



ASSESSMENT OF SEISMIC RELIABILITY OF RC FRAMED BUILDINGS USING A VECTOR-VALUED INTENSITY MEASURE

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ABSTRACT

This study evaluates seismic reliability of RC framed buildings with two different lateral load resisting systems. Four example buildings with different heights and different lateral load resisting systems are analyzed through incremental dynamic analysis under two sets of ordinary and near-fault records. A vector-valued intensity measure is used for reliability evaluation. The results of case studies show that in near-fault ground motions there is a slight difference between seismic reliability of MRF and dual systems, particularly for high-rise buildings. In contrast, adding shear walls improves seismic reliability of both of low-rise and high-rise buildings in ordinary ground motions.

Keywords: Seismic reliability; vector-valued intensity measure; RC frames; incremental dynamic analysis; near-fault ground motions

1. INTRODUCTION

Reinforced concrete framed buildings are a prevalent form of structures in seismically active zones. Moment resisting frame (MRF) system and dual system (consisting of MRF and RC shear walls) are mainly used as earthquake resistant systems in RC framed buildings. Despite of the worldwide popularity of RC frames in construction, some past earthquake events have caused extensive damage and collapse of RC buildings which resulted in enormous monetary and catastrophic life losses (e.g. 1994 Northridge earthquake in USA, 1999 Kocaeli earthquake in Turkey, 2003 Bam earthquake in Iran). Many researchers have evaluated the seismic performance of existing RC frames through various approaches (e.g. [1-8]).

However it should be noted that there are significant uncertainties in structural capacity and seismic excitations, therefore a probabilistic approach seems to be more appropriate for assessment of seismic performance of a structure. In this approach the reliability analysis can be implemented to evaluate the performance of a structural system. Reliability analysis

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has been applied in numerous researches which deal with structural engineering problems (e. g. assessment of redundant trusses [9], fatigue in structures under random loadings [10], wind-excited structures [11], reliability-based optimization [12], seismic performance of steel framed buildings [13] and specifically seismic performance of RC frames [14, 15]).

Dymiotis et al. [14] applied reliability analysis for investigation of uncertainties in structural response and capacity of RC frames. They proposed a method based on probabilistic approach for evaluation of reliability of RC frames subjected to earthquake excitations. In their study data from shaking table tests of small-scale frames were used for statistical description of the critical structural response. They applied their methodology for presenting vulnerability curves of a case study RC framed building.

Soares et al. [15] have presented a formulation for reliability estimation of RC buildings considering both physical and geometrical non-linearity. Their method represents the mechanical behavior of RC structures at the failure stage, which is governed by possible large displacement effects, tension stiffening effects and concrete softening behavior. In their work the reliability model was based on adaptive failure surfaces representing the mechanical model responses. They verified that the safety factors for RC buildings provided by international design codes are adequate, but in some cases would be conservative.

However, the Pacific Earthquake Engineering Research Center (PEER) has developed a methodology for performance-based earthquake engineering, in which the probabilistic seismic demand analysis is applied to estimate the structural reliability [16]. In this method ground motion hazard is coupled by the structural response by means of a parameter named intensity measure (*IM*). The structural response parameter is termed Engineering Demand Parameter (*EDP*). Besides this *IM*-based method, other methods have been proposed for coupling seismic hazard and structural response (e. g. [17, 18]). Estimation accuracy and computational costs (sufficiency and efficiency [19]) are two important issues about *IM*-based method. Vector-valued intensity measures have been proposed to resolve these issues [16], but a main challenge still remains in calculating probabilistic seismic hazard (PSHA), especially for near-fault records. This issue can be treated by modifying vector probabilistic seismic hazard analysis (VPSHA) [20].

The principal aim of this paper is to evaluate the seismic reliability of RC framed buildings with different types of lateral load resisting systems and different heights. This goal is achieved by using a vector-valued intensity measure. In details, four example buildings with MRF and dual systems are designed based upon present building codes and then the structural seismic response is simulated by nonlinear time history analysis of three-dimensional models. These analyses are carried out through Incremental Dynamic Analysis (IDA) under two sets of ordinary records and near-fault records. Based on PEER framework, seismic reliability of example buildings is obtained by coupling ground motion hazard with results from dynamic analysis. The results show that in near-fault ground motions there is a slight difference between seismic reliability of MRF and dual systems, particularly in high-rise buildings. In contrast, it is observed that adding shear walls improves seismic reliability of both of low-rise and high-rise buildings in ordinary ground motions.

2. INTENSITY MEASURE AND RECORD SELECTION

An intensity measure (*IM*) is a parameter that measures the intensity of an earthquake ground motion. For example Peak Ground Acceleration (PGA) or spectral acceleration at the fundamental period of a structure ($S_a(T_1)$) are scalar *IM*s. In fact in PEER probabilistic framework, the *IM* is a parameter of a ground motion record that links the ground motion hazard to structural response. Since it is assumed that structural response is dependent on *IM*, a concern emerges about the effects of other properties of the ground motion on structure. This issue is termed *sufficiency* by Luco and Cornell [19]. In other words, if the estimated *EDP* distribution not only depends on a single *IM*, but also on the other properties of selected records, the results would not be reliable and this *IM* is called insufficient. This shortcoming of *IM* can be resolved by increasing the number of *IM* components and so, the properties of the ground motions can be described more sufficiently. This *IM* which consists of more than one parameter is called a vector-valued intensity measure which is denoted as ***IM*** in this paper.

Efficiency is another issue about the properties of an intensity measure. An *IM* is more efficient when it can reduce the number of nonlinear analyses, without loss of estimation accuracy [19]. If by using an ***IM***, one can more completely explain the effects of ground motions properties on a structure, the variability in *EDP* distribution will be declined, therefore fewer nonlinear dynamic analyses are needed. It is the advantage of an efficient ***IM*** that results in reducing computational costs.

In this section, first an efficient and sufficient ***IM*** is considered for ordinary ground motions. Then it is shown that this ***IM*** is also sufficient and efficient for near-fault ground motions which may have a velocity pulse. It is also discussed how the records for dynamic analyses are selected with respect to this ***IM***.

2.1 Ordinary ground motions

The *PGA* of a record has been used as a scalar *IM* in the past. More recently, spectral acceleration at T_1 ($S_a(T_1)$) is used as a scalar *IM* and has been found to be more effective than *PGA*, as it is a period specific *IM* and somehow is related to structural properties [21]. But this scalar *IM* is not efficient for a multi-degree-of-freedom, non-linear structural model [22]. As mentioned before, a vector-valued intensity measure would provide more information about ground motions than a scalar *IM*. Baker and Cornell [22] have proposed an ***IM*** which consists of two components: $IM_1 = S_a(T_1)$ and $IM_2 = R_{T_1, T_2}$. The first component, IM_1 , is spectral acceleration at first-mode period. The second component, IM_2 , is defined as the ratio of $S_a(T_2)$ to $S_a(T_1)$ ($IM_2 = R_{T_1, T_2} = S_a(T_2)/S_a(T_1)$). They investigated the efficiency and sufficiency of this vector, and proposed the optimal second period (T_2). They searched for T_2 value that resulted in maximum reduction of standard deviation of prediction errors. They found when high levels of nonlinear structural behavior was observed, T_2 was larger than T_1 and R_{T_1, T_2} ratio was dependent on average level of ductility. They also showed that the standard deviation of the annual frequency of exceeding a given *EDP* is reduced, thus by applying this ***IM*** the number of records and analyses for reliability assessment are decreased. In this paper an ***IM*** with $IM_1 = S_a(T_1)$ and $IM_2 = S_a(2T_1)/S_a(T_1)$ is used for reliability assessment. This vector is also efficient and sufficient for near-fault ground motions, as discussed further below.

2.2 Near-fault ground motions

In some near-fault records, a velocity pulse is observed in the perpendicular direction to the fault rupture. This feature is called directivity effect and in this paper these types of ground motions are referred as pulse-like records. It should be noted that the other features of near-fault motions, like fling steps, are not mentioned in this paper. The pulse-like near-fault ground motions are challenging for characterizing structural seismic reliability [23]. In these records the S_a value at moderate to long periods is relatively high and as a consequence, the nonlinear response of structure increases to a level that is not discerned by using S_a as a scalar IM [24, 25].

An IM which consists of $S_a(T_1)$ and a second component as a measure of spectral shape is found to be effective for seismic reliability assessment of a structure which is subjected to pulse-like records. Baker and Cornell [23] found that the IM consisting of $S_a(T_1)$ and R_{T_1, T_2} can take into account the effect of pulse periods on the structural response. They concluded that choosing $T_2 = 2T_1$ would be effective in the cases where moderate to excessive nonlinearity is observed in the structure.

2.3 Record selection

In this study nonlinear dynamic analysis is used for assessment of seismic response of structures. The ground motions are scaled to different levels of intensity and for each level analysis is carried out. Iervolino and Cornell [26] have investigated the effect of record selection and scaling of records on the structural nonlinear response. They hypothesized that scaling of records does not affect the nonlinear response of structures. Then it was investigated when their hypothesis might not be true. They considered ordinary records and some near-fault records with directivity effects. They compared two methods of record selections in various case studies. In first method the records were carefully chosen to correspond to a definite magnitude and distance scenario. In the second method, the records were selected randomly from a large catalog. The effect of the degree of scaling by first-mode spectral acceleration level was also investigated. They found no significant reason for careful selection of the records based on magnitude and distance. Moreover, the concern over record scaling was not justified.

However, Baker [27] has proposed a robust tool for record selection, which is called Conditional Mean Spectrum (CMS). The CMS provides the expected (mean) response spectrum, conditioned on occurrence of a target spectral acceleration value at the period of interest.

Since the present study investigates the comparative performance of example buildings, the ground motions are selected based on Iervolino and Cornell [26] suggestion and these records are applied along weak principal axis of buildings. Based upon these assumptions the seismic reliability of example buildings is obtained conservatively, and comparative assessment of structures is not supposed to be changed significantly.

The records sets are tabulated in Table 1. These records come from the PEER Strong Motion Database and they have been used in a research for a test-bed building in Van Nuys field [16]. All the selected records are from sites with USGS soil type D. The set of near-fault ground motions consists of records that some of them do not have a velocity pulse, because it is observed that not all pulse-like ground motions are equally severe with respect to a given structure [23].

The maximum distance to rupture surface for selected near-fault records is less than 10 km.

Table 1: Ground motions database

Earthquake Site, Station, Date	Magnitude	R (km)	PGA (g)
<i>Ordinary ground motions (OG)</i>			
Imperial Valley, Compuertas, 10/15/79	6.5	32.6	0.186
Imperial Valley, El Centro Array#13, 10/15/79	6.5	21.9	0.139
Landers, Joshua Tree, 06/28/92	7.3	11.6	0.274
Landers, Coolwater, 7/23/92	7.3	21.2	0.417
Loma Prieta, Agnews State Hospital, 10/18/89	6.9	28.2	0.172
Loma Prieta, Anderson Dam (Downstream), 10/18/89	6.9	21.4	0.244
Cape Mendocino, Fortuna - Fortuna Blvd, 04/25/92	7.1	23.6	0.116
Northridge, LA - Pico & Sentous, 01/17/94	6.7	29.0	0.186
Northridge, Sun Valley - Roscoe Blvd, 01/17/94	6.7	12.3	0.443
San Fernando, Palmdale Fire Station, 02/09/71	6.6	25.4	0.151
<i>Near-source ground motions (NG)</i>			
Imperial Valley, El Centro Array#9, 5/19/40	7.0	8.3	0.313
Imperial Valley, Bonds Corner, 10/15/79	6.5	2.5	0.775
Imperial Valley, Holtville Post Office, 10/15/79	6.5	7.5	0.253
Imperial Valley, Aeropuerto Mexicali, 10/15/79	6.5	8.5	0.327
Coalinga, Pleasant Valley P.P. – yard, 05/02/83	6.4	8.5	0.592
Northridge, Sepulveda VA, 1/17/94	6.7	8.9	0.939
Northridge, Nordhoff Fire Sta, 01/17/94	6.7	9.2	0.552
Coyote Lake, Gilroy Array #2, 08/06/79	5.7	7.5	0.339
Coyote Lake, Gilroy Array #3, 08/06/79	5.7	6.0	0.272
Coyote Lake, Gilroy Array #4, 08/06/79	5.7	4.5	0.387

3. NONLINEAR DYNAMIC ANALYSIS

3.1 Configuration of example buildings

The case studies are 6 and 20-storey residential buildings with two different types of earthquake resisting systems (MRF and dual systems). The storey height is 3 m in all buildings. All of four example buildings have the same floor plan as shown in Figure 1. The plan of example buildings is adopted from one of geometric configurations in a study that was related to evaluation of seismic performance of RC buildings [28]. These buildings are designed according to ACI 318-05 code [29]. These buildings are located in a site in Van Nuys, California, with USGS soil type D. The beams and columns have rectangular sections. The floors are 15cm thick slabs modeled as rigid diaphragms. Concrete with $f'_c=40$ MPa is used for all concrete sections and the nominal yield strength of steel bars is 285

MPa. It is assumed that strength of materials have lognormal distribution with the mean of nominal values. The member sizes and fundamental period (T_1) of designed buildings are summarized in Table 2.

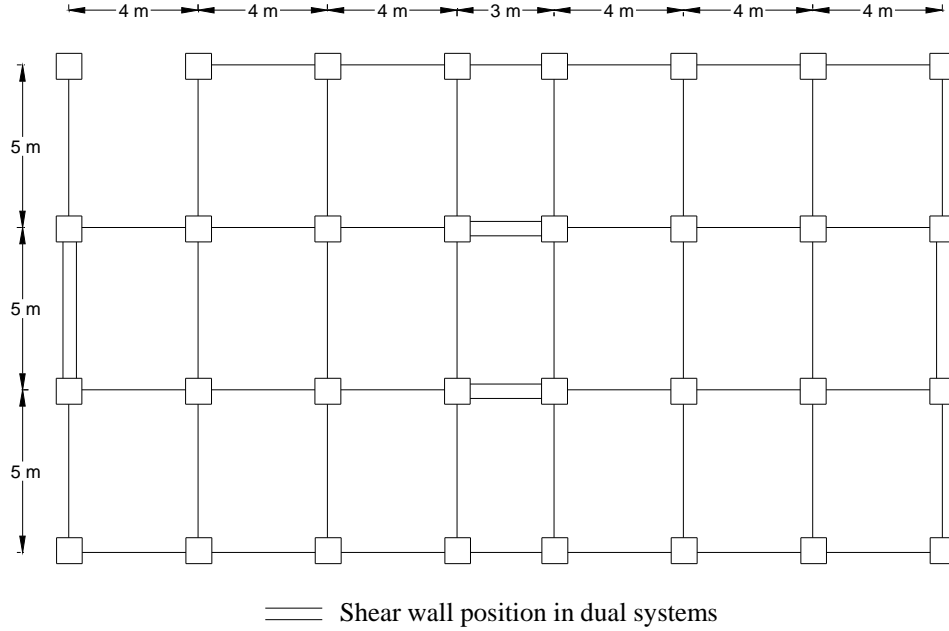


Figure 1. Plan of the example buildings

Table 2: Member sizes of the example buildings

Building	Story	Column section size, cm	Beam section size, cm	Shear wall thickness, cm
6-story MRF ($T_1 = 0.88$ s)	5-6	50x50 (1% reinforcement)	50x50	-
	1-4	50x50 (2.55% reinforcement)	50x50	-
6-story Dual ($T_1 = 0.57$ s)	3-6	50x50 (1% reinforcement)	40x40	35
	1-2	50x50 (2.55% reinforcement)	40x40	35
20-story MRF ($T_1 = 2.12$ s)	17-20	60x60 (1% reinforcement)	50x60	-
	13-16	60x60 (2.15% reinforcement)	50x60	-
	5-12	80x80 (2.2% reinforcement)	50x60	-
	3-4	100x100 (2.8% reinforcement)	50x60	-
	1-2	110x110 (3.2% reinforcement)	50x60	-
20-story Dual ($T_1 = 1.66$ s)	13-20	60x60 (1% reinforcement)	50x60	35
	5-12	80x80 (1% reinforcement)	50x60	35
	3-4	100x100 (1.5% reinforcement)	50x60	35
	1-2	110x110 (2.8% reinforcement)	50x60	35

3.2 Nonlinear structural modeling

This study deals with nonlinear dynamic analysis of structural systems, so reliable nonlinear

models should be adopted. SAP2000 (version 12) nonlinear program [30] is applied for nonlinear response simulation. Inel and Ozmen [31] have studied the effects of plastic hinge properties on nonlinear analysis of RC buildings. Based on their observations, default-hinge properties provided in SAP2000 are appropriate for nonlinear dynamic analysis of buildings which are designed based upon recent codes. Since our example buildings are designed according to recent codes, the nonlinear behavior of beams and columns are replicated by SAP2000 plastic hinges models with bilinear stress-strain relationships (FEMA 356 [32], Tables 6-7 and 6-8).

Nonlinear models for shear walls need more attention, because walls have in plane and out of plane behaviors. For all shear walls in example buildings nonlinear behavior is assumed in flexure, and shear deformation is thought to be elastic. Layered nonlinear shell, provided in SAP2000, is used for modeling the nonlinear behavior of walls, as it allows us to define different layers for concrete and reinforcing, and the behavior in different component directions. The in plane behavior is nonlinear for flexure in vertical direction and out of plane behavior is kept linear. Three material types are used in modeling shear walls: steel material for longitudinal and transversal steel bars, unconfined concrete for wall sections, and confined concrete for boundary elements. The stress-strain curves of these materials are shown in Figure 2.

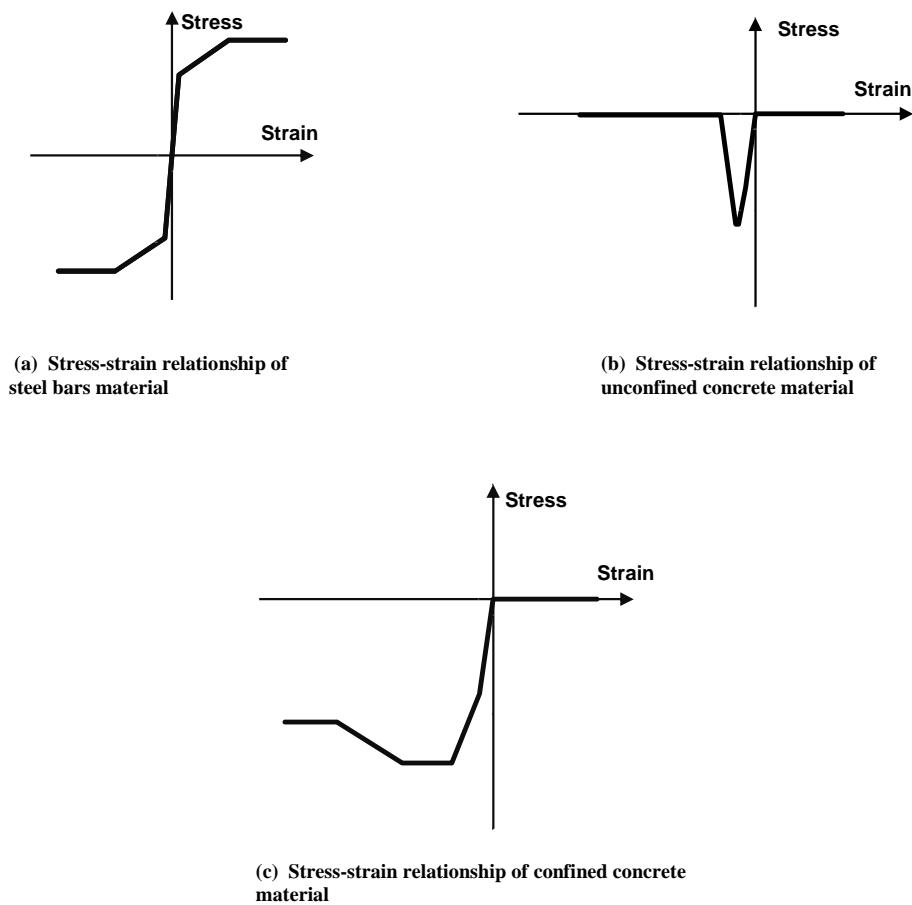


Figure 2. Stress-strain curves of materials used for shear walls modeling

3.3 P-Delta analysis

The lateral movement of a story mass to a deformed position results in second order overturning moments. In structural analysis this effect is generally called P-Delta effect. Wilson [33] has developed an algorithm that through considering geometric stiffness, excludes P-Delta effects into the basic formulation of the stiffness matrix.

In considering the P-Delta effects in SAP2000 analysis, at first the gravity loads are applied in a nonlinear load case using a ramp function that ramps-up over a length of time that is long compared to the first period of the structure (for example ten times), and then held constant for an equal length of time. Modal damping is set to a high value, for example 0.99. Following that, the nonlinear time-history analysis for an earthquake record is carried out. In other words results from a previous nonlinear analysis are initial condition for dynamic nonlinear analysis.

3.4 Incremental dynamic analysis

Incremental dynamic analysis (IDA) is a method for evaluation of demand and capacity of structures under seismic loading. This method needs nonlinear time-history analysis of a structure subjected to a set of ground motions which are scaled to several intensity levels [34]. Performing an IDA requires several important steps. After creating an appropriate model for the structure and selecting a set of records, each record is incrementally scaled to intensity levels. Then, for each intensity level nonlinear time-history analysis is carried out. Incrementing is stopped when the numerical non-convergence is encountered, or when the straight line slope between consecutive points in IDA curve is less than 20% of elastic slope. In the following the elastic slope is defined. Here each record is scaled to various levels of $S_a(T_1)$ and maximum interstory drift ratio is considered as structural state variable or Damage Measure (DM). Maximum interstory drift angle, observed in any storey of building, is an appropriate DM for framed buildings as it is concerning the joint rotations and both global and local storey collapse [35].

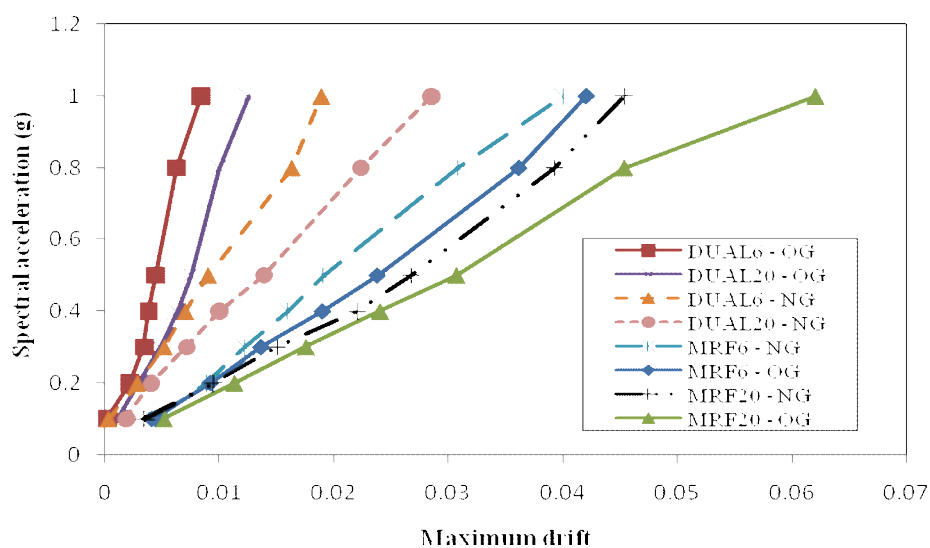


Figure 3. IDA curves of example buildings

IDA is carried out for each example building and resulting median IDA curves are illustrated in Figure 3. The resulting structural response from IDA can be integrated with the results of Probabilistic Seismic Hazard Analysis (PSHA) to calculate the seismic reliability; however the IDA data can help better understanding of the structural behavior.

Figure 3 verifies that all curves demonstrate an elastic linear segment which ends at $S_a(T_l) \approx 0.2g$. The slope of this segment is called elastic stiffness for each curve. This behavior is commonly observed in structures which have initial linear elastic elements [34]. MRF systems have larger drift demands in comparison with dual systems; it confirms that shear walls reduce the drift demands of structures. For all buildings, the IDA curves express the same relationship between structural response parameter and earthquake intensity measure, it can be seen that there is a correlation between the two, that the lower the spectral acceleration, the smaller the drift demand.

4. SEISMIC RELIABILITY

Based on work of PEER center, the structural seismic response is called Engineering Demand Parameter (EDP) [16]. The maximum interstory drift ratio is chosen as *EDP*. The limit-states proposed by FEMA 356 [32] (which are in term of drift angel) are used to define performance levels for buildings. The limit-states are Immediate Occupancy (IO), Life Safety (LS) and Collapse Preventions (CP). The drift limits of 1%, 2% and 4% are used for defining the limit-states of IO, LS and CP for RC moment frames and in the same way limit-states of 0.5%, 1% and 2% are used for RC frames with shear walls.

The Mean Annual Frequency (MAF) of exceeding a given level of *EDP* (drift hazard), is given by following equation:

$$\lambda_{EDP}(z) = \int_{im_1} \int_{im_2} P(EDP > z | IM_1 = im_1, IM_2 = im_2) \left| \frac{\partial \lambda_{IM}(im_1, im_2)}{\partial im_1 \partial im_2} \right| dim_1 dim_2 \quad (1)$$

Where λ_{IM} is the joint seismic hazard of \mathbf{IM} , and the term $P(EDP > z | IM_1 = im_1, IM_2 = im_2)$ is the probability distribution of structural response for a given \mathbf{IM} . Numerical integration can be used for calculating drift hazards form Equation (1) (this is the value that should be estimated efficiently). For probabilistic assessment of a structure from Equation (1), the joint probabilities of IM_1 and IM_2 values (λ_{IM}) at a site should be estimated. In other words, for a given site it should be known how often ground motions with given \mathbf{IM} will happen. Bazzurro and Cornell [20] have proposed Vector-valued Seismic Hazard Analysis (VPSHA) for sites subjected to ordinary ground motions. They characterize the distribution of $\ln R_{T1, T2}$ and $\ln S_a(T_l)$ for an earthquake event as a joint normal random variable, where the conditional dependence between these two variables is taken into account using a correlation coefficient. This method can be modified for sites subjected to near-fault ground motions. By revising the means, variances and correlation coefficients of the ground motion predictions, the possible occurrence of pulse-like motions are taken into account. This adjusting can be applied in existing attenuation models for pulse-like ground motions. In

this study the prediction model for pulse-like ground motions suggested by Bozorgnia and Betero [36] is adopted.

Each sets of records are scaled to different levels of IM_1 , since for each level both of $S_a(T_1)$ and $S_a(2T_2)$ are changed by same scale factor, the $IM_2 = S_a(2T_1)/S_a(T_1)$ remains unchanged by scaling. Therefore, for prediction the response as a function of both IM components, a supplement to scaling is needed. A linear least-square regression model [37] can be used to incorporate information from IM_2 . In fact, for estimation of probability distribution of structural response, we should first separate out the collapse responses, and then deal with remaining no-collapse responses. For estimation of the probability of collapse, $P(C)$, logistic regression [37] is used for estimation of collapse probability:

$$P(C | IM_1 = im_1, IM_2 = im_2) = \frac{e^{\beta_0 + \beta_1 im_2}}{1 + e^{\beta_0 + \beta_1 im_2}} \quad (2)$$

In which β_1 and β_1 are coefficients obtained by regression analysis and are different for different values of IM_1 . C indicates collapses (C is equal to 1 for collapse cases and 0 otherwise).

Now, IM_2 is incorporated in prediction of no-collapse. For any record scaled to IM_1 , there is one IM_2 value and one EDP value, so $\ln EDP$ can be estimated through linear least-square regression [37]. From regression, for no-collapse the mean value of $\ln EDP$ is:

$$E[\ln EDP | IM_1 = im_1, IM_2 = im_2] = \beta_2 + \beta_3 \ln im_2 \quad (3)$$

Where β_2 and β_3 are obtained by regression analysis. $\ln EDP$ has a normal distribution [16], so the probability for no collapse is given by:

$$P(EDP > z | IM_1 = im_1, IM_2 = im_2, no\ collapse) = 1 - \phi\left(\frac{\ln z - (\beta_2 + \beta_3 \ln im_2)}{\sigma_e}\right) \quad (4)$$

Where $\phi(\cdot)$ is standard normal distribution and σ_e is logarithmic standard deviation determined from the regression analysis.

Using total probability theorem [38] and combining the two possibilities of collapse or no collapse, we can obtain the conditional probability that EDP exceeds z as follows:

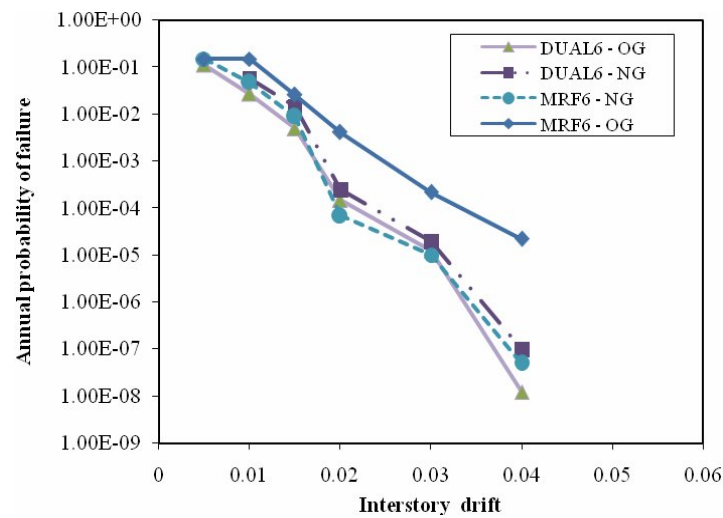
$$P(EDP > z | IM_1 = im_1, IM_2 = im_2) = P(C) + [1 - P(C)][1 - \phi\left(\frac{\ln z - (\beta_2 + \beta_3 \ln im_2)}{\sigma_e}\right)] \quad (5)$$

Where $P(C)$ is given by Eq (2).

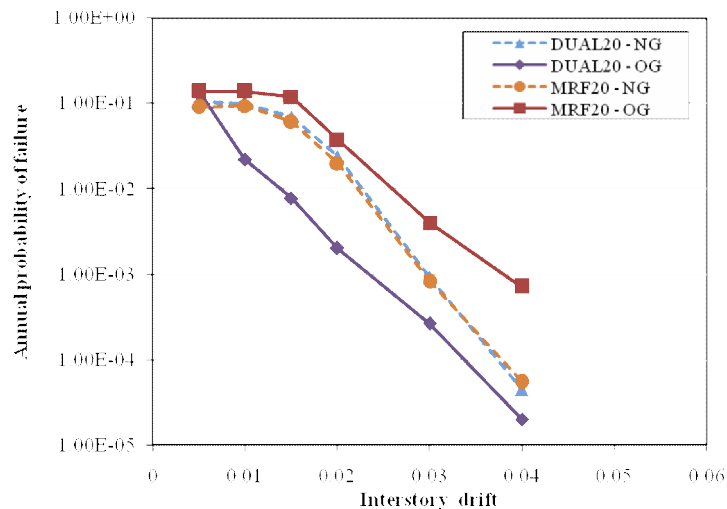
Now we can calculate the MAF of exceeding a certain limit-state capacity using Equation (1). The MAF of exceeding drift limits are obtained for example buildings and are summarized in Table 3 and are also shown in Figure 4 in the form of performance curves.

Table 3: Limit states probabilities

		IO	LS	CP
Ordinary ground motions	MRF6	1.40×10^{-01}	4.19×10^{-03}	2.07×10^{-05}
	Dual6	1.21×10^{-01}	2.73×10^{-02}	1.50×10^{-04}
	MRF20	1.48×10^{-01}	4.01×10^{-02}	6.97×10^{-04}
	Dual20	1.22×10^{-01}	2.22×10^{-02}	1.98×10^{-03}
Near-source ground motions	MRF6	4.65×10^{-02}	7.04×10^{-05}	4.98×10^{-08}
	Dual6	1.52×10^{-01}	5.73×10^{-02}	2.50×10^{-04}
	MRF20	9.00×10^{-02}	2.00×10^{-02}	5.50×10^{-05}
	Dual20	1.34×10^{-01}	9.90×10^{-02}	4.50×10^{-05}



(a) Drift hazards of 6-storey buildings



(b) Drift hazards of 20-storey buildings

Figure 4. Performance curves of example buildings

5. RESULTS AND DISCUSSIONS

Based upon IDA curves (Figure 3) and MAF results (Figures 4a and 4b), the comparative performance of structures can be described. In Figure 3, the trend between median response and $S_a(T_1)$ are compared for example buildings subjected to ordinary and near-fault records. The IDA curves imply that the MRF systems behavior is different from dual systems, and moreover, the behavior of a specific structural system is dependent on type of ground motions. For example, the drift demands of 6-storey building with MRF system subjected to near-fault records are lesser than drift demands when they are subjected to ordinary records.

In contrast, the dual systems have larger drift demands in near-fault ground motions comparing to ordinary ground motions. This difference is clear in Figure 3 for 6- and 20-storey buildings with dual systems.

These results can be justified by pulse-like characteristic of near-fault records, as these types of records tend to impose energy to structure in a short time interval. Since the mechanism of energy dissipating is different in MRF and dual systems, so their behavior is totally different in near-fault ground motions. In MRF systems the inelastic behavior appears in plastic hinges that form in beams and columns, but in shear walls an inelastic region should form at the lower part of the walls, so it makes it difficult for shear walls to dissipate energy in a short while, and as a consequence the drift demands increase for dual systems in near-fault ground motions.

These observations can help better understanding the results obtained from Equation (1). Figure 4a compares the MAF of exceeding a given EDP level for 6-storey buildings in ordinary and near-fault ground motions. As it was stated in previous section, the performance levels (IO, LS and CP) of MRF systems and dual systems were defined in term of drift limits, so the performance of 6-storey buildings with different earthquake resistant systems can be compared in different limit-states. All buildings have the same probability of failure at small drifts, but as the drift increases the difference becomes clear.

The 6- storey building with MRF system has lesser reliability in ordinary ground motions in comparison with near-fault ground motions. These results are conforming to the results of drift demands obtained from IDA curves. However the limit-state probabilities of 6-storey buildings with dual systems have a small difference in both types of ground motions, but the dual systems are more reliable in ordinary ground motions. The most significant difference is observed between limit-state probabilities of MRF and dual systems in ordinary ground motions (Figure 4b). Since the shear walls decrease the drift demands, so they have a considerable effect on increasing seismic reliability.

In Figure 4b the limit-state probabilities of 20-storey buildings are compared for near-fault and ordinary records. There is a noticeable difference between reliability of 20-storey buildings with MRF and dual systems in ordinary ground motions, so it can be inferred that adding shear walls will improve the seismic reliability of 20-storey buildings in ordinary records. On the other hand a slight difference is observed between performance curves of 20-storey buildings in near-fault ground motions. This implies that dual systems are not more beneficial than MRF systems for 20-storey buildings in near-fault ground motions.

6. SUMMARY AND CONCLUSIONS

The comparative performance of RC lateral load resisting systems is assessed through seismic reliability analysis. PEER probabilistic frame work is applied for obtaining the drift hazards of buildings. A vector-valued intensity measure consisting $S_a(T_1)$ and $S_a(2T_1)/S_a(T_1)$ is used for reliability analysis evaluation. The structural response is simulated by incremental dynamic analysis with two sets of ordinary ground motions and near-fault ground motions. Example buildings are 6-storey and 20-storey buildings with MRF and dual systems. The results of structural analysis are coupled with results of probabilistic seismic hazard analysis and mean annual frequency of exceeding given performance levels is obtained. The performance levels are Immediate Occupancy, Life Safety and Collapse Prevention. The limit-states are adopted from FEMA 356 for MRF and dual systems.

The results of IDA data and drift hazards of case studies help in comparing the two structural systems with different heights in ordinary and near-fault ground motions. It is observed that the seismic performance of buildings is dependent on adopted lateral load resisting system, building's height and type of ground motions. The added shear walls are effective for both 6-storey and 20-storey buildings in ordinary ground motions, as they enhance the seismic reliability. In contrast, in near-fault ground motions there is a slight difference between seismic reliability of dual systems and MRF systems. This difference becomes less as the height of buildings increases, so it can be said that shear walls are not much advantageous than MRF systems in this class of ground motions. It should be noted that these results cannot be generalized for all RC buildings and large enough case studies and more ground motions should be considered, but the procedure used in this paper, can be implemented for other structures.

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