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Average spectral acceleration: Ground motion duration evaluation

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Abstract. The quantitative assessment of the seismic collapse risk of a structure requires the usage of an optimal intensity measure (IM) which can adequately characterise the severity of the ground motion. Research suggests that the average spectral acceleration (Sa_{avg}) may be an efficient and sufficient alternate IM as compared to the more traditional first mode spectral acceleration, $Sa(T_1)$, particularly during seismic collapse risk estimation. This study primarily presents a comparative evaluation of the sufficiency of the average spectral acceleration with respect to ground motion duration, and secondarily assesses the impact of ground motion duration on collapse risk estimation. By assembling a suite of 100 historical ground motions, incremental dynamic analysis of 60 different inelastic single-degree-of-freedom (SDF) oscillators with varying periods and ductility capacities were analysed, and collapse risk estimates obtained. Linear regression models are used to comparatively quantify the sufficiency of Sa_{avg} and $Sa(T_1)$ using four significant duration metrics. Results suggests that an improved sufficiency may exist for Sa_{avg} when the period of the SDF system increases, particularly beyond 0.5, as compare to $Sa(T_1)$. In reference to the ground motion duration measures, results indicated that the sufficiency of Sa_{avg} is more sensitive to significant duration definitions that consider almost the full wave train of an accelerogram ($SD_{a5\text{-}95}$ and $SD_{v5\text{-}95}$). In order to obtain a reduced variability of the collapse risk estimate, the 5-95% significant duration metric defined using the Arias integral ($SD_{a5\text{-}95}$) should be used for seismic collapse risk estimation in conjunction with Sa_{avg}.

Keywords: average spectral acceleration; sufficiency; significant duration; earthquake-induce ground motion

1. Introduction

An essential aspect of seismic collapse risk assessment is the selection of hazard consistent ground motion for nonlinear time history analysis (Kwong and Chopra 2015, Lin *et al.* 2013, Bayati and Soltani 2016). Notable seismological parameters considered during selection include the distribution of earthquake's magnitude, source-to-site distance, epsilon and shear wave velocity, just to mention a few. Nevertheless, as noted by Bradley (2011), the severity of a seismic excitation depends primarily on three aspects; the frequency composition of the seismic waves, the amplitude of the accelerogram, and the duration of the earthquake-induced ground motion. A complete characterisation of an earthquake-induced ground motion must at least capture the duration of the portion of the signal where the seismic energy dominates, through a duration metric (Bommer *et al.* 2009). Nonetheless, explicitly accounting for the impact of ground motion when selecting ground motion for response history analysis, is yet to be fully considered (particularly in seismic design guidelines) since previous studies which sought to evaluate the influence of ground motion duration have contrasting views on this subject. However, ground motion duration becomes heavily important when structural systems are subjected to high magnitude earthquakes (Bradley 2011, Raghunandan and Liel 2013). Evidently, recent severe earthquakes such as those of Sumatra (magnitude 9.1, 2004), Mauli (magnitude 8.8, 2010) and Tohuku (magnitude 9.0, 2011) were of longer durations, with some sites experiencing ground shaking of about 40-270s (Luca *et al*. 2011). A potential reason for not explicitly considering the influence of ground motion duration in current assessment methodologies (code-based or performance-based), may be as a result of seismic deaggregation curves not providing severe mean intensities measures for an earthquake scenario at a particular exceedance level (2% or 10% probability in 50 years) during record selection. This is supported by one recent research work which reported no visible impact of ground motion duration for accelerograms with lower values of spectral acceleration, as opposed to accelerograms with higher spectral values and with longer durations, exhibiting larger peak interstory drift and damages (Barbosa *et al*. 2017). Other researchers conclude that the correlation between ground motion duration and spectral deformations as statistically insignificant (Iervolino *et al.* 2006, Shome *et al.* 1998, Ruiz-Garcia 2010). Nonetheless, as noted by Hancock and Bommer (2006), researches aimed at assessing the influence of ground motion duration should qualify their conclusions by stating for instance, the assumed constitutive models for components of the structural system, the damage measures considered, the primary intensity measure employed, and much more importantly the definition of the ground motion duration measure utilised during assessment. In spite of this inconclusive evidence and debate among researchers as to whether ground motion duration is an influential parameter

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that affects structural response, it is expected that accelerograms with longer duration be more damaging than those obtained from shorter duration, provided that these records have constant amplitude and similar frequency characteristics. Intuitively, given that two ground motions records have the same energy content, the likelihood of having the shorter duration record to be more destructive is also expected (Bommer *et al.* 2009, Bommer and Martínez-Pereira 2008).

The real issue is that the information conveyed by the influence of ground motion duration is very minimal and may not contribute that much to global damage of a structural system. Therefore, as noted by Bommer *et al.* (2009), it is desirable to identify a ground motion parameter that can quantify the effect of ground motion duration indirectly during seismic risk assessment. In view of the issues raised above, the search for ground motion parameters (intensity measures) that can effectively capture the influence of ground motion duration during seismic hazard analysis and collapse risk assessment is still ongoing. One practical approach is by employing an intensity measure (IM) that will provide a great deal of sufficiency with respect to ground motion duration. An intensity measure is said to be sufficient if estimated collapse intensities are not dependent on any other ground motion parameters employed during selection of accelerograms, but exclusively to the parameter under consideration. In other words, the bias in structural response quantities for a ground motion selection and modification (GSMS) procedure is reduced when a sufficient IM is employed. This is particularly advantageous in risk-based assessment; thus estimated site-specific seismic demand hazard curves will be unbiased provided selected records are hazard consistent with respect to an intensity measure that is sufficient (Kwong and Chopra 2015). A generally accepted frequency domain-based IM is the 5% damped first modal spectral acceleration ($Sa(T_1)$). As opposed to this scalar IM, other current approaches for quantifying the severity of ground motions, have focused on vector intensity measures. Bradley (2010), developed a holistic and generalized framework for ground motion selection that allows for any number of intensity measures under the assumption that the vector of intensity measures follow a multivariate lognormal distribution. This framework identified the shortfall of the originally proposed conditional mean spectrum (Baker 2011, Vacareanu *et al.* 2014), an alternative target spectrum for ground motion selection, which primarily assumes that the severity of a ground motion can be quantified solely by its spectral acceleration. It admits any vector of intensity measures which are deemed to significantly affect structural response. The definition of a correlation matrix for the vector of IMs is required. Researches in the development of empirical correlation equations for ground motion duration parameters such as significant duration, with other intensity measures are still ongoing (Bradley 2011, Jayaram *et al.* 2011, Lee 2012). The lack of these empirical correlation relationships, coupled with the inadequate validation of the assumed joint probability distribution of the vector intensity measures, makes the use scalar intensity measures in seismic risk assessment still a suitable alternative.

One recently proposed intensity measure that has been reported to be efficient (reduced variability in collapse estimates) and sufficient for probabilistic seismic demand analysis, is the average spectral acceleration (geometric mean of spectral ordinates within a bounded period range) (Sa_{avg}) (Eads *et al.* 2015). By using the first modal period of vibration (T_1) as the primary period, they suggested that a period range of 0.2 T_1-3.0 T_1 at 0.01s spacing be used for computing this IM.

$$Sa_{avg}(r_1T_1,\ldots,r_nT_1) = \sqrt[n]{\prod_{i=1}^{n} Sa(r_iT_1)} \tag{1}$$

A comparative assessment of Sa_{avg} with $Sa(T_1)$ as intensity measures for collapse risk assessment revealed that the likelihood of Sa_{avg} to being more sufficient than $Sa(T_1)$ exists, when an appropriate period range for definition is considered, as suggested above. Nevertheless, IM sufficiency was evaluated with respect to magnitude, source-to-site distance and epsilon. In a bid to fully appreciate the advantage of using Sa_{avg} for providing a more stable collapse risk estimate, the present paper seeks to evaluate the sufficiency of Sa_{avg} with respect of ground motion duration.

2. Ground motion-duration metrics

The frequency content, amplitude and duration of an earthquake time series are important parameters that reflects some particular feature of the seismic excitation (Bommer *et al.* 2009, Bradley 2011). Numerous definitions of ground motion duration have been proposed, and as recognized by (Bommer *et al.* 2009, Hancock and Bommer 2006), each definition can be found in one of three major categories, i.e., significant, bracketed and uniform duration. These definitions have been formulated in order to capture the portion of the earthquake signal that is considered strong. Nevertheless, most research activities heavily focus on significant duration metrics for assessing ground motion duration effect of structural response (Kempton and Stewart 2006). Bracketed duration is computed from an acceleration time history by selecting specified threshold acceleration and finding the time difference between the first and last excursion of the signal that is above this threshold. Therefore, one can infer that this definition is highly subjective, since it depends on the choice of thresholds considered. Uniform duration also requires the specification of threshold acceleration; however, it is computed as the time interval for which the acceleration time history is above this threshold; hence making it shorter than bracketed acceleration computed at the same threshold. These two-duration metrics can be viewed as absolute metrics because of the need for predefined threshold acceleration. Another disadvantage of using these metrics is that it tends to zero when peak ground acceleration of a record is lesser than the threshold acceleration. Hence, a suitable and preferred measure for characterizing ground motion is the significant duration (*SD*). Generally, and conventionally, it is defined as the time interval within which a bounded portion of the Arias integral is accumulated for a particular acceleration time history. The Arias intensity (*AI*) defines the total energy of a particular accelerogram and can be computed by evaluating the following integrals

$$AI = \frac{\pi}{2g}\int_0^T a^2(t)dt \qquad (2)$$

where $a(t)$ the evolutionary acceleration ordinate of the accelerogram is, g is the acceleration due to gravity and T is the total ground motion recorded duration. With the aid of Husid plots (Husid 1969), which characterizes the build-up of released seismic energy, the energy bounds for which the significant duration is computed can be easily specified.

The two commonly used bounds that are herein adapted in this study are the 5-75% and 5-95% of Arias integral; with the former representing the energy content from the body waves whereas the latter nearly accounting for the full wave train. We denote the time interval, that is, the significant duration for 5-75% Arias integral as $SD_{a5\text{-}75}$ and that for 5-95% Arias integral as $SD_{a5\text{-}95}$. The subscript '*a*' has been attached since the evaluated Arias integral were from acceleration summations. These two definitions are arguably the most used proxies, however as noted by Sarma (1971), an alternate to energy characterization, currently referred as energy integral (Kempton and Stewart 2006) can also be used for quantify ground motion duration. The energy integral is computed as

$$EI = \int_0^T v^2(t)dt \qquad (3)$$

We therefore define $SD_{v5\text{-}75}$ and $SD_{v5\text{-}95}$ as the time intervals for which 5-75% and 5-95% of the energy integral area cumulated respectively. Using these duration measures ($SD_{a5\text{-}75}$, $SD_{a5\text{-}95}$, $SD_{v5\text{-}75}$ and $SD_{v5\text{-}95}$) the paper primarily focuses on evaluating the sufficiency of Sa_{avg} with respect to ground motion duration when selected as an intensity measure for collapse risk assessment.

3. Methodology

The sufficiency of IM was primarily assessed using single degree of freedom (SDF) systems. The natural oscillatory periods of vibration of considered SDF system ranged from 0.1-1.0, at intervals of 0.1. For each SDF system, a tri-linear backbone curve with 20% hardening slopes is defined at six target displacement ductility levels (U_f=2, 4, 6, 8, 10 and 12). The capping ductility was specified as 90% of the target displacement ductility (Adom-Asamoah and Osei 2018). A mass proportional damping at 5% damping ratio was also accounted for. The defined constitutive model is able to admit strength deterioration and stiffness degradation, as well as in-cyclic strength degradation (Adom-Asamoah and Osei 2016). A similar approach to modelling such SDF systems was implemented by Mousavi *et al.* (2011), however they neglected the influence of cyclic deterioration for the sake of simplicity.

By selecting a total of 100 arbitrary ground motions from shallow crustal earthquakes, nonlinear response history analyses are performed. The selected bin of

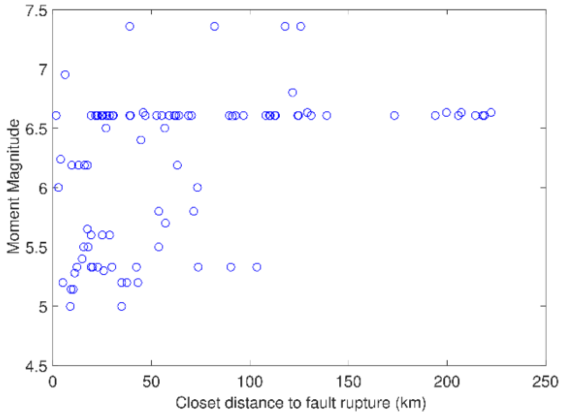


Fig. 1 Magnitude distance distribution of arbitrarily selected ground motions

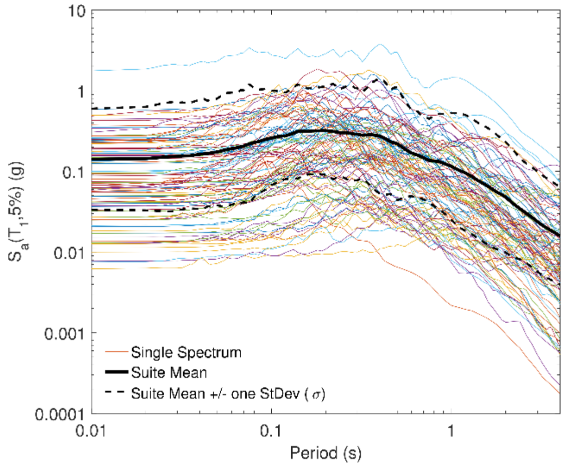
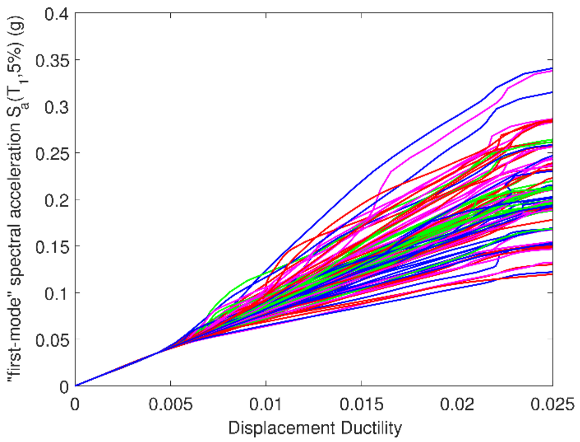


Fig. 2 Response spectra of selected records at 5% damping ratio in the logarithmic scale

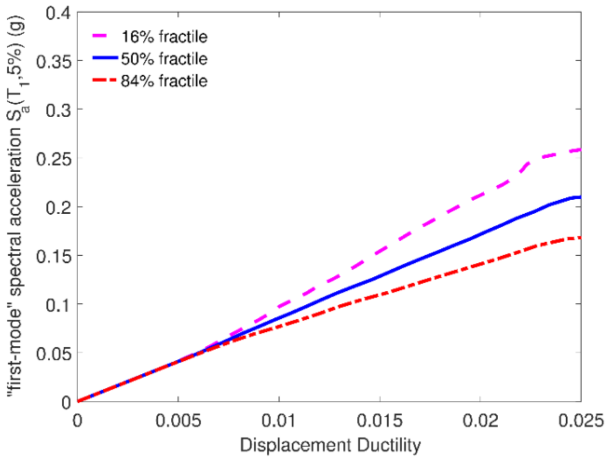
historical records was within a magnitude range of 5-7.36 with their closest distance to fault rupture lying between 10-230 km (see Fig. 1).

These ground motions were obtained from the PEER strong ground motion database. The acceleration response spectra of the selected suite of ground motions at 5% damping is shown in Fig. 2. Also shown in Table A1 of Appendix 1, are the significant duration measures for the individual records.

Through incremental dynamic analysis (IDA), seismic collapse capacities were obtained with the aid of the hunt and fill algorithm (Vamvatsikos and Cornell 2002) (see Fig.3). Two demand measures discussed above, Sa_{avg} and $Sa(T_1)$, were used for seismic collapse capacity evaluation. The collapse point is a function of the structure (SDF system) and ground motion (Kwong and Chopra 2015), and is defined as the minimum intensity at which an arbitrarily selected ground motion record causes dynamic instability (larger displacement amplitudes when the intensity measure is increased marginally (Vamvatsikos and Cornell 2002)). This procedure has been extensively used in other researches (Eads *et al.* 2015, Mousavi *et al.* 2011) for seismic collapse assessment. Hunt and fill algorithm ensured that within a specified tolerance, additional



(a)



(b)

Fig. 3 Incremental dynamic analysis output for SDOF system having vibrational period of 0.7 and ductility value of 4

response history analyses are performed in the vicinity of the last range of intensities for accurately location of collapse point. Alternatively, less efficient numerical algorithm such as bisection method could be employed.

The sufficiency of these two collapse intensity measures (Sa_{avg} and $Sa(T_1)$) were evaluated for the four ground motion duration measures ($SD_{a5\text{-}75}$, $SD_{a5\text{-}95}$, $SD_{v5\text{-}75}$ and $SD_{v5\text{-}95}$) as discussed above. By IM sufficiency, we mean that the estimated collapse capacity for a particular structural system is not affected by the distribution of the duration measures of the ground motion set employed. In evaluating the sufficiency of IM, a linear regression analysis is performed for each structural system (period and ductility dependent). Initially, the natural logarithms of the estimated collapse capacities for each arbitrary record, alongside its constant duration metric are obtained for a particular structural system. Using the duration metric under consideration as the explanatory variable and the collapse capacity as the response variable, a standard linear regression model is formulated as given in Eq. (4).

$$E[IM|SD=x]=\beta_o+\beta_1 x \tag{4}$$

where $E\ [IM|SD=x]$ is the expected value of the collapse intensity conditioned on some value x, the significant duration of the ground motion, and β_o and β_1 are regression coefficients. In order to ensure that $E\ [IM|SD=x]$ does not depend on some value of x (that is $E\ [IM|SD=x]=\beta_o$), a coefficient hypothesis test is performed, where the null hypothesis is defined as $\beta_1=0$. A 5% significance level is selected since it is normally used for studies that have sought to evaluate the sufficiency of an intensity measure (Eads *et al.* 2015, Iervolino *et al.* 2006). The null hypothesis is rejected when the p-value is less than this significance level (0.05), and the intensity measure is deemed not sufficient with respect to the particular ground motion duration for a given structural system. Utilizing the 10×6 SDF systems, for the 100 records, a 100×60 matrix of seismic collapse capacity data is created for each IM for appropriate statistical analysis.

4. Results and discussion

4.1 Evaluation of IM with respect to ground motions duration metrics

4.1.1 Assessment based on vibrational period

As previously discussed, the null hypothesis of acquiring a collapse capacity (IM) which does not depend on distribution of ground motion duration metrics within an arbitrary ensemble of ground motion dataset ($\beta_1=0$) is accepted given that the p-value for a particular structural system (period and ductility dependent) is greater than the chosen 5% significance level. Fig. 4 shows the distribution of *p*-values at various vibrational periods of the SDF systems irrespective of the displacement ductility level. Also shown is a dashed line that corresponds to a p-value of 0.05; above which the collapse intensity measure (Sa_{avg} and $Sa(T_1)$) investigated is deemed to be sufficient with respect to a particular class of ground motion duration metric ($SD_{a5\text{-}75}$, $SD_{a5\text{-}95}$, $SD_{v5\text{-}75}$ and $SD_{v5\text{-}95}$).

Results revealed that by computing the collapse point using the defined average spectral acceleration (Sa_{avg}), none of the considered SDF systems that possessed a vibrational period of 0.1, were sufficient with respect to any of the ground motion duration metrics considered. Comparatively, with the exception of ground motion duration metrics that were defined using the energy integral ($SD_{v5\text{-}75}$ and $SD_{v5\text{-}95}$) having resulted in 50% of the SDF systems being sufficient for $Sa(T_1)$ at T_1 of 0.1, those duration metrics defined using the Arias integral ($SD_{a5\text{-}75}$ and $SD_{a5\text{-}95}$) did not allow for IM sufficiency. By inspection, there is a gradual improvement in the sufficiency of Sa_{avg} with respect to $SD_{a5\text{-}75}$ as the vibrational period of the SDF systems increases with the exception of those systems with T_1 of 0.8 and at displacement ductility levels of 8 and 10. Nonetheless, the performance of Sa_{avg} as a sufficient intensity measure was improved when the period range is limited to the SDF period range of 0.5-1.0. Particularly, the proportions of SDF systems within this period range which were sufficient with respect to $SD_{v5\text{-}95}$ and $SD_{a5\text{-}95}$ for Sa_{avg}, were 78% and 100% respectively (see Table 1). Nonetheless, it appears that ground motion duration metrics that considers only the body waves of the accelerograms ($SD_{a5\text{-}75}$ and $SD_{v5\text{-}75}$) may

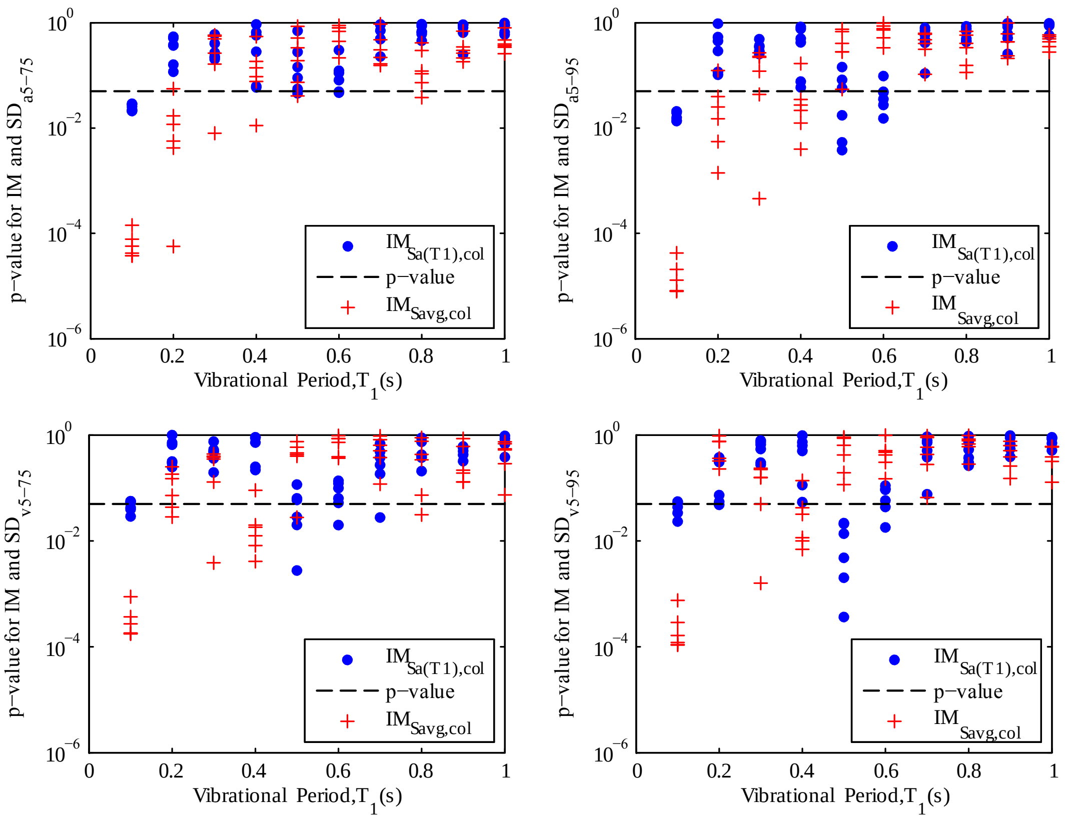


Fig. 4 *p*-values for IM and significant duration metrics considering periods of SDF systems

Table 1 Percentage of SDF systems with p-value > 0.05

Significant duration measure	$0 < T_1 < 1.0$		$0 < T_1 < 0.5$	
	$Sa(T_1)$	Sa_{avg}	$Sa(T_1)$	Sa_{avg}
$SD_{a5\text{-}75}$	83	75	89	94
$SD_{a5\text{-}95}$	77	70	78	100
$SD_{v5\text{-}75}$	87	73	86	94
$SD_{v5\text{-}95}$	80	78	78	100

allow for improved sufficiency when the collapse capacity is defined as $Sa(T_1)$ (see Table 1) as compared to those that considered the full wave train of the ground motion ($SD_{a5\text{-}95}$ and $SD_{v5\text{-}95}$). On the other hand, Sa_{avg} can be deemed as more sufficient than $Sa(T_1)$ when it is related to ground-motion significant duration metrics that employs the full wave train of responses ($SD_{a5\text{-}95}$ and $SD_{v5\text{-}95}$) for SDF systems within the period range of 0.5-1.0 (see Fig. 4).

4.1.2 Assessment based on ductility capacity

The effect of displacement ductility capacities of SDF systems on sufficiency of the aforementioned intensity measures is also evaluated with respect to the considered duration metrics (see Fig. 5). As seen pictorially, there is no observed trend in the sufficiency of selected intensity measure when considering ductility, for any ground motion duration metric. However, it appears $Sa(T_1)$ may be much more sufficient than Sa_{avg}, particularly for SDF systems with lower vibrational periods and lower ductility capacities. Evidently for $SD_{a5\text{-}95}$ and $SD_{v5\text{-}95}$, Sa_{avg} proved more sufficient irrespective of the ductility level of SDF systems when compared to $Sa(T_1)$. As noted by Novikova and Trifunac (1994), the duration of an accelerogram depends on the frequency of the motion. The predominant and peak frequencies of an earthquake-induced motion plays an important role in the damaged sustained by a system under excitation as argued by Ö zer *et al.* (2012). Severe ground motion with comparatively longer duration will normally have the locations and values of their frequencies different (Ö zer *et al.* 2012). This suggest that by using ground motion significant duration metrics which employs the full wave train of pulses, the analyst may be able to capture their effects as the predominant and peak frequencies may be located within that boundary, as opposed to those that considers the body waves only. This can be one physical reason to explain the observed results.

4.2 Influence of ground motion duration on structural collapse using Sa_{avg}

Assessment of the effect of ground motion duration on the collapse capacity of a structural system is highly influenced by the measure used to quantifying the severity of the ground shaking, i.e., the intensity measure employed in the risk assessment framework. Exclusive to the intensity measure selected for the seismic performance assessment, other structural properties of the system, such as its first modal period of vibration, the extent of deformability quantified using a ductility measure, and the influence of spectral shape, are among the many influential parameters that affects the collapse capacity of a structure. It now generally agreed that intensity measures that explicitly or implicitly capture the effect of spectral shape should be

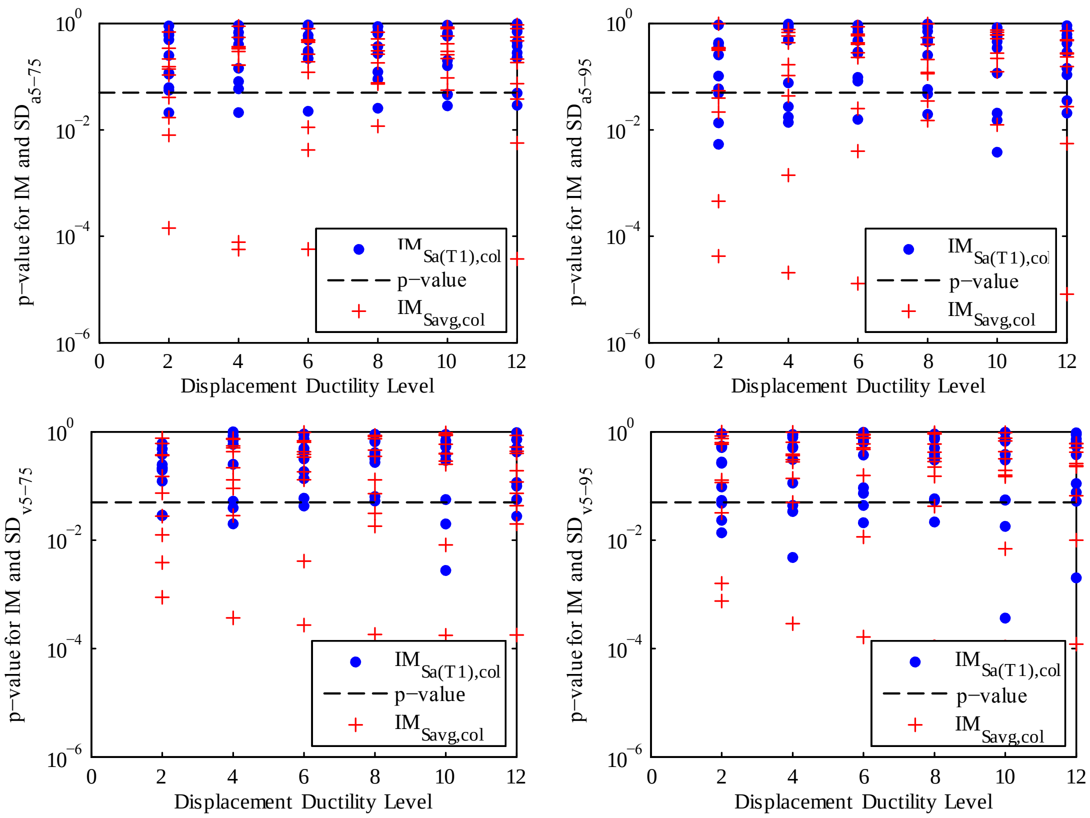


Fig. 5 *p*-values for IM and significant duration metrics considering ductility of SDF systems

used in collapse risk assessment, since the shape of the response spectra of ground motions that cause collapse are of a peculiar type. Since $Sa(T_1)$ represents the value of the response spectral at a single period, it is unable to capture the influence of higher modes and the effect of period elongation of building configurations that are susceptible to these variations. Hence record-to-record variability which emanates from performing nonlinear time history analysis using a suite of ground motions is expected to be larger when $Sa(T_1)$ is employed during risk assessment. Nonetheless, as noted by (Eads *et al*. 2016), using the ratio of $Sa(T_1)$ to Sa_{avg} (*SaRatio*) as a metric of spectral shape is more suitable for predicting collapse intensity as compared to other proxies such as *epsilon* (Baker and Cornell 2008), *eta* (Mousavi *et al.* 2011) and *Np* (Bojórquez and Iervolino 2011). This suggests that selecting records without explicitly considering spectral shape, it may be useful to perform the seismic risk assessment by utilizing Sa_{avg} since it implicitly captures spectral shape.

In quantifying the influence of ground motion duration, a generalized linear model framework is formulated. One main assumption of the crude least-square regression model is that the probability distribution of the response variable is normal. Various transformation methods are applied to data whose empirical distributions are not normal (for instance using a logarithmic or square root transformation) before parametric estimation is carried out. Generalized linear models seek to relax this assumption by providing link or linear transformation functions for any arbitrarily distribution of response variables. It is generally agreed that an analyst can approximate the distribution of collapse intensities of response history analysis as lognormal. This explains why we effected a natural logarithmic transformation to the collapse estimates used for assessing the sufficiency the considered IMs with respect to duration discussed above. In this section, we assume a gamma distribution for the estimated collapse intensities. This positively skewed distribution is able to closely match that of a lognormal distribution by varying its parameters. In addition, since collapse intensities are positive, they conform to the strict domain of values valid for a gamma distribution. For a normal distribution, the link function is identity, whereas a reciprocal link function exists to relate the expected values of the response variable to that of the predictor variables when the distribution of the responses is gamma-approximated. Using a particular significant duration metric, the ductility of the SDF system and its oscillatory period, a gamma distributed generalized linear model is presented as in Eq. (5).

$$E[Sa_{avg}] = (X'\beta)^{-1} \tag{5}$$

Where X' is a row vector of predictor variables, β is the column vector of estimated regression coefficients, and $E[Sa_{avg}]$ is the expected value of the collapse intensity. In performing this regression study, we used the full set of collapse intensities estimated for the 60 SDF systems understudy. Table 2 presents estimates of the model parameters when a particular ground motion duration measure is employed in this nonlinear multivariate generalised regression model. In order to ascertain the reliability, robustness and accuracy of these models at

Table 2 Model parameters of generalized linear models

	Significant duration model parameters			
	$SD_{a5\text{-}75}$	$SD_{a5\text{-}95}$	$SD_{v5\text{-}75}$	$SD_{v5\text{-}95}$
Intercept	-0.3633	-0.3578	-0.3628	-3.61E-01
Ductility	-0.0003	-0.0003	-0.0003	-0.0003
Period	4.85859	4.85874	4.85932	4.860192
Duration	-0.0011	-0.0009	-0.0008	-0.00059

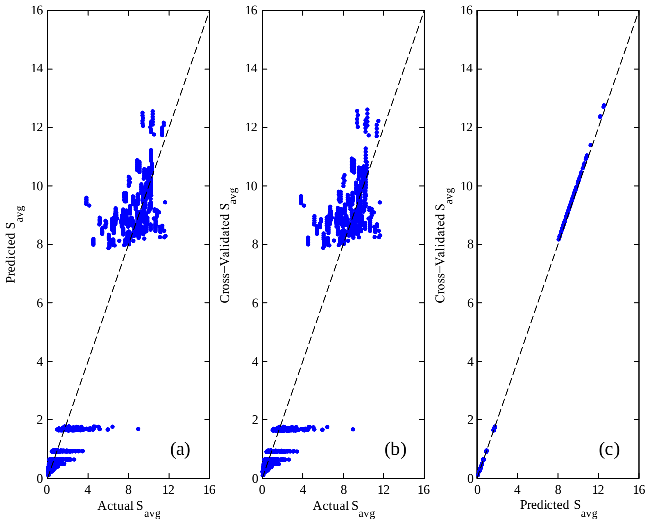


Fig. 6 Cross-Validation results for $SD_{a5\text{-}95}$ using the generalized linear model of Sa_{avg}

predicting the seismic collapse capacities, a cross-validation analysis of the generalized models is performed. In cross-validation, we initially assemble a reduced dataset (preferably about 90% of the original dataset (Raghunandan and Liel 2013)) and perform a regression analysis to obtain a new predictive model (Fig. 6(b)). Using this new predictive model, collapses intensity estimates of the dataset that was excluded is computed. Finally, to ascertain the reliability of the original generalized linear model (Fig. 6(a)), an *x-y* plot of the estimated collapse intensity for both the original and the new predictive models is graphed, and its robustness is characterised by its ability to lie along the 45^o line. Fig. 6 shows the cross-validation results for the $SD_{a5\text{-}95}$ generalized model presented in Table 2. This was obtained by randomly excluding one-sixth of the complete dataset. As evidently seen, the final regression model was adequate at predicting the collapse intensities for the excluded dataset (Fig. 6(c)).

As seen in Table 2, the sign of the significant duration parameter suggests an increase in collapse intensity when an accelerogram has a larger significant duration, which seems counter-intuitive. It is generally expected that since longer significant duration records may have a comparatively large number of cycles within that time interval were most of the seismic energy is released, significant strength and stiffness deterioration will prevail, thereby accelerating the rate of collapse. Nonetheless, in the situations where the seismic energy of two accelerograms are the same, it is expected that the short duration accelerogram may have larger pulses that may cause abrupt

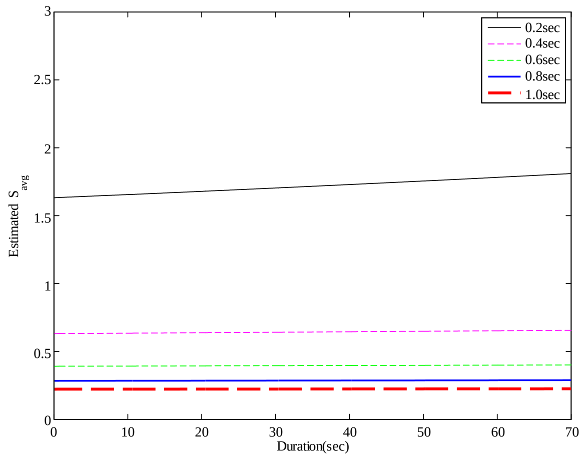


Fig. 7 Effect of duration on seismic collapse estimates for SDOF systems with different periods

Table 3 Metrics for assessing adequacy of fitted generalized linear models

	$SD_{a5\text{-}75}$	$SD_{a5\text{-}95}$	$SD_{v5\text{-}75}$	$SD_{v5\text{-}95}$
p-value for ductility	0.62716	0.62721	0.62694	0.62691
Dispersion	0.23617	0.23608	0.23638	0.23674
R-squared	0.93949	0.94105	0.93967	0.94012
Sum of squared error	2.17E+03	2.11E+03	2.16E+03	2.15E+03

collapse of the system. However, as seen in Table 2, the effect of significant duration of the accelerogram may not be substantial, particularly for systems with larger periods of vibration since, the weighting functions (model parameters) are on the extremes. In order to explore the validity of this assertion, we provide a plot of the predicted collapse intensities against the significant duration, for 5 different SDF systems having varying fundamental periods of vibration, but all with a constant ductility capacity (Fig. 7).

The graph suggests that the effect of ground motion duration of collapse intensity estimation dies off when the fundamental modal period of vibration increases (particularly above 0.5 for this study). This suggests sufficiency of the IM (Sa_{avg}) at predicting collapse estimates that are independent of the distribution of the record set collected for response history analysis. Similar findings were obtained when the ductility capacity of the SDF systems was varied, suggesting that the influence of the deformability limit of the SDF system is not significant in seismic collapse intensity estimation. This is statistically confirmed with p-values of the ductility measure in the generalized linear models being greater than 0.05, hence suggesting its mild influence of collapse intensity estimation (see Table 3).

4.3 IM efficiency

The search for an optimal seismic IM is particular important in collapse risk assessment. Notably measures for identifying an optimal IM are its ability to be sufficient at characterising the severity of the ground motion; its propensity to provide collapse estimates with low variability when a suite of ground motion is employed for response history analysis (efficiency); easily related to the hazard posed at a site through a ground motion prediction model (predictability); and it distribution not being biased when scaled accelerograms are employed as compared to un-scaled ground motions (scale robustness) (Bradley *et al*. 2010). IM efficiency ensures that the irreducible aleatorical uncertainties that emanates from utilizing different records possessing varying frequency contents is minimal. It is normally quantified in seismic collapse risk assessment by the standard deviation or dispersion of the collapse intensities for a specific structure. A preliminary comparative assessment of the dispersion in the generalized model for both collapse intensities metrics (Sa(T_1) and Sa_{avg}) studied revealed that quantification using the average spectral acceleration leads to reduced variability comparatively. Furthermore, as reported in Table 3, the best ground motion significant duration metric which improves efficiency of seismic collapse estimation is the $SD_{a5\text{-}95}$. This significant duration metric has already been identified as the most used in studies characterizing the influence of ground motion duration on structural collapse. Finally, it is worth noting that all generalized linear model yielded high coefficient of determination greater than 0.9.

5. Conclusions

The present study primarily sought to evaluate the sufficiency of a ground motion intensity measure in the quantification of the seismic collapse risk of structural system, with respect to ground motion duration. Previous research works has revealed that the average spectral acceleration, an intensity measure, is efficient and sufficient for performing probabilistic seismic risk assessment. Nonetheless, such studies evaluated its sufficiency with respect to causal parameters such as magnitude, source-to-site distance and epsilon. There was no explicit consideration of the effect of ground motion duration in such studies. However, the state-of-knowledge of the effect of ground motion duration on structural response have still not totally ascertained its influence, with various researches suggesting contrasting views. A comparative assessment of the sufficiency of two intensity measures with respect to ground motion duration (average spectral acceleration and the conventional first mode spectral acceleration) was drawn by assembling a suite of ground motions and performing incremental dynamic analysis, in order to determine the seismic collapse capacity of inelastic SDF systems of different periods of vibration and ductility capacities.

Results revealed that improved sufficiency of the average spectral acceleration (Sa_{avg}) may exist when the vibrational period of the single-degree-of-freedom (SDF) oscillator is within the range of 0.5-1.0. This suggests that it may be a suitable collapse intensity measure for mid-rise buildings of comparable periods of excitation. In reference to the ground motion duration measures considered in this study, results indicated that significant duration definitions that considers the almost the full wave train ($SD_{a5\text{-}95}$ and $SD_{a5\text{-}95}$) may be suitable for ensuring sufficiency of the average spectral acceleration (Sa_{avg}), whereas those definitions that considers only the body wave of the accelerogram ($SD_{a5\text{-}75}$ and $SD_{a5\text{-}75}$), the use of the conventional first mode spectral acceleration may be preferred for collapse risk assessment. In all circumstances, no apparent trend exists for the spatial variations in the ductility capacities of the single-degree-of freedom systems, with respect to sufficiency.

Secondarily, by approximating the empirical distribution of the collapse intensity estimate that were defined using the average spectral acceleration (Sa_{avg}) as gamma distributed, a generalized linear regression model with an inverse link function was formulated in order to assess the impact of ground motion duration on seismic collapse risk estimation. Results indicated that improved efficiency (quantified using the standard deviation of the models) exists for SDF systems with periods above 0.5, and for models that employs the 5-95% significant duration metric defined using the Arias integral ($SD_{a5\text{-}95}$). Further research work should be carried out to fully corroborate the findings of this limited study. Verification studies on other structural systems such as deteriorating multi-degree-of-freedom systems considering the influence of P-delta effect should be performed in order to ascertain the sufficiency of the average spectral acceleration for seismic collapse risk assessment.

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Appendix A

Table A1 Ground motion significant duration metrics

Earthquake				Recording Station	SD_{a5-75}	SD_{a5-95}	SD_{v5-75}	SD_{v5-95}
ID NO.	M	Year	Name	Name				
1	6	1935	Helena_ Montana-01	Carroll College	1.2	2.5	1.3	9.7
2	6	1935	Helena_ Montana-02	Helena Fed Bldg	0.3	0.8	0.6	9.4
3	5.8	1937	Humbolt Bay	Ferndale City Hall	9.5	23.2	17.4	29.9
4	5	1938	Imperial Valley-01	El Centro Array #9	7.6	15.8	9.6	17.3
5	5.5	1938	Northwest Calif-01	Ferndale City Hall	4.1	11.6	9.0	13.9
6	6.95	1940	Imperial Valley-02	El Centro Array #9	17.7	24.2	10.1	25.7
7	6.6	1941	Northwest Calif-02	Ferndale City Hall	9	22.2	14.0	29.0
8	6.4	1941	Northern Calif-01	Ferndale City Hall	4.8	15.5	9.4	22.0
9	6.5	1942	Borrego	El Centro Array #9	21.5	37.2	33.1	38.8
10	5.6	1951	Imperial Valley-03	El Centro Array #9	15.2	27.6	16.2	26.4
11	5.8	1951	Northwest Calif-03	Ferndale City Hall	5.8	15.4	10.8	21.6
12	7.36	1952	Kern County	LA - Hollywood Stor FF	18.6	33.5	21.0	37.0
13	7.36	1952	Kern County	Pasadena - CIT Athenaeum	16.6	29.5	24.0	38.1
14	7.36	1952	Kern County	Santa Barbara Courthouse	12.4	33.6	12.8	34.5
15	7.36	1952	Kern County	Taft Lincoln School	10.7	30.3	35.7	42.0
16	5.2	1952	Northern Calif-02	Ferndale City Hall	5.7	18.4	9.8	28.8
17	6	1952	Southern Calif	San Luis Obispo	3.8	13.3	11.9	20.1
18	5.5	1953	Imperial Valley-04	El Centro Array #9	12.2	14.3	12.2	15.2
19	5.3	1954	Central Calif-01	Hollister City Hall	9.8	25.1	14.1	35.7
20	6.5	1954	Northern Calif-03	Ferndale City Hall	6.8	19.4	7.7	17.8
21	5.4	1955	Imperial Valley-05	El Centro Array #9	8.4	20	10.4	27.1
22	6.8	1956	El Alamo	El Centro Array #9	23	40.9	27.7	44.3
23	5.28	1957	San Francisco	Golden Gate Park	1.2	5	1.6	8.8
24	5	1960	Central Calif-02	Hollister City Hall	13.6	34.7	22.6	43.8
25	5.7	1960	Northern Calif-04	Ferndale City Hall	10	28.4	25.6	53.6
26	5.6	1961	Hollister-01	Hollister City Hall	9.9	18.7	13.8	25.5
27	5.5	1961	Hollister-02	Hollister City Hall	7.6	16.5	12.4	26.0
28	6.19	1966	Parkfield	Cholame - Shandon Array #12	14.7	29	20.4	31.2
29	6.19	1966	Parkfield	Cholame - Shandon Array #5	2.4	7.5	10.2	31.6
30	6.19	1966	Parkfield	Cholame - Shandon Array #8	5.9	13.1	6.9	17.8
31	6.19	1966	Parkfield	San Luis Obispo	8	17.8	11.3	18.7
32	6.19	1966	Parkfield	Temblor pre-1969	1.3	5.5	10.9	22.4
33	5.6	1967	Northern Calif-05	Ferndale City Hall	4.1	22.1	4.1	9.7
34	5.2	1967	Northern Calif-06	Hollister City Hall	15.4	32.1	13.6	41.4
35	6.63	1968	Borrego Mtn	El Centro Array #9	25	49.3	18.5	47.5
36	6.63	1968	Borrego Mtn	LA - Hollywood Stor FF	17.9	26.3	37.9	60.7
37	6.63	1968	Borrego Mtn	LB - Terminal Island	31.9	37.9	18.9	28.1
38	6.63	1968	Borrego Mtn	Pasadena - CIT Athenaeum	25	37.4	34.1	36.8
39	6.63	1968	Borrego Mtn	San Onofre - So Cal Edison	19.3	28	26.3	37.2
40	5.33	1970	Lytle Creek	Castaic - Old Ridge Route	4.9	10.1	20.6	26.5
41	5.33	1970	Lytle Creek	Cedar Springs Pumphouse	1.4	3.2	7.0	11.6
42	5.33	1970	Lytle Creek	Cedar Springs_ Allen Ranch	3.4	21.9	1.1	4.8
43	5.33	1970	Lytle Creek	Colton - So Cal Edison	7.5	18.3	12.2	28.2
44	5.33	1970	Lytle Creek	Devil's Canyon	1	2.2	10.5	28.8
45	5.33	1970	Lytle Creek	LA - Hollywood Stor FF	5.2	12.8	1.2	4.5
46	5.33	1970	Lytle Creek	Lake Hughes #1	3.9	6.9	7.2	13.5
47	5.33	1970	Lytle Creek	Puddingstone Dam (Abutment)	5.4	10.2	3.6	7.4
48	5.33	1970	Lytle Creek	Santa Anita Dam	2.9	5.1	6.2	10.2
49	5.33	1970	Lytle Creek	Wrightwood - 6074 Park Dr	1.7	3.2	2.3	4.9
50	6.61	1971	San Fernando	2516 Via Tejon PV	27.6	54.2	1.2	4.9
51	6.61	1971	San Fernando	Anza Post Office	8.1	15.4	40.2	50.1
52	6.61	1971	San Fernando	Bakersfield - Harvey Aud	24.1	35.3	9.8	21.9

Table A1 Continued

53	6.61	1971	San Fernando	Borrego Springs Fire Sta	13.6	22	32.5	35.6
54	6.61	1971	San Fernando	Buena Vista - Taft	13.6	21.6	16.7	19.3
55	6.61	1971	San Fernando	Carbon Canyon Dam	9	18.9	12.2	18.3
56	6.61	1971	San Fernando	Castaic - Old Ridge Route	10.6	16.8	20.2	28.2
57	6.61	1971	San Fernando	Cedar Springs Pumphouse	4.6	10.2	11.5	26.3
58	6.61	1971	San Fernando	Cedar Springs_ Allen Ranch	5.8	10.4	4.7	9.5
59	6.61	1971	San Fernando	Cholame - Shandon Array #2	17.9	25.5	9.7	11.6
60	6.61	1971	San Fernando	Cholame - Shandon Array #8	16.9	23.7	18.2	25.5
61	6.61	1971	San Fernando	Colton - So Cal Edison	5.1	7.3	19.6	24.9
62	6.61	1971	San Fernando	Fairmont Dam	3.6	14.4	5.8	7.3
63	6.61	1971	San Fernando	Fort Tejon	5	8.3	15.7	37.1
64	6.61	1971	San Fernando	Gormon - Oso Pump Plant	4.1	7.2	4.5	8.0
65	6.61	1971	San Fernando	Hemet Fire Station	9.4	23.9	4.9	6.4
66	6.61	1971	San Fernando	Isabella Dam (Aux Abut)	20.2	26.5	17.5	33.8
67	6.61	1971	San Fernando	LA - Hollywood Stor FF	5.2	13.4	25.0	32.7
68	6.61	1971	San Fernando	LB - Terminal Island	41.5	52.4	8.3	16.8
69	6.61	1971	San Fernando	Lake Hughes #1	8	18.4	32.9	42.3
70	6.61	1971	San Fernando	Lake Hughes #12	2.8	12	3.1	15.1
71	6.61	1971	San Fernando	Lake Hughes #4	4.2	13	3.1	15.2
72	6.61	1971	San Fernando	Lake Hughes #9	3	11.8	4.3	17.8
73	6.61	1971	San Fernando	Maricopa Array #1	18.2	29.6	4.9	16.8
74	6.61	1971	San Fernando	Maricopa Array #2	14.3	22.4	23.0	30.8
75	6.61	1971	San Fernando	Maricopa Array #3	16.6	23.4	11.6	16.3
76	6.61	1971	San Fernando	Pacoima Dam (upper left abut)	5.8	7.3	15.2	21.9
77	6.61	1971	San Fernando	Palmdale Fire Station	10	18.9	4.5	6.6
78	6.61	1971	San Fernando	Pasadena - CIT Athenaeum	6.7	14.5	11.1	18.7
79	6.61	1971	San Fernando	Pasadena - Old Seismo Lab	5.7	14.1	9.0	15.2
80	6.61	1971	San Fernando	Pearblossom Pump	7.1	13.7	5.3	15.3
81	6.61	1971	San Fernando	Port Hueneme	38.5	49.2	12.3	21.5
82	6.61	1971	San Fernando	Puddingstone Dam (Abutment)	6.4	14.3	39.6	43.7
83	6.61	1971	San Fernando	San Diego Gas & Electric	15.3	24.2	12.4	24.2
84	6.61	1971	San Fernando	San Juan Capistrano	17.5	48.2	15.3	21.6
85	6.61	1971	San Fernando	San Onofre - So Cal Edison	15.8	35.5	21.8	55.4
86	6.61	1971	San Fernando	Santa Anita Dam	6.4	11.3	37.3	44.0
87	6.61	1971	San Fernando	Santa Felita Dam (Outlet)	7.4	23.6	12.4	21.9
88	6.61	1971	San Fernando	Tehachapi Pump	3.7	9.5	23.9	28.7
89	6.61	1971	San Fernando	UCSB - Fluid Mech Lab	28.5	49.3	4.0	8.1
90	6.61	1971	San Fernando	Upland - San Antonio Dam	7.1	14.3	42.4	47.8
91	6.61	1971	San Fernando	Wheeler Ridge - Ground	7.2	17.9	7.0	18.1
92	6.61	1971	San Fernando	Whittier Narrows Dam	7.2	21.5	16.7	25.9
93	6.61	1971	San Fernando	Wrightwood - 6074 Park Dr	7.3	11.7	10.5	26.3
94	6.24	1972	Managua_ Nicaragua-01	Managua_ ESSO	4.9	10.6	12.0	15.1
95	5.2	1972	Managua_ Nicaragua-02	Managua_ ESSO	2.8	8.1	6.3	11.0
96	5.65	1973	Point Mugu	Port Hueneme	6.1	13.8	2.4	6.3
97	5.14	1974	Hollister-03	Gilroy Array #1	1.2	2.7	3.5	10.9
98	5.14	1974	Hollister-03	Hollister City Hall	3.6	10.9	0.8	2.8
99	5.14	1974	Hollister-03	San Juan Bautista_ 24 Polk St	4.9	9.6	5.0	18.2
100	5.2	1975	Northern Calif-07	Cape Mendocino	4.3	5.7	6.4	13.7