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COLLAPSE ASSESSMENT OF TALL CONCRETE MOMENT FRAME INDUCED BY COLUMNS SHEAR FAILURE DUE TO NEAR FIELD AND FAR FIELD EARTHQUAKES

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ABSTRACT

Due to the possible significant equipments and the large number of inhabitants, safety of high-rise buildings has always been a main concern. Considering the disastrous effects of a possible failure, structural performance of these buildings against instantaneous loads (like earthquakes) should be investigated. This study is aimed at investigation of failure extension and total collapse of high-rise concrete moment frames, which suffers from insufficient shear strength at columns. To this aim a case study (constructed in 1970) was considered. In order to evaluate structural behavior against lateral loads, push over analysis was performed; then its performance against near-field and far-field earthquakes evaluated. The structure was analyzed with incremental dynamic analysis method (IDA) using 56 near-field earthquakes records and 44 far-field earthquakes records; and fragility curves were obtained. The results indicated that shear failure in columns can affect the overall behavior of structure and probability of collapse is higher in near field earthquakes than far field earthquakes.

INTRODUCTION

Seismic design provisions and construction practice in regions of high seismicity have been based primarily on an understanding of the anticipated behaviour of low to mid rise construction. In extrapolating design and detailing provisions for use in high rise construction, structural systems have been limited in height or not permitted [7].

Due to high vibration period of tall buildings, base shear coefficient of these structures is so low compared with conventional structures; but if tall building structures exposed to near field earthquakes, seismic responses can be amplified and probability of collapse increases [8].

Laboratory studies and observations after the earthquakes, have demonstrated that the reinforced concrete columns which have low shear reinforcements with high spaces are vulnerable in terms of occurring shear failure during the earthquakes [9]. Shear failure in the reinforced concrete columns causes reduction of the axial bearing capacity of the elements; although the process of this capacity reduction is not known well. When the bearing capacity of a column decreases due to the shear failure, it is necessary that surrounding columns be able to bear the shear loads of the damaged column otherwise the failure develops and may lead to a general collapse of the structure [3].

By increasing awareness about inelastic behaviour and observing the occurred failure due to earthquakes, scientists came to conclusion that the moment frames, which have been built before 1975, have fundamental shortages in the field of seismic resistance [1]. Abbie Liel (2008) by studying

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about the reinforced concrete structures with moment frame system, which have been made before 1970 in California, found that these are among non-ductile structures because these structures have been built before applying the terms and details of building's seismic regulations, so these are highly vulnerable against seismic loads.

One of the fundamental differences between elements of the reinforced concrete moment frames that have been built before 1970 and new moment frames is the amount of usage and details of shear reinforcements. Lack of the shear reinforcements and their inappropriate details in older structures, leads to brittle behaviour of the reinforced concrete elements and probability of the shear failure in these elements under seismic loads, increases. Also concrete confining hasn't done well due to the lack of shear reinforcements and it leads to a reduction of the deformation capacity of the reinforced concrete members [1]. It is worth to mention that mechanism of the shear failure is non-ductile and instantly; consequently, the bearing capacity of the member is reduced and may lead to axial failure of the members and general instability. Therefore, the occurrence of this type of the failure can be associated with harmful effects.

Sezen [9] and Lin [6] evaluated numbers of reinforced concrete columns that were vulnerable due to the shear failure under the periodic lateral loads in the real scale, as far as gravity bearing of the columns reaches zero. The results showed that the axial failure of the columns may not occur immediately after the shear failure and column's drift ratio at the moment of occurring axial failure depend on the axial stress of the column's section and the amount of shear reinforcements.

This study aimed to evaluate the performance of the tall reinforced concrete moment frame that vulnerable to shear and axial failure of columns. For this evaluation have been tried to model the effects of shear and axial failure of the columns using the results which have been proposed by Elwood and Moehle.

PROPERTIES OF THE STUDIED FRAME

The tall reinforced concrete moment frame investigated in this study has 26 stories and is located in Tehran. This moment frame has fixed supports. Compressive strength of concrete and yield strength of reinforcing bars are 4 ksi and 60 ksi respectively. Figure 1 shows stories plan and view of the studied frame. Table 1 and Table 2 show section properties of columns and beams respectively.

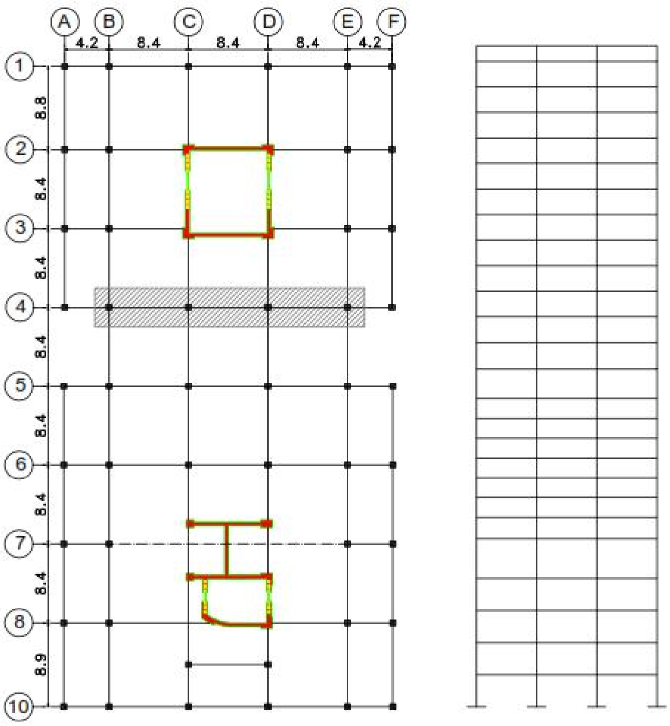

Figure 1. Plan of the stories and view of the studied frame

Table 1. Section properties of columns

Story	Height (m)	Perimeter Columns Section			Central Columns Section		
		Width×Depth (cm)	ρ	ρ”	Width×Depth (cm)	ρ	ρ”
1	4.52	200×120	0.011	0.0045	120×120	0.017	0.0065
2	4.52	140×120	0.01	0.0048	120×120	0.013	0.0056
3-4	4.52	140×120	0.01	0.0048	120×120	0.01	0.0056
5	5.58	140×100	0.01	0.0064	100×120	0.012	0.0056
6	3	125×100	0.011	0.0054	100×120	0.012	0.0056
7	2.8	125×100	0.01	0.0054	100×100	0.014	0.0045
8	2.8	125×90	0.011	0.0054	90×100	0.013	0.0062
9-12	2.8	125×90	0.011	0.0054	90×100	0.01	0.005
13	4.1	125×90	0.011	0.0054	90×100	0.01	0.005
14	3.8	125×90	0.011	0.0054	90×100	0.01	0.005
15-18	3.6	125×70	0.012	0.0054	90×90	0.012	0.005
19-25	3.6	80×70	0.01	0.008	80×70	0.01	0.008
26	2.2	40×70	0.01	0.0056	40×70	0.01	0.0056

Table 2. Section properties of beams

Story	Width×Depth (cm)	ρ	ρ’	ρ”
1-10	140×50	0.006	0.009	0.0028
11-21	100×50	0.007	0.01	0.0032
22-26	80×50	0.003	0.008	0.0032

NONLINEAR MODELING OF THE STRUCTURE

Nonlinear model of the studied structure was built as a two dimensional structure with several degrees of freedom in OpenSees software. Nonlinear modelling of beams and columns of the moment frame was performed as a concentrated plasticity. Therefore, the central area of beams and columns were modelled by using “Elastic Beam-Column” elements and plastic joints were modelled by using “Zero Length” elements.

In order to evaluating performance of reinforced concrete structures that vulnerable to shear failure, it is necessary to use suitable model that considering shear and axial failure of columns [8]. So, in this study limit state material was used. In fact limit state material which was proposed by Elwood and Moehel is a hysteretic material with one or more limit curves. Figure 2 displays the performance of the limit state material. As Figure 2 shows, a flexural backbone curve was assigned to column plastic hinge. If displacement of column increases and backbone curve reaches the shear limit curve, it means that shear failure occurs and behaviour of column is modified by decreasing in stiffness and strength until the strength reaches zero. When the strength of column reaches zero, gravitational instability occurs and column fails. Shear and axial limit curves are related to the cross section properties of columns that define according to Eq.(1) and Eq.(2) respectively.

$$\frac{\Delta_s}{L} = \frac{3}{100} + 4\rho'' - \frac{1}{500}\frac{v}{\sqrt{f_c'}} - \frac{1}{40}\frac{P}{A_g f_c'} \geq \frac{1}{100} \quad (1)$$

$$\frac{\Delta_a}{L} = \frac{4}{100} \times \frac{1+(\tan\theta)^2}{\tan\theta + P\left(\frac{S}{A_{st} f_{yt} d_c \tan\theta}\right)} \quad (2)$$

In Eq.1 and Eq.2, $\frac{\Delta_s}{L}$ and $\frac{\Delta_a}{L}$ are drift ratios corresponding to shear and axial failure respectively. ρ'', v, f_c', P, A_g, θ, A_{st}, f_{yt} and d_c are transverse reinforcement ratio, shear stress, concrete compressive strength, axial force, cross section, shear crack angle, transverse reinforcement cross section, transvers reinforcement yield strength and depth of concrete core respectively.

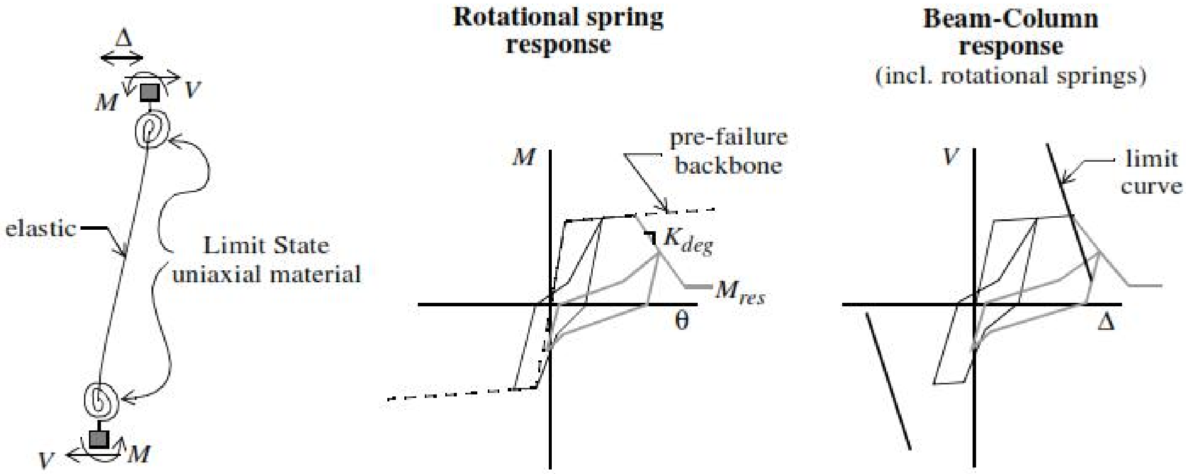

Figure 2. Performance of the limit state material to model the shear and axial failure of the column [2]

The hysteretic behaviour of columns was determined by using Haselton's relations [5]. Shear and axial limit curve of columns were defined according to Eq. 1, Eq.2 and columns cross section properties.

The Hysteretic material was used for consideration of non-linear behaviour of the beams. Similar to columns, moment and rotation parameters of beams were determined by using Haselton's Relations [5]. According to the recommendations of ATC-72, rigid end offset were used to model the beam-to-column connections [7]. In order to consider the effects of the cracked reinforced concrete sections, results of Haselton's studies which connecting reduced stiffness coefficient to axial member force, were used [7].

EIGEN VALUE ANALYSIS

In order to scaling earthquake records, vibration periods of the structure should be determined. Eigen value analysis was performed and vibration period of first, second and third vibration mode calculated equal to 3, 1.2 and 0.64 seconds respectively. Figure 3 shows stories drift ratio in first vibration mode.

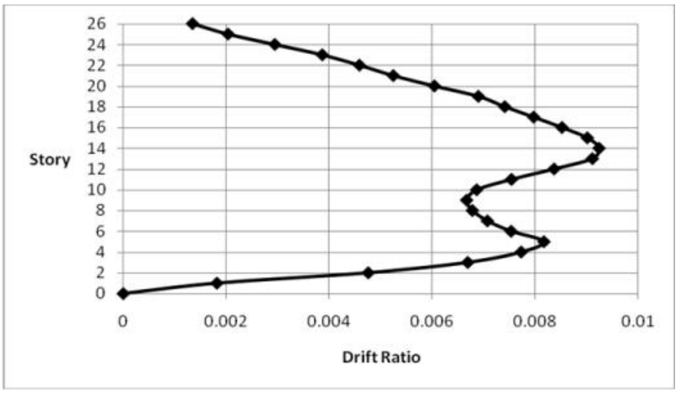

Figure 3. Stories drift ratio in first vibration mode

As figure 3 displays, 13th, 14th and 15th stories are critical stories. In accordance to drift ratio, can be expected shear and axial failure getting started form critical stories and then extend to other stories.

PUSHOVER ANALYSIS

In order to evaluating structure's behaviour under lateral loads, push over analysis has done. First, the structure has analyzed under gravity load combination according to Eq.(3) [7]. Then stories of frame have pushed under lateral loads as same as the first mode shape of the frame and push over curve has plotted.

$$DL + 0.2LL \tag{3}$$

In Eq.(3), DL is dead load and LL is live load of structure.

Figure 4 shows push over curve (base shear versus maximum inter story drift ratio) of the frame. As figure 4 shows, after shear failure of columns in critical stories, inter story drift ratio increases and lateral strength decreases until axial failure mode has occurred.

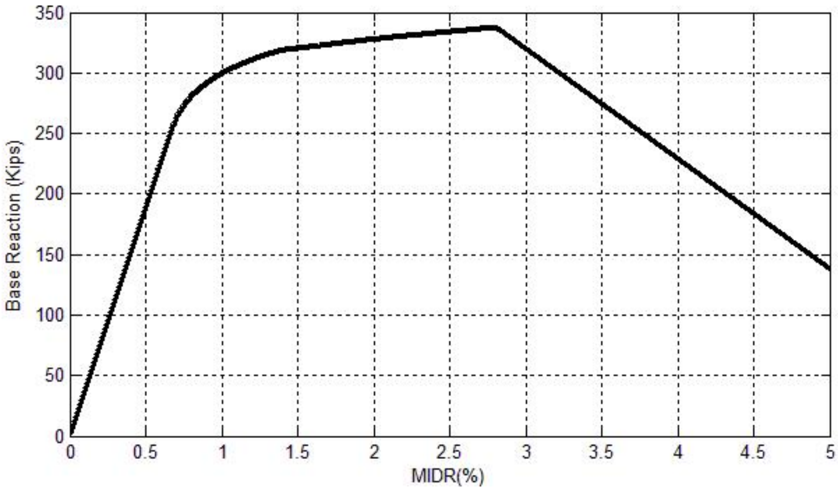

Figure 4. Push over curve of frame (Base shear versus maximum inter story drift ratio)

Figure 5 shows push over curve (base shear versus roof displacement) of frame. As figure 5 shows, after shear failure of columns in critical stories, total lateral strength immediately reduced. According to figure 5, ductility capacity of structure can be determined.

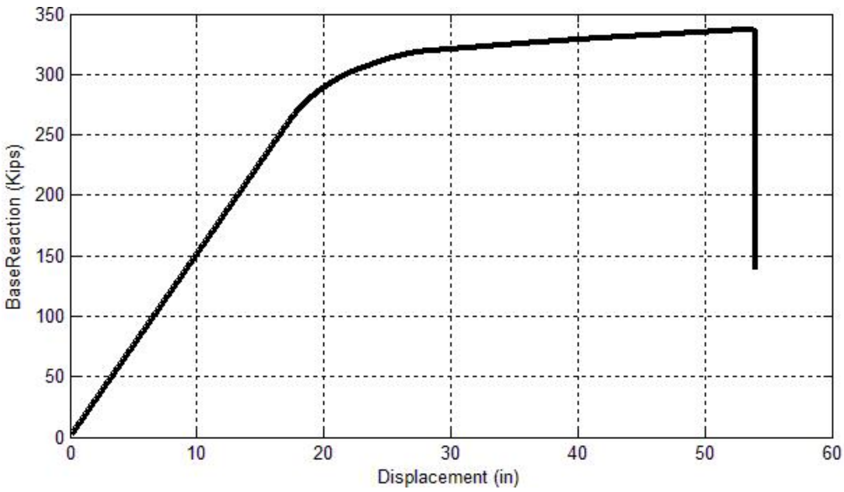

Figure 5. Push over curve of frame (Base shear versus roof displacement)

The period based ductility for a given structure is defined as the ratio of ultimate roof drift displacement to the effective yield roof drift displacement [4]. Eq.(4) shows calculation of period based ductility of structure.

$$\mu_T = \frac{\Delta_u}{\Delta_{y,eff}} \tag{4}$$

Ultimate roof drift displacement is a displacement that base shear achieved 80 percent of base shear capacity and effective yield roof drift displacement is defined by Eq.(5).

$$\Delta_{y,eff} = C_0 \frac{V_{\max}}{W}\left[\frac{g}{4\pi^2}\right]\left(\max(T, T_1)\right)^2 \tag{5}$$

Where C_0 relates fandamental mode displacement to roof displacement, $\frac{V_{\max}}{W}$ is the maximum base shear normalized by building weight, g is the gravity constant, T is the fandamental period and T_1 is the fandamental period using eigen value analysis. Ultimate and yield displacement of frame were calculated as 53.99" and 17.45" respectively, so period based ductility is 3.1.

INCREMENTAL DYNAMIC ANALYSIS

Incremental nonlinear dynamic analysis (IDA) is kind of a parametric analysis that has been used for comprehensive evaluation of the structure's behaviour under seismic loads. In this analysis, one or several earthquake records affect the structure. In order to place the structure in an appropriate range of the earthquake intensities, numerical coefficients are applied to the earthquake records. IDA leads to production of a curve which displays the structure's response under alternatives earthquake intensity known as IDA curve [10].

Earthquake records used for IDA analysis have been chosen according to FEMA P695. This record series consists of 28 pairs of near field and 22 pairs of far field earthquake's records [4].

First step of incremental dynamic analysis is choosing appropriate intensity measure. Intensity Measure (IM), characterizing the strength of an earthquake ground motion, is used to predict the seismic responses of a structure. Recent studies have demonstrated that using Sa(T1) as the seismic scaling index for near field ground motions may introduce large variability in the estimated seismic demands [11]. FEMA P695 introduce first mode spectral acceleration as suitable intensity measure for collapse assessment of structures; but in order to higher vibration modes effects in behaviour of tall building structures, it seems using of first mode spectral acceleration as intensity measure is not appropriate. Furthermore, if pulse period of near field earthquake record is in the range of higher vibration modes of structure, disability of first mode spectral acceleration in evaluating of structure's performance will be evident. So, in order to consider higher vibration modes effects in behaviour of structure, modified intensity measure that proposed by Yahya Abadi and Tehranizade [11] was used. Eq.(4) shows the modified intensity measure that were used in this study.

$$(S_{res})_{rms} = \left[\frac{1}{n}\sum_1^n S_{res}^2(T_i)\right]^{0.5}, T_a \le T_i \le T_b \tag{4}$$

$$T_{i+1} = T_i + \Delta T$$

In Eq.(4), $(S_{res})_{rms}$ is root mean squre of spectral responses that is proposed as modified intensity meseare by Yahya Abadi and Tehranizade. $S_{res}(T_i)$ is spectral psudo acceleration in T_i and ΔT is the time step of spectral response monitoring. Studies have shown that range of spectral response monitoring for tall building is $[0.3T_1, 1.9T_1]$ that T_1 is first vibration mode period. İn order to reduce errors, ΔT is taken as 0.1 second in this study.

To perform the IDA analysis, hunt &fill algorithm was used to enhance the intensity of applied records. This algorithm was developed in order to present the IDA curve of the structure with reasonable accuracy for each earthquake record by using of 12 nonlinear analysis [10].

In order to draw the collapse fragility curves of the structure, the limit state of collapse was considered in 3 ways, as follows:

- The maximum drift ratio among the story is 10%
- The last point where the slope of the IDA curves reaches to less than 20 % of initial slope
- Numerical divergence

Figure 6 and 7 show IDA curves of frame under near field and far field earthquake records respectively.

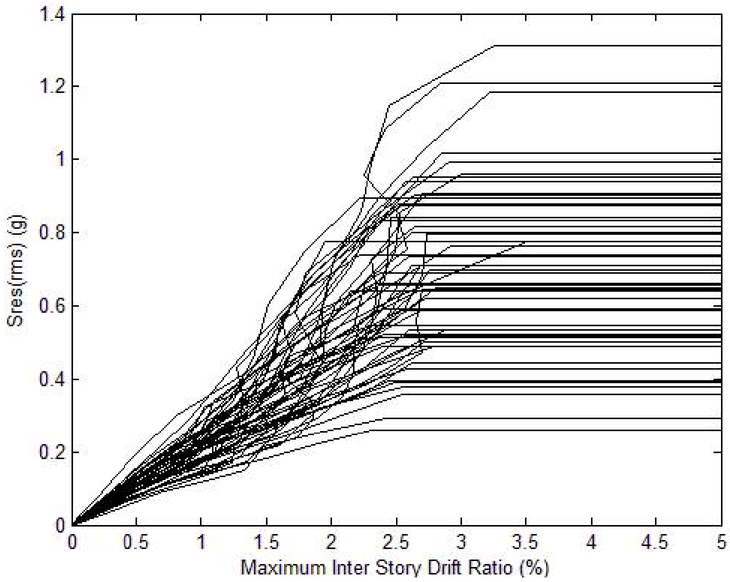

Figure 6. IDA Curves of frame under near field earthquake records

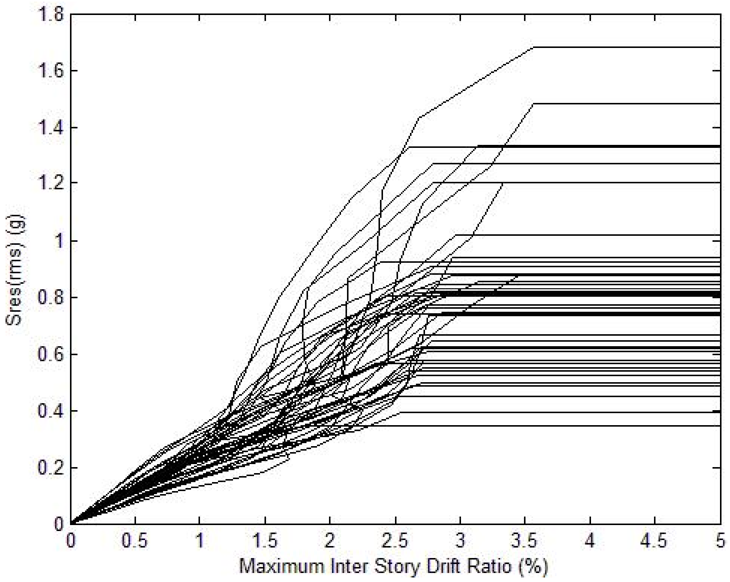

Figure 7. IDA Curves of frame under far field earthquake records

Seismic fragility is a conditional probability of collapse versus various earthquake intensities. In other words, fragility curve is a function of seismic fragility versus earthquake intensities that can evaluate structural behavior in different performance level. Figure 8 shows fragility curves of studied frame due to near field and far field earthquake records.

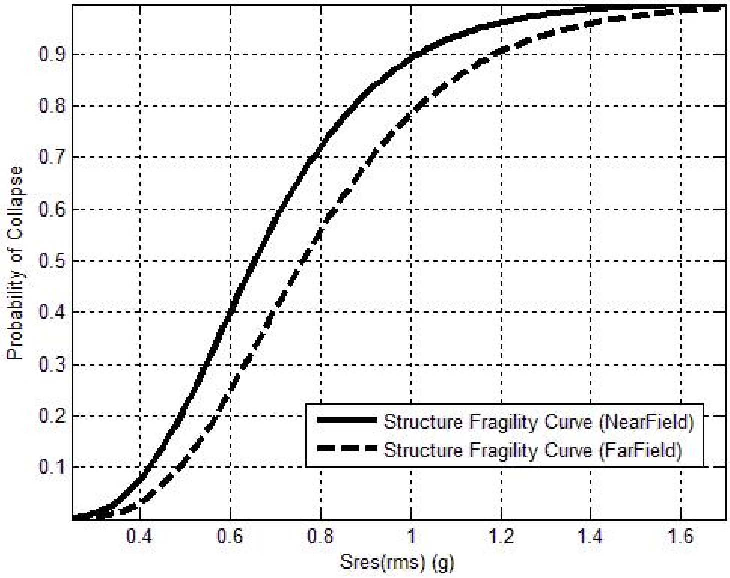

Figure 8. Fragility Curves of frame under Near Field and Far Field earthquake records

FEMA P695 recommends collapse margin ratio (CMR) parameter that can evaluate structural condition against earthquake records collection. Eq.(5) shows the way to calculate CMR.

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}} \tag{5}$$

In Eq.(5), $\hat{S}_{CT}$ is a median collapse intensity that can calculate from fragility curve and S_{MT} is a earthquake intensity at MCE level that calculate according to design response spectrum and fandamental period of the structure. İn this study, modified intensity measure was used for IDA analysis, so in order to calculate S_{MT}, the same strategy as calculation of modified intensity measure was used.

Table 3. Determination of Collapse Margin Ratio

Analysis Case	$\hat{S}_{CT}$	S_{MT}	CMR
Near Field Records	0.654	0.353	1.853
Far Field Record	0.761	0.353	2.156

CONCLUSIONS

According to push over curves of the structure can be found that overal behaviour of studied frame doesn’t have suitable ductility duo to weak shear strength of columns. Push over curves show columns shear failure of critical story have occurred before the base shear reaches to maximum base shear capacity. Lateral stiffness and strength decreases rapidly due to this failure mode. In other word, before reaching to flexural displacement capacity, shear capacity decreased due to shear failure of columns.

By comparing IDA curves of structure can be deduced that near field earthquake records lead to failure and collapse of the structure in lower intensity level in comparison with far field earthquake records. This can be justified by pulsed nature of near field earthquake records. Existence of strong pulses with high amplitude in near field earthquake records (particularly in forward directivity records) leads to brittle behavior of structure exposed to these earthquake records. In other hand due to brittle nature of structure, shear and axial failure of columns occurred at lower intensities compared with far field earthquakes.

Comparison of fragility curves of structure clearly shows that studied frame had lower capacity exposed to near field records compared with far field records. In other words, in the same intensities of near field and far field earthquake records, probability of collapse due to near field earthquakes higher than far field earthquakes. Also, according to CMR results of structure under near field and far field earthquake records can be realized that probability of collapse in near field earthquake is higher than far field earthquakes.

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