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A response spectra based intensity measure for seismic system reliability

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ABSTRACT: This work studies a newly proposed seismic intensity measure (IM) and evaluates its efficiency and sufficiency with respect to ground motion records and structural response, with the end goal of computing structural system reliability under seismic loads for design application. An intensity measure's efficiency is the variance of engineering demand parameters (EDPs) across different ground motions at a particular value of IM. Sufficiency corresponds to the statistical independence of EDP at a given IM from ground motion parameters such as magnitude, site to rupture distance and soil type. The proposed intensity measure $(I_{RS})$is based on spectral acceleration at the natural time period $Sa(T_1)$ and the geometric mean of spectral acceleration over a fixed range of periods RS_{avg}. Unlike previously proposed IM's, RS_{avg} is measured over a predetermined range of time periods (independent of natural time period) to capture higher mode effects and strength degradation. Proposed IM is verified with a single degree of freedom (DOF) lumped mass system, 5 DOF lumped mass system, archetype steel frames, and single storey reinforced concrete (RC) shear wall building by statistical studies on non-linear time history analysis results. Twenty-two far field ground motion acceleration records (forty-four individual directional components) specified by the Federal Emergency Management Agency's P695 document are used. It is found that I_{RS} is highly efficient and sufficient for structures with time period more than $1sec$ and moderately efficient for shorter period structures compared to ten other intensity measures from the literature.

1. INTRODUCTION

Earthquake resistant structural design has advanced significantly, from designing for lateral loads in the range of 6-10% of structural weight up to the 1950's (Priestley et al. (2007)) to designing for target performance levels using incremental nonlinear time history analyses in the 2000's (FEMA-P695 (2009)). Since earthquakes are rare events having low occurrence probability, and since elastic force demands from ground motion accelerations result in orders of magnitude larger sections compared to dead, live or wind loads, it is economically acceptable to perform seismic design for a predefined damage level without significant strength degradation (Priestley et al. (2007)). This permits designing for reduced seismic forces by energy dissipation through plastic hinge formation or non structural damage.

In simplified equivalent lateral force, response spectrum or time history analysis based design currently permitted by building codes (e.g. IS1893 (2002)), seismic forces obtained from an elastic analyses are scaled using an appropriate response reduction factor that depends on a designated energy dissipative element's ductility. System damage for the corresponding earthquake level is estimated from inelastic displacements obtained by amplifying the elastic displacement with appropriate displacement amplification factors (SEAOC (2008)). The physical quantity predominantly used in component limit states, for all loads including earthquakes, is force. Although force based design is most popular and simplifies the design procedure, it has demerits such as distribution of seismic forces based on initial stiffness, in-effect assuming that various structural elements yield simultaneously, and a single and unique force reduction factor for a given material and structural type (Priestley et al. (2007)), without considering the entire system, except through empirical adjustments made during subsequent component-wise designs. As a result, the performance/safety level achieved using force based design is usually unknown and non-uniform across different structural systems.

Performance based seismic design evaluates the exceedance probability of limit state LS (for example, interstorey drift ratio) using Equation 1 by performing nonlinear dynamic time-history analysis:

$$\lambda[LS] = \int\int_{DM,IM} G[LS|DM]dG[DM|IM]||d\lambda[IM]| \quad (1)$$

where DM represents a structural damage measure such as roof displacement, $G[LS|DM]$ is the cumulative distribution function of LS given DM and and $dG[DM|IM]$ is the probability density function of DM given IM. $\lambda[LS]$ is the annual exceedance probability of a limit state function and $\lambda[IM]$ is mean annual frequency of exceeding each value of IM (ground motion hazard at a particular site). $d\lambda[IM]$ represents a differential with respect to IM.

The intensity measure used to calculate $\lambda[LS]$ in Equation 1 should have specific characteristics known as efficiency and sufficiency for unbiased probabilistic estimation with regard to randomness of ground motion records. An IM's efficiency is the ability to predict mean structural response (DM) at a given level with minimal dispersion across ground motion records. Sufficiency evaluates the independence of IM from ground motion parameters such as earthquake magnitude (M) and site to rupture plane distance (R). This enables Equation 1 to be generalized due to conditional independence of IM with respect to M and R. FEMA P-695 (FEMA-P695 (2009)) recommends 22 pairs of far-field ground motion records for measuring an IM's efficiency and sufficiency.

Since performance based seismic design requires multiple non-linear time history analysis at varying ground motion levels, monitoring the structural response of various elements in the system can potentially provide a direct design pathway without relying on empirical code based modifications for overstrength and displacement amplification. While the end goal of the author's ongoing study is to estimate seismic system reliability of reinforced concrete shear wall buildings, and subsequently design for specified system reliability targets, the study presented here documents the efficiency and sufficiency of a new intensity measure proposed for that purpose.

2. Proposed intensity measure for seismic system reliability

2.1. *A review of existing intensity measures*

Preliminary studies on intensity measures attempted to identify ground motion record parameters that cause damage to structural systems. One such parameter is acceleration pulse duration, which is the time between successive zero crossings of a ground motion acceleration history. If the duration of these acceleration pulses is more than fundamental natural period T_1 then structures are more likely to experience severe damage, irrespective of peak ground acceleration or velocity (Anderson and Bertero (1987)). Relative suitability of intensity measures were quantified using two proposed statistical properties – reduced record-to-record dispersion (efficiency) and independence from magnitude and distance (sufficiency) (Luco and Cornell (2007)).

The most popular intensity measure in current practice is spectral acceleration at fundamental time period $(Sa(T_1))$ (Shome et al. (1998)), defined as the maximum acceleration of a linear elastic single degree of freedom (SDOF) system subjected to ground motion record, typically with 5% viscous damping. Even though it has a direct application in design parameters such as design base shear, it does not include the effect of inelasticity and also excludes the contribution from time periods other than the fundamental period (Alavi and Krawinkler (2001); Mehanny (2000)).

Shome (1999) proposed a vector valued intensity measure to predict mean structural response by including spectral acceleration (Sa) at second mode, but questions regarding the inelasticity and higher modes contributions remained unaddressed. Unlike the Sa at second period, Cordova et al. (Cordova et al. (2000)) proposed a scalar IM by combining $Sa(T_1)$ with Sa at a period twice of T_1 to include the effect of inelasticity. Luco and Cornell (2007) proposed structure specific IM's based on elastic and inelastic displacements by involving the contribution from first and second modes.

To overcome the difficulties of implementing vector-valued IM in practical applications and the sensitivity of inelastic systems to multiple time periods, the geometric mean of elastic spectral ordinates(Sa_{avg}) was proposed by several authors, some of which are listed in Table 1. Since the effectiveness of Sa_{avg} depends on the time period range and time period intervals, distinct period ranges (proportional to T_1) have been suggested by several authors (Table 1) to account for higher mode contribution and elongated periods due to structural softening.

2.2. *Proposed intensity measure*

Most IM's designed to overcome the drawbacks of $Sa(T_1)$ are based on a geometric mean of spectral acceleration over a period range that is a function of fundamental period T_1 (Table 1). A new intensity measure (I_{RS}) was proposed (Mohan and Chatterjee (2022)) by combining Rs_{avg} and α (Equation 2):

$$I_{RS} = (Rs_{avg})^{\alpha} \tag{2}$$

where Rs_{avg} is the geometric mean of response spectra ordinates measured over a fixed period range between 0.01 to 2.5s. This pre-defined period range was chosen to account for higher mode effects and stiffness degradation during non-linear response. Further, a dimensionless parameter α $(Sa(T_1)/g)$ was included to lay a special emphasis on the fundamental mode's contribution.

Notation	Details
PGA	Peak ground acceleration
$Sa(T_1)$	Spectral acceleration at T_1
IM_3	$Sa(T_1)(Sa(2T_1)/Sa(T_1))^{0.5}$ Cordova et al. (2000)
IM_4	$Sa_{avg}(T_1-2T_1)$ Bianchini et al. (2009)
IM_5	$Sa_{avg}(0.5T_1-1.5T_1)$ Shakib and Pirizadeh (2014)
IM_6	$Sa_{avg}(0.2T_1-1.6T_1)$ Adam et al. (2014)
IM_7	$Sa_{avga}(0.2T_1-3T_1)$ Eads et al. (2015)
IM_8	$Sa(T_1)/Sa_{avg}(0.2T_1-3T_1)$ Eads et al. (2016)
IM_9	$Sa_{avg}(T_1-1.6T_1)$ Tsantaki et al. (2017)
IM_{10}	$Sa(T_1)(Sa_{avg}(T_1-2T_1)/Sa(T_1))^{0.4}$ Bojórquez et al. (2017)
I_{RS}	$(Rs_{avg})^{\alpha}$

Table 1: List of IM's used for comparative study

2.3. *Validation*

In a preliminary study (Mohan and Chatterjee (2022)), three different models were studied for the evaluation of proposed intensity measure (Figure 1). Out of these, SDOF and 5 DOF lumped mass $(T_1 = 1.05s)$ systems are modeled in ABAQUS (Systems (2009)) using two dimensional connector elements (CONN2D2) having translational DOF along the orientation of the element. Damping is incorporated by using Rayleigh damping coefficients corresponding to the prescribed damping ratio.

The archetype steel frame models (4, 8, and 12 storey having T_1 1.62s, 2.29s, *and* 3.12s respectively) were built in OPENSEES (Mazzoni et al. (2006)) using a concentrated plasticity approach at beam-column joints, whose nonlinear behaviour was modelled using a modified Ibarra-Krawinkler bilinear rotational stiffness (Lignos and Krawinkler (2011)). Rayleigh damping coefficients are obtained by using the frequencies corresponding to the fundamental and 10% of fundamental frequency for 2.5% critical damping. All models were analyzed implicit dynamically using Newmark integration method by applying a ground motion acceleration time history at the fixed base.

Statistical properties of the IM's are obtained using far field ground motion records set proposed by FEMA P695 FEMA-P695 (2009). Efficiency is measured by the standard deviation of the structure response at a specific intensity level by assuming that they follow log-normal distribution (Shome et al. (1998)). Sufficiency is measured by using the simple relative sufficiency proposed (Dávalos and Miranda (2019)), where gradient of regression analysis with respect to M and R is compared for various IM's. It concludes that IM having the least slope is more sufficient and gives unbiased results in estimation of probabilistic quantities.

Although SDOF system is defined by using the fundamental period, response due to inelastic behaviour subjected to ground motion may not be measured efficiently using $Sa(T_1)$. Hence a basic parametric study was done to verify the efficiency of IM's with respect to various time periods for a elasto-plastic SDOF system. Figure 2 indicates that efficiency of PGA decreases with increasing time period. Dispersion of IM's based on geometric mean of spectral ordinates ($IM_3 - IM_{10}$) is limited between 20% to 35%. The efficiency of proposed intensity measure I_{RS} decreases with increasing time period due to its emphasis on higher mode effects. It is highly efficient at time periods greater than 1sec with dispersion limited to less than 10%.

Following the parametric study, IM characteristics were investigated for multiple degree of freedom (MDOF) lumped mass and steel framed models. Figure 3 represents the dispersion of different intensity measures for 5 DOF lumped mass, and 4, 8 and 12 storey steel frame models (Mohan and Chatterjee (2022)). Proposed I_{RS} limits dispersion to around 10% for lumped mass and frame models. Simple relative sufficiency study on the pro-

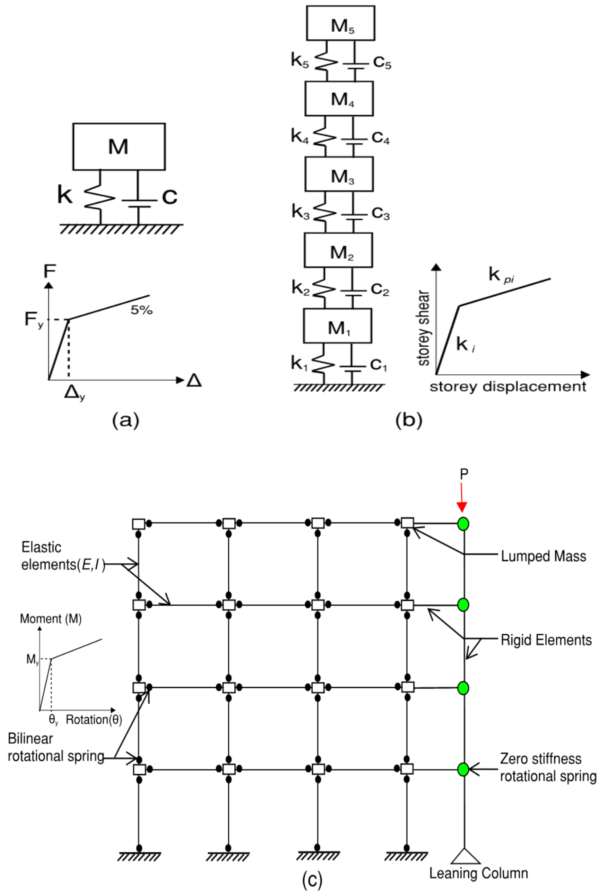

Figure 1: (a) SDOF system (b) 5 DOF lumped mass model (c) 4 storey archetype frame

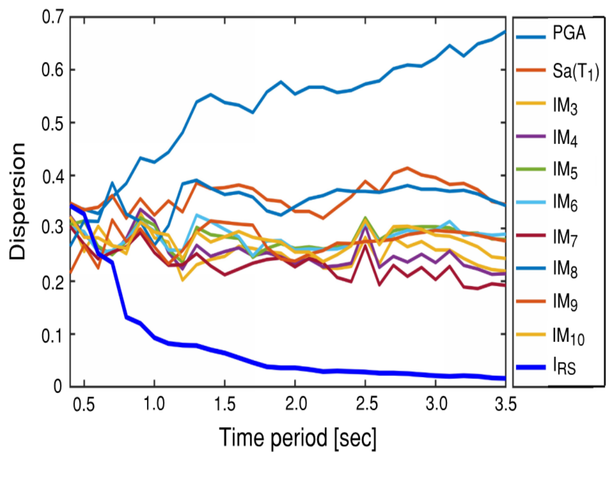

Figure 2: Dispersion of SDOF system with time period

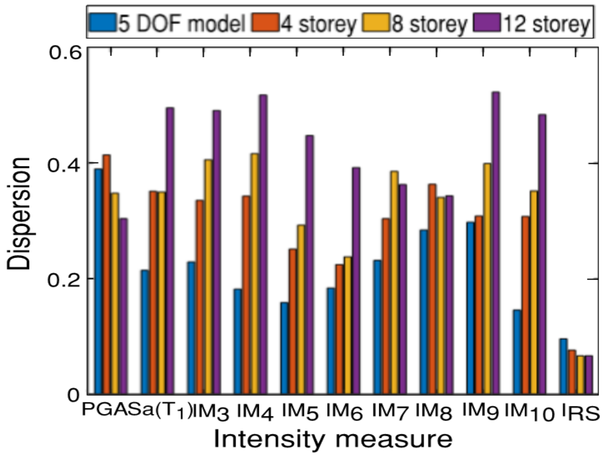

Figure 3: Dispersion of 5 DOF lumped mass and frame models

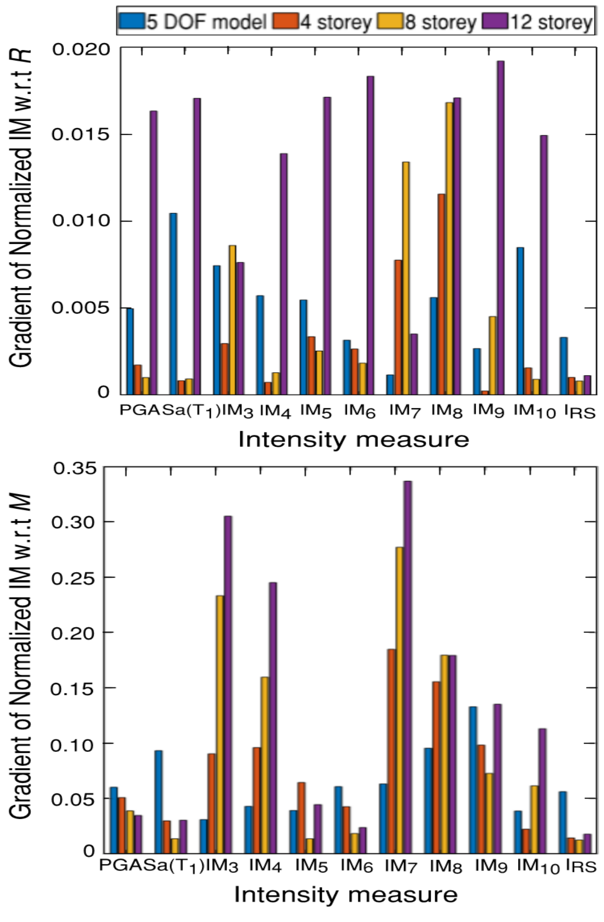

Figure 4: Slope of linear regression analysis with respect to R & M of 5 DOF lumped mass and framed models

posed models is shown in Figure 4. It can be seen that for both lumped and framed models, gradient (slope) from linear regression analysis is small compared to the other IMs with respect to both R & M. The results indicate that the proposed intensity measure I_{RS} can potentially minimize bias with respect to ground motion magnitude and distance during probabilistic performance estimation of steel framed structures, including nonlinear response and higher mode effects. This paper extends statistical studies on I_{RS} to reinforced concrete shear wall buildings, as discussed in the next section.

3. IDEALIZED REINFORCED CONCRETE SHEAR WALL BUILDING

Reinforced concrete (RC) shear wall buildings are designed such that lateral forces are resisted by shear walls whereas beam and column support gravity loads without losing vertical load carrying capacity due to high deformations under seismic forces. In this work, computational model of a single storey RC shear wall building is developed using ABAQUS (Systems (2009)). Floor plan of the model is considered as $10.5m \times 10.5m$ and columns are positioned at an interval of $3.5m$. Shear walls are placed along the outer frame between the centre columns. Beam-column joints are modelled as hinge connections except at shear wall interfaces Figure 5. Shear wall is modelled using two type of finite elements namely C3D8R (continuum 3D solid element having 8 nodes with reduced integration) for concrete and T3D2 (3D truss element having 2 nodes) for steel rebar. Reinforcement is modeled as tie elements embedded in concrete such that the rebar has identical translational DOF as host element i.e., concrete, but with independent rotational DOFs. Two different grade of steels are used, Fe 250 grade for stirrups and Fe 415 grade for longitudinal rebar. Percentage of steel used in beam, column, slabs, and shear wall is 0.5%, 0.5%, 0.35%, and 0.35% respectively.

Beam-column joints are shear force resisting only, that is, their moment capacity is zero or small enough to yield under lateral deformations. Hinge connections are modelled in ABAQUS by constraining translational degrees of freedom between beam face (defined as master as shown in Figure 5).

Behaviour of concrete is defined by using damaged plasticity approach, which includes compression hardening, tension softening, compression

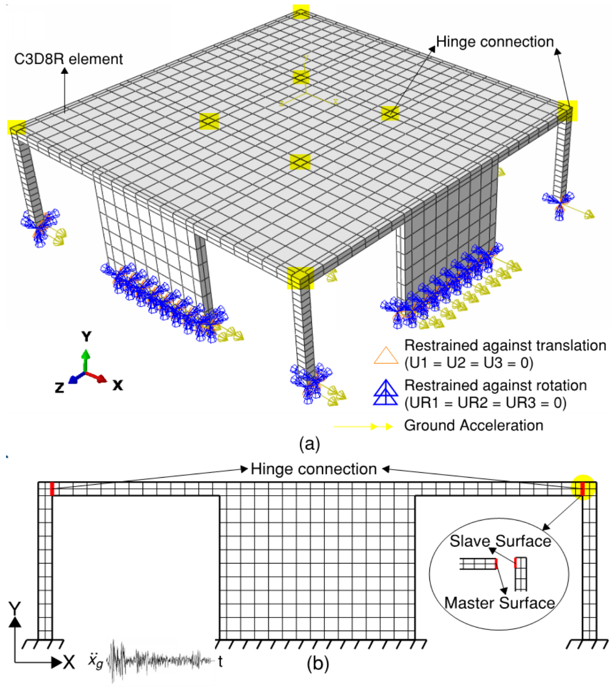

Figure 5: (a) Single storey reinforced concrete shear wall building (b) 2D section view

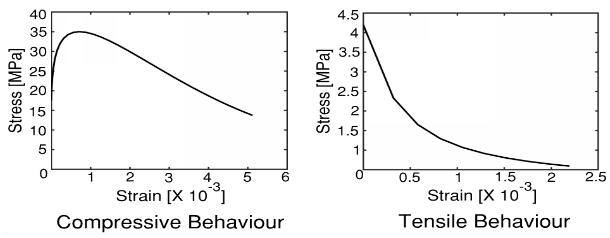

Figure 6: Stress - Strain behaviour of concrete in compression and tension

damage and tension damage (Figure 6, Nguyen and Livaoğlu (2020)). In the present study, damage property of concrete is omitted due to convergence difficulties at onset of softening.

Additional seismic mass is included at the floor level by using inertia property as nonstructural mass distributed over the volume. Rayleigh damping coefficients (α_m & β_k) are obtained by using first and second mode frequency at 5% damping ratio ($\zeta = \frac{1}{2}(\frac{\alpha_m}{\omega_i} + \beta_k \omega_i)$).

Structural responses of these models were obtained using implicit dynamic Newmark integration method by applying ground motion acceleration or displacement at the fixed base. Results are presented in next section.

4. Results and discussion

Response of the single storey RC model subjected to the Northridge 1994 earthquake (FEMA-P695 (2009)) at the top of shear wall and static push over analysis are shown in Figure 7(a) & (b) respectively. Figure 7(a) represents the inelastic response of the model obtained at the intensity level ($Sa(T_1)$) of $0.99g$ using implicit dynamic analysis. From Figure 7(b), ultimate base shear is observed approximately at $10mm$ using static pushover analysis. Further responses of these model were obtained for far-field set ground motions records with an aim to verify the characteristics of various intensity measures.

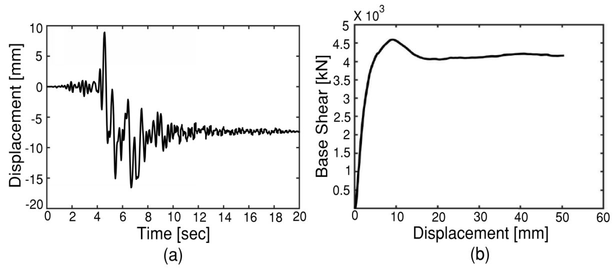

Figure 7: (a) Displacement of single storey RC building model at the top of shear wall subjected to Northridge 1994 earthquake (FEMA-P695 (2009)) (b) Static push over curve

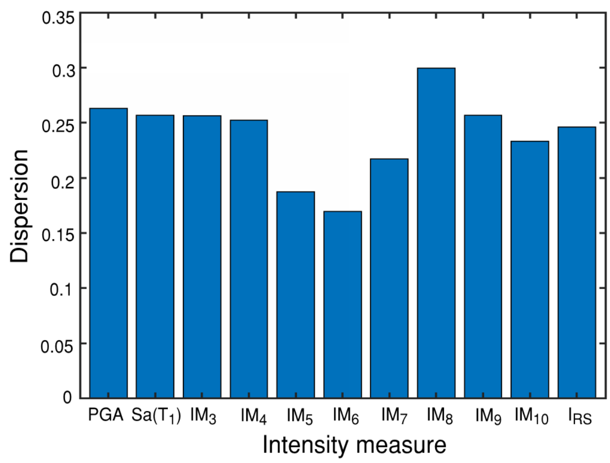

Figure 8: Dispersion of RC shear wall building

Figure 8 represents the variation in dispersion of various IMs for single storey RC building with shear walls. It can be observed from the graph that IM_6 is most efficient by limiting the dispersion to

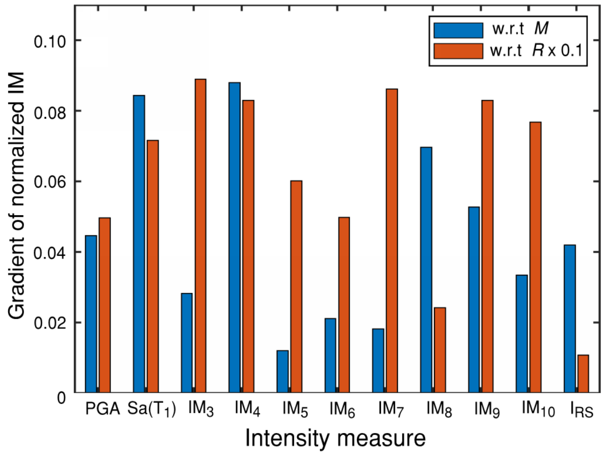

Figure 9: Slope of linear regression analysis with respect to R & M of RC shear wall building

17.56%. Except for the IM_6 & IM_5, dispersion of other IM's is beyond the 20% range. As discussed previously that proposed I_{RS} is more efficient for time periods more than $1sec$, for the present case dispersion is limited to 24.62%, which is lower than $Sa(T_1)$. Hence, proposed I_{RS} is moderately efficient for short period structures.

Simple relative sufficiency with respect to R and M for the proposed model is shown in Figure 9. It can be seen that proposed I_{RS} has the least slope with respect to R compared to the other IMs and for with respect to the M even though it is not the least, it is in the medium range. From these observations, it is understood that the proposed I_{RS} is moderately efficient and sufficient for short period structures.

5. Conclusions and future direction

Structural design to resist earthquakes fundamentally differs from gravity or wind design because damage is permitted at designated energy dissipating components within the system. Traditionally, a force based approach is used for earthquake engineering through response reduction factors that are calibrated to available ductility or energy dissipative capacity. The drawbacks of such an approach are well documented, and has spurred research for appropriate physical quantities that can represent structural demand and capacity in a seismic design context. Popular among these are peak ground acceleration, spectral acceleration and various geometric means of spectral acceleration coordinates.

A new intensity measure that calculates geometric mean of spectral acceleration coordinates over a predefined time period range (to account for higher mode effects) coupled with an exponent that is a function of the fundamental time period, was recently proposed by the authors. Studies presented in this paper demonstrate its high efficiency and sufficiency, particularly for structures with time periods greater than 1.0 *sec*. Various structural systems including SDOF and MDOF springs, steel frames with concentrated plastic hinges and single storey concrete shear wall buildings were considered. These preliminary studies are to be extended through more detailed modelling including structural damage, and to larger building systems with several bays and storeys.

The structural modelling protocol presented here has included components that are not part of the "lateral system", such as diaphragms and gravity frames, with the goal of capturing various primary and secondary limit states under ground acceleration records. An example "secondary limit state" is damage to diaphragms due to excessive global deformation. Future focus of this project will lie on further development towards a seismic system reliability approach to generally predict and prevent various primary and secondary limit states under earthquake loads.

6. Acknowledgment

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