curves for diverse levels of performance in Figure 17, it can be seen that up to the PGA value, about 1g of fragility curve of all levels of performance have an approximately same probability of close to zero. In other words, up to the PGA of about 1, the connection will reach none of levels of performance of LS, IO and CP and this indicates the connection strength and precise results of fragility curve drawing because in case of modeled frame and drawn fragility curve of the frame, the frame will reach level of performance of IO in a lower PGA. Given the drawing of failure curves in previous chapter, the probability of each level of performance of each connection can be studied for a specific PGA. It can also be seen that for a PGA of about 2g, the connection in IO level reaches to a probability of 100%. It means that from this PGA value, the connection will be no more efficient in IO level while for the performance levels of CP and LS, it can be seen that probabilities of exceedance are roughly closer to each other and have a more significant value.

13. Conclusion

- Comparison of IDA curves in Figure 16 shows that most of the curves in PGA of below 1g have reached the CP level of performance and also 90% of IDA curves below 1.5g have reached the CP level of performance. Only the seismic record No.10 has reached the CP level of performance with a PGA of 2g. All of these evidences show that log-normal distribution of IDA curves has a skewness to the right and middle of this distribution occurs in the PGA close to 1g. Finally, it indicates that semi-rigid connections with low fixed-end percentage have far lower capacity compared to fixed connections.
- It can be concluded from fragility curves for diverse levels of performance that semi-rigid connection shortly losses its strength due to have insufficient formability for IO level of performance. As it can be seen from comparison of fragility curves for diverse levels of performance, up to the PGA of about 1g, fragility curve of all levels of performance have an approximately same probability of close to zero. In other words, up to the PGA of about 1g, the connection will reach none of levels of performance of LS, IO and CP and this indicates the connection strength and precise results of fragility curve drawing because in case of modeled frame and drawn fragility curve of the frame, the frame will reach level of performance of IO in a lower PGA. It can also be seen that for a PGA of about 2g, the connection in IO level reaches to a probability of 100%. It means that from this PGA value, the connection will be no more efficient in IO level while for the performance levels of CP and LS, it can be seen that probabilities of exceedance are roughly closer to each other and have a more significant value.
- From comparison of pushover curve of this connection with other accredited existing connections in FEMA, it can be seen that the area under the moment-rotation curve for this connection is far lower than fixed and accredited connections. Besides, the area under the moment-rotation curve is directly associated with formability. Therefore, it can be concluded that formability of this connection is lower than that of fixed and accredited connections.
- From comparison of pushover curve and cyclic loading curve, it can be seen that both diagrams are well-matched and beside, moment-rotation curve is the push of cyclic loading curve. Finally, consistence of these two curves approves the model accuracy and present study.

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