Selection of ground motion for performing incremental dynamic analysis of existing reinforced concrete buildings in India

M. M. Maniyar[1,]* and R. K. Khare[2]

[1]Department of Structural Engineering, Sardar Patel College of Engineering, Mumbai 400 058, India
[2]Department of Civil Engineering, Shri Govindram Seksaria Institute of Technology and Science, Indore 452 003, India

In this article, a suite of 20 ground motion time histories has been selected from all available recorded Indian earthquake events based on a detailed statistical study performed on various ground motion parameters like peak ground acceleration, peak ground velocity, peak ground displacement, acceleration RMS, velocity RMS, displacement RMS, Arias intensity, characteristic intensity, spectral acceleration, acceleration spectrum intensity and significant duration. Statistical analysis has been performed by scaling the time histories to uniform values of various parameters considered, singly and in combination. Minimum, maximum and median values, standard deviations, and lognormal deviations have been calculated. The selected set of earthquakes in this article effectively captures the variability in response to the randomness of input motion. It represents different rates of energy input to the structures as well as different effective durations. At the same time inelastic response of single degree of freedom system shows less lognormal dispersion when scaled to median spectral acceleration for a narrow band of time-period surrounding the fundamental period, indicating converged response. Median spectral acceleration for period range significant to population of structures under consideration has been proposed as an efficient intensity measure.

Keywords: Earthquakes, ground motion, RC buildings, statistical analysis.

LOW-RISE, non-seismic, reinforced concrete (RC) frame buildings designed only for gravity loads constitute the major stock of existing buildings in India. Seismic performance assessment of such buildings has become significantly important in view of the observed lack of seismic resistance of such buildings during recent earthquakes. The state-of-the-art for evaluating seismic response of buildings has progressively moved from elastic static analysis to elastic dynamic, nonlinear static and finally nonlinear dynamic analysis. In the last case the convention has been to run one to several different records, each once, producing one to several 'single-point' analyses, mostly used for checking the designed structure. Recently, incremental dynamic analysis (IDA)[1] has emerged as a parametric analysis method to estimate more thoroughly structural performance under seismic loads.

Indian standard criteria for earthquake-resistant design of structures (IS 1893 Part I, 2002) mention that 'Time history method of analysis, when used, shall be based on an appropriate ground motion and shall be performed using accepted principles of dynamics'. Knowing the random nature of ground motion, the question arises whether one record is enough or more are required? Dynamic response is sensitive to ground-motion record applied for dynamic analysis. It is not possible to conclude a single time history as the future probable earthquake ground motion at a particular location applied to a particular structure for seismic analysis. More than one engineering ground-motion parameter significantly influences the response of structures simultaneously. The selected ground-motion parameters should be capable of capturing all intensity, frequency content, and duration information that significantly affects the elastic and inelastic response of structural systems. No single parameter is ideally suited for this purpose, and, unfortunately, the best choice of parameters depends, sometimes weakly and sometimes strongly, on the structural system and the performance level to be evaluated. This issue, which implies a search for efficient intensity measures (IM) leading to demand predictions with smaller dispersion, is one of the basic challenges of performance-based earthquake engineering.

The present work deals with selection of a set of ground motions based on its effect on the seismic demand evaluation of elastic and inelastic systems. The objective of the present study is to create a bin of Indian recorded time histories to be used in inelastic dynamic analysis of typical existing medium-rise, non-seismic, RC-frame buildings. The selected set of recorded ground motions with appropriate scaling in terms of IMs that are sufficient will make the seismic demand almost independent of the location, source mechanism, magnitude and distance for a specific class of structures. Presently, there is no established procedure to select such sets of ground motions.

*For correspondence. (e-mail: mmmaniyar@yahoo.com)

Available databases of strong ground-motion time histories in India

Recorded ground-motion time series contain valuable characteristics and information that can be used directly or indirectly in seismic analysis and design. It reflects realistic information of ground-motion parameters like amplitude, frequency, energy content, duration, phase characteristics, and also the characteristics of source, path and site. In India, the strong motion instrumentation programme began yielding results since it was started in the mid-sixties by the Department of Earthquake Engineering (DEQ), Indian Institute of Technology, Roorkee. The digital data of various earthquake events in India[2], having magnitude between 6.9 and 5.4, recorded by the instruments installed under the above-mentioned programme at DEQ are given in Table 1.

Criterion and process for primary selection of records and forming record bins

In the present work a set of 20 Indian time-history records has been selected, such that variability of different ranges of earthquake magnitude, source-to-site distance, intensity, frequency content and duration applicable for period ranges of existing non-seismic RC-frame buildings can be studied. We now summarize the different criteria for forming the record bins.

Definition of magnitude–distance (M–R) bins

Magnitude and distance range for the bins should be selected such that they define systematic differences in response to spectral shapes between the bins. The differences in M and R for each bin should be large enough to have an adequate record population to choose from, but small enough so that ‘unreasonable’ amounts of scaling of records to the design spectrum would not be required. The farthest distance for near-source $M–R$ bins should be large enough to capture records potentially having significant near-source ground-motion characteristics. The farthest $M–R$ bins may be selected on the basis of ground-motion amplitudes.

For the now widely used IDA, Vamvatsikos and Cornell[1] used a suite of 20 time histories from the large magnitude–short distance bin (LMSR, $6.1 < Mw < 6.9$, 15 km < R < 33 km), assuming that the same ground motions can be used to evaluate seismic demands over a wide range of hazard levels. The same LMSR bin of time histories has been used by Dhakal *et al.*[3] in the identification of critical ground motions and by Mander *et al.*[4], for seismic risk assessment of bridges. Chopra and Chintanpakdee[5] organized 232 ground motions into 13 ensembles of ground motions, representing large or small earthquake magnitudes and distances. Ibarra and Krawinkler[6] organized 80 ground motions into four bins representing large or small earthquake magnitudes to determine global collapse of frame structures under seismic excitations.

Magnitude considerations: The recorded earthquake events (132 records) were divided into two categories, viz. large magnitude and small magnitude, similar to the work done by others[1,3–6]. The events having moment magnitude from 6.1 to 6.9 ($6.1 < Mw \le 6.9$) are considered as large-magnitude events, and those having moment magnitude from 5.4 to 6.1 ($5.4 < Mw \le 6.1$) are considered as small-magnitude events.

Distance considerations: Based on source-to-site distance of the records, a primary division of small radius (12 km < $R \le$ 40 km) and large radius (40 km < $R \le$ 90 km) was made, similar to earlier studies[1,3–6]. A distinction was made, however, between near-fault ground motions and those recorded more than about 12 km from the fault rupture zone. The stretching of large radius to a high value of 90 km was done to include more number of available earthquake records in that category. Amplitude parameters were checked while selecting the records.

Number of records in the set

Shome and Cornell[7] have shown that for mid-rise buildings, 10–20 records are usually enough to provide sufficient

Table 1. Available Indian strong motion records

Earthquake event	Date	Mw	Number of accelerograph recording stations
Dharmsala Earthquake	26 April 1986	5.5	09
India–Burma Border Earthquake	18 May 1987	5.7	14
India–Bangladesh Border Earthquake	6 February 1988	5.8	18
India–Burma Border Earthquake	6 August 1988	6.8	33
India–Burma Border Earthquake	10 January 1990	6.1	14
Uttarkashi Earthquake	20 October 1991	6.5	13
India–Burma Border Earthquake	6 May 1995	6.4	09
India–Burma Border Earthquake	8 May 1997	5.5	11
Chamoli Earthquake	29 March 1999	6.4	11

accuracy in the estimation of seismic demands, assuming a relatively efficient IM, like spectral acceleration, *Sa* (*Tn*; 5%). They have also shown that to obtain an estimate of the median (geometric mean, defined as the exponential of the arithmetic mean of the log demand estimates) response within a factor of $\pm X$ (e.g. ± 0.1) with the accuracy of 95% confidence, the number of records required for the dynamic analysis is given by

$$n = \frac{4.0\beta^2}{X^2}, \qquad (1)$$

where β is the dispersion from the median.

The target number of records in each primary bin is kept as 10 or less, as scanty data are available. Only two records are available for large-magnitude near-fault category. Similarly, only 10 records are available for large-magnitude, small-radius category in the available database.

Development of primary record sets

In the present study, in all, 42 time histories have been selected primarily from available time histories. Records having very low amplitude and duration were not considered. The selected records were classified into five magnitude–distance bins for performing rigorous statistical evaluation based on the variation of pseudo-spectral acceleration (*Sa*), at $T = 0.8$ s per unit peak ground acceleration (PGA), within the five bins. The record bins are designated as follows:

- Large magnitude–near fault bin (LMNF; $6.1 < Mw \leq 6.9$, 12 km $\leq R$);
- Large magnitude–short distance bin (LMSR; $6.1 < Mw \leq 6.9$, 12 km $< R \leq 40$ km);
- Large magnitude–long distance bin (LMLR; $6.0 < Mw \leq 6.5$, 40 km $< R \leq 90$ km);
- Small magnitude–short distance bin (SMSR; $5.4 < Mw \leq 6.0$, 12 km $< R \leq 40$ km); and
- Small magnitude–long distance bin (SMLR; $5.4 < Mw \leq 6.0$, 40 km $< R \leq 90$ km).

Table 2 displays the time histories selected for all the bins.

Selection time histories

Twenty ground motions are to be selected such that variability of ground that significantly affects the elastic and inelastic response of the existing RC buildings under consideration should be effectively be captured. At the same time, the selected set should give a highly converging predictable response when scaled to effective IM. Most of the methods proposed for selection of recorded earthquake ground motions for time-history analysis focus on matching the response spectra of selected ground motions with the given design spectra. However, the seismic design spectra in the seismic codes represent an average value from statistics. The methodology adopted to choose suitable earthquake recordings to be used in nonlinear dynamic analyses consists of systematic investigations of the elastic and inelastic response of single-degree-of-freedom (SDOF) systems subjected to different types of earthquake recordings, and different earthquake characteristics.

Spectral accelerations as the primary IM

Ground-motion characteristics that are used in developing criteria for time-history record selection include response spectral shape over a period range of structural significance. Shome and Cornell[7] found that seismic demand estimates are strongly correlated with the linear–elastic spectral response acceleration at the structure fundamental period, *Tn*, also called the spectral intensity *Sa* (*Tn*). To select 20 records from all available magnitude–distance categories, it is assumed that the pseudo-spectral acceleration (*Sa*), at $T = 0.8$ s/unit PGA, is the primary ground-motion parameter and it is evaluated for all records. Records showing *Sa* ($Tn = 0.8$ s)/PGA < 0.3 are omitted as the frequency content of these is not significant for fundamental period of the buildings under consideration, which constitute the major stock of existing low-rise, non-seismic RC-frame buildings in India designed only for gravity loads.

Inelastic displacement as primary engineering demand parameter

Inelastic, constant-ductility (=3) displacement response is computed by means of time-integration of the equation of motion of a SDOF system having fundamental period of 0.8 s. Figure 1 represents actual nonlinear structural response by means of an elasto-plastic representation of the system subjected to LMLR1 ground motion. In this way, energy dissipated through hysteresis is explicitly modelled, with only a relatively small viscous damping quantity (5%) added to the system, to somehow represent non-hysteretic energy dissipation mechanisms.

Selected time histories

A final set of 20 time histories was selected, which simultaneously satisfies the largest lognormal dispersion in spectral acceleration scaled/unit PGA and least lognormal dispersion in inelastic displacement when records are scaled to median *Sa* for *Tn* in the range 0.6–1.0 s. In this

Table 2. Selected time histories for all the bins

Record Id	Earthquake event	Magnitude (Mw)	Station	R (km)	PGA (g)	Component
LMNF						
1 LMNF	Chamoli 1999	6.4	Gopeshwar	8.7	0.199	Longitudinal
2 LMNF	Chamoli 1999	6.4	Gopeshwar	8.7	0.359	Transverse
LMSR						
1 LMSR	Uttarkashi 1991	6.5	Bhatwari	19.3	0.253	Longitudinal
2 LMSR	Uttarkashi 1991	6.5	Bhatwari	19.3	0.247	Transverse
3 LMSR	Uttarkashi 1991	6.5	Ghansiali	38.0	0.118	Longitudinal
4 LMSR	Uttarkashi 1991	6.5	Ghansiali	38.0	0.117	Transverse
5 LMSR	Uttarkashi 1991	6.5	Uttarkashi	32.5	0.242	Longitudinal
6 LMSR	Uttarkashi 1991	6.5	Uttarkashi	32.5	0.309	Transverse
7 LMSR	Chamoli 1999	6.4	Joshimath	32.2	0.091	Longitudinal
8 LMSR	Chamoli 1999	6.4	Joshimath	21.1	0.063	Transverse
9 LMSR	Chamoli 1999	6.4	Ukhimath	32.2	0.091	Longitudinal
10 LMSR	Chamoli 1999	6.4	Ukhimath	32.2	0.097	Transverse
LMLR						
1 LMLR	Uttarkashi 1991	6.5	Barkot	54.87	0.095	Longitudinal
2 LMLR	Uttarkashi 1991	6.5	Karnprayag	68.89	0.079	Transverse
3 LMLR	Uttarkashi 1991	6.5	Purola	69.33	0.093	Transverse
4 LMLR	Uttarkashi 1991	6.5	Koteshwar	60.5	0.101	Longitudinal
5 LMLR	Uttarkashi 1991	6.5	Purola	69.33	0.075	Longitudinal
6 LMLR	Uttarkashi 1991	6.5	Srinagar	57.85	0.067	Longitudinal
7 LMLR	Uttarkashi 1991	6.5	Tehri	49.63	0.073	Longitudinal
8 LMLR	Chamoli 1999	6.4	Ghansiali	73.84	0.083	Transverse
9 LMLR	Chamoli 1999	6.4	Ghansiali	73.84	0.073	Longitudinal
10 LMLR	Chamoli 1999	6.4	Tehri	88.35	0.062	Transverse
SMSR						
1 SMSR	India–Burma Border 1997	5.7	Jellalpur	24.45	0.117	Longitudinal
2 SMSR	India–Burma Border 1997	5.7	Jellalpur	24.45	0.138	Transverse
3 SMSR	India–Burma Border 1997	5.7	Katakhal	39.91	0.107	Longitudinal
4 SMSR	India–Burma Border 1997	5.7	Katakhal	39.91	0.162	Transverse
5 SMSR	North East India 1986	5.2	Dauki	27.16	0.090	Transverse
6 SMSR	North East India 1986	5.2	Pynursla	21.67	0.093	Longitudinal
7 SMSR	North East India 1986	5.2	Umsning	39.39	0.101	Longitudinal
8 SMSR	North East India 1986	5.2	Umsning	39.39	0.076	Transverse
9 SMSR	Dharmsala 1986	5.5	Bandlakhas	24.18	0.124	Transverse
10 SMSR	Dharmsala 1986	5.5	Nagrotabagwan	12.29	0.149	Longitudinal
SMLR						
1 SMLR	India–Burma Border 1997	5.7	Silchar	55.89	0.095	Longitudinal
2 SMLR	India–Burma Border 1997	5.7	Silchar	55.89	0.152	Transverse
3 SMLR	India–Burma Border 1997	5.7	Ummulong	70.63	0.155	Longitudinal
4 SMLR	India–Burma Border 1997	5.7	Ummulong	70.63	0.101	Transverse
5 SMLR	India–Burma Border 1997	5.7	Shillong	83.41	0.072	Longitudinal
6 SMLR	India–Bangladesh Border 1988	5.8	Pynursla	83.25	0.049	Longitudinal
7 SMLR	India–Burma Border 1987	5.7	Laisong	89.76	0.061	Transverse
8 SMLR	North East India Earthquake 1986	5.2	Nongkhlaw	52.76	0.093	Transverse
9 SMLR	North East India Earthquake 1986	5.2	Saitsama	44.79	0.113	Longitudinal
10 SMLR	North East India Earthquake 1986	5.2	Saitsama	44.79	0.139	Transverse

R, Closest distances to fault rupture; PGA, Peak ground acceleration.

article dispersion refers to the standard deviation of the natural logarithm of the values. The median of Sa values is estimated by computing the geometric mean of the data:

$$\breve{S}a = \exp(\overline{\ln Sa}), \tag{2}$$

where $\overline{\ln Sa}$ is the mean of natural logarithm of Sa data.

The standard deviation of the natural log of Sa values is computed by:

$$\beta_{Sa|\mathrm{PGA}} = \sigma_{\ln Sa} = \sqrt{\sum_{i=1}^{n} \frac{(\ln Sa_i - \overline{\ln Sa})^2}{n-1}}. \tag{3}$$

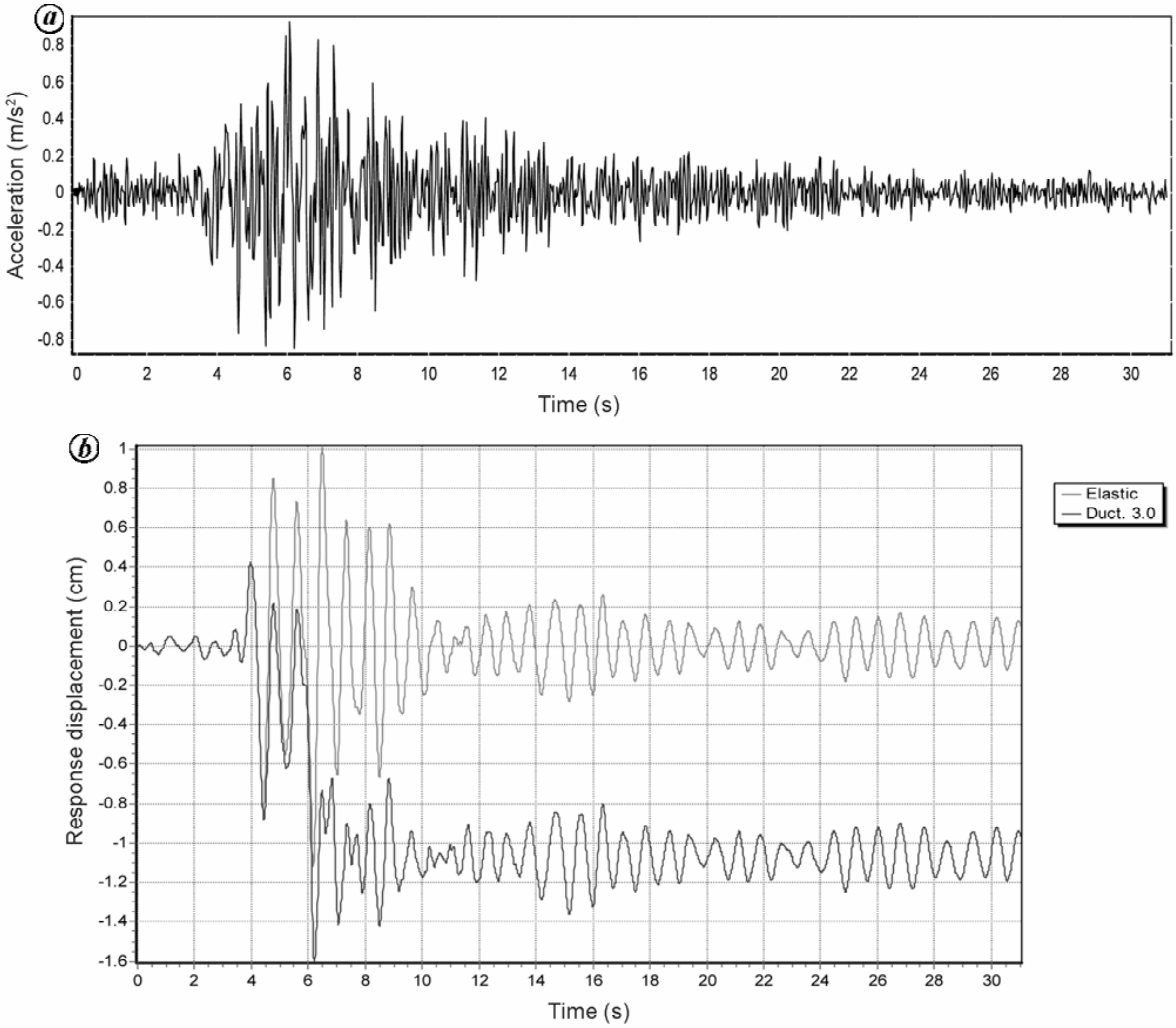

Figure 1. LMLR1 time history (***a***) and inelastic displacement response (***b***).

When the set of time histories shown in Table 3 is scaled to unit PGA, a wide range of spectral acceleration, i.e. min = 0.396 m/s^2 to max = 1.86 m/s^2, is covered. At the same time when scaled to median *Sa* ($0.6 \leq Tn \leq 1.0$), inelastic response of SDOF system shows less lognormal dispersion of 0.29.

Scaling of records

According to the empirical findings by Sewell[8], the linear response of SDOF system is independent of *M* and *R*. Shome and Cornell[7] have later shown that scaling is not a bias for the multi-degree-of-freedom (MDOF) systems and proper scaling can reduce dispersion of response to one-fourth that of actual dispersion. All selected ground motions have been uniformly scaled to 1 m/s^2 (unit) PGA and applied to the linear and nonlinear SDOF systems having natural period ranging from 0.6 to 1.0 s to evaluate various ground-motion parameters. A detailed statistical study is presented by varying the scaling parameter.

Ground-motion parameters

Since the first strong ground motion was recorded in 1933, a large number of strong ground motions have been recorded in the world. On the basis of these ground motions, researchers have proposed different parameters to characterize the ground-motion damage potential. These parameters range from a simple instrumental peak value to that resulting from a complicated mathematical derivation. Ground-motion parameters are essential for describing the important characteristics of strong ground motion in compact, quantitative form. Many parameters have been proposed to characterize the amplitude,

Table 3. Selected ground motions (20) for performing dynamic analysis

Record Id	Earthquake event	Station	Component	Sa, 5% T = 0.8 s (m/s^2)	U_{max}* (cm)
3 LMSR	Uttarkashi 1991	Ghansiali	Longitudinal	0.915	1.50
4 LMSR	Uttarkashi 1991	Ghansiali	Transverse	1.061	1.28
5 LMSR	Uttarkashi 1991	Uttarkashi	Longitudinal	1.223	1.61
6 LMSR	Uttarkashi 1991	Uttarkashi	Transverse	1.152	1.39
7 LMSR	Chamoli 1999	Joshimath	Longitudinal	0.679	2.16
9 LMSR	Chamoli 1999	Ukhimath	Longitudinal	0.627	1.70
10 LMSR	Chamoli 1999	Ukhimath	Transverse	1.055	0.84
1 LMLR	Uttarkashi 1991	Barkot	Longitudinal	0.741	2.11
4 LMLR	Uttarkashi 1991	Koteshwar	Longitudinal	0.599	1.71
5 LMLR	Uttarkashi 1991	Purola	Longitudinal	0.475	1.42
6 LMLR	Uttarkashi 1991	Srinagar	Longitudinal	0.396	1.96
7 LMLR	Uttarkashi 1991	Tehri	Longitudinal	0.752	2.55
8 LMLR	Chamoli 1999	Ghansiali	Transverse	0.686	1.35
9 LMLR	Chamoli 1999	Ghansiali	Longitudinal	0.684	2.25
10 LMLR	Chamoli 1999	Tehri	Transverse	1.857	1.31
1 SMSR	India–Burma Border 1997	Jellalpur	Longitudinal	0.934	1.17
2 SMSR	India–Burma Border 1997	Jellalpur	Transverse	1.020	0.88
3 SMSR	India–Burma Border 1997	Katakhal	Longitudinal	1.357	1.78
1 SMLR	India–Burma Border 1997	Silchar	Longitudinal	1.076	1.08
5 SMLR	India–Burma Border 1997	Shillong	Longitudinal	1.162	1.69
Median				0.925	1.55
Dispersion				0.374	0.29

U_{max}*, Maximum inelastic displacement response when scaled to median Sa, 5%, 0.6 s < T < 1.0 s.

frequency content and duration of strong ground motions; some describe only one of these characteristics, while others may reflect two or three. Because of the complexity of earthquake ground motions, identification of a single parameter that accurately describes all important ground-motion characteristics is not possible. Various significant engineering parameters have been evaluated and detailed statistical study has been carried out on parameters calculated for the 20 time histories selected. Suitability of various ground-motion parameters as IM has been discussed and conclusions have been drawn. Modified parameters have also been defined.

Response spectrum

Elastic response spectral shape over a period range of significance to structural response has been found to be closely correlated to inelastic structural response and behaviour[9]. The response spectrum describes the maximum response of a SDOF system to a particular input motion as a function of the natural period and damping ratio of the SDOF system. The acceleration time histories are scaled with respect to PGA to arrive at uniform PGA. The response spectra for the selected earthquake ground motions scaled to the same IM, that is, PGA = 1.0 m/s^2, are constructed.

Significant range of period for response spectrum: The median fundamental period of representative sample buildings for the population of buildings under consideration is 0.8 s. The period range may include those shorter than the fundamental structure period because of higher mode effects, as well as longer than the fundamental structure period because of structure softening to longer periods in the inelastic range. Analytically calculated fundamental period varies in reality within a small range. Therefore, the spectral shape of the records over the defined period of 0.6–1.0 s range has been considered for comparison to the median and variation of spectral shapes for all the records. Figure 2 shows the response spectra for spectral acceleration along with median spectra for period range significant for the buildings under study, when time histories are scaled to unit PGA.

Which spectral acceleration is to be used as IM?: Variation of spectral acceleration/unit PGA is displayed in Figure 2. Table 4 shows the lognormal standard deviation in the spectral accelerations for Tn ranging from 0.6 s to 1.0 s. The maximum dispersion was 0.495, which is fairly high. Thus it represents the efficiency of the selected ground motions in representing variability of ground motion. In search of better IMs all the spectral accelerations have been scaled to Sa, Tn = 0.8 s. Figure 3 shows the dispersion of spectral accelerations when scaled to Sa, Tn = 0.8 s. This scaling further reduces the dispersion in Sa. Table 4 shows the lognormal dispersion in Sa at different time periods when scaled to Sa, Tn = 0.8 s. It is important to note here that Sa varies for different fundamental periods for a given time history, and at the same time it also varies for different time histories at a given fundamental period. Median Sa for significant period range is calculated for each ground motion. The scaling parameter selected as this median *Sa* for the corresponding ground motion further reduces variation from ground motion to ground motion. Figure 4 displays the variation of *Sa* when time histories are scaled to corresponding median *Sa* for significant periods ($0.6 \text{ s} \leq Tn \leq 1.0$ s). It can be clearly observed that the median *Sa* for *Tn* ranging from 0.6 to 1.0 s for each ground motion is the most efficient IM. Table 4 confirms this fact.

Table 4. Variation of spectral acceleration for various periods considered within sub-range

Sa when scaled to unit PGA					
Period (s)	0.60	0.70	0.80	0.90	1.00
Dispersion	0.449	0.495	0.374	0.284	0.368
Variation of *Sa* when scaled to *Sa*, $Tn = 0.8$ s					
Period (s)	0.60	0.70	0.80	0.90	1.00
Dispersion	0.327	0.258	0.000	0.277	0.431
Variation of *Sa* when scaled to *Sa*, median, $0.6 \text{ s} \leq Tn \leq 1.0$ s					
Period (s)	0.60	0.70	0.80	0.90	1.00
Dispersion	0.273	0.215	0.104	0.208	0.367

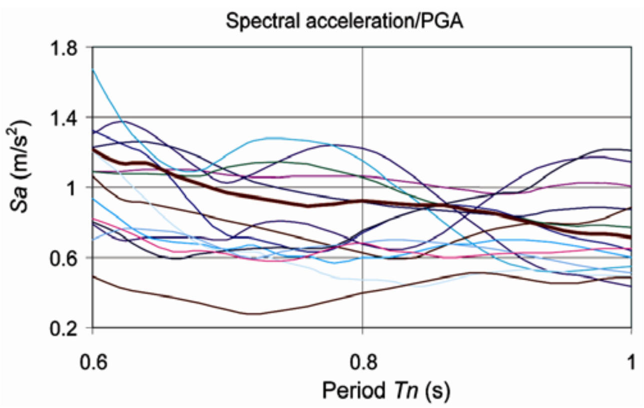

Figure 2. Variation of spectral acceleration (*Sa*) when scaled to unit peak ground acceleration (PGA) for individual ground motions (thick line shows median values).

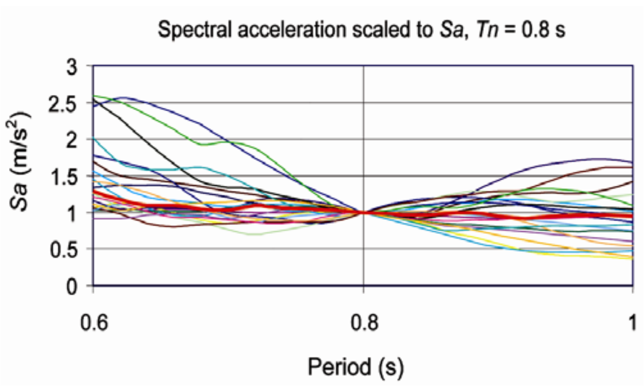

Figure 3. Variation of *Sa* when scaled to *Sa*, $Tn = 0.8$ s for individual ground motions (thick line shows median values).

Response spectrum intensity

The intensity of shaking of an earthquake at a given site is represented by the velocity spectrum intensity (VSI)[10], defined as the area under the elastic velocity spectrum, between the periods 0.1 and 2.5 s. A mixed representative population of structures has fundamental periods between 0.1 and 2.5 s. The VSI is therefore defined as

$$\mathrm{VSI}(\xi) = \int_{0.1}^{2.5} Sv(\xi, T)\mathrm{d}T. \tag{4}$$

The significant fundamental period range (*Tn*) for the present study is 0.6–1.0 s. Hence the definition of VSI is modified as

$$\text{Modified VSI}(\xi) = \int_{0.6}^{1.0} Sv(\xi, T)\mathrm{d}T. \tag{5}$$

Table 5 shows the variation of various spectrum intensities as defined above. It is interesting to note that the dispersion in modified acceleration spectrum intensity (MASI) reduces to a very low value of 0.101, when time histories are scaled to median *Sa* for that time history.

To characterize strong ground motions for analysis of structures having fundamental period less than 0.5 s, Von Thun and Rochim[11] introduced the ASI, defined as

$$\mathrm{ASI}(\xi) = \int_{0.1}^{0.5} Sa(\xi, T)\mathrm{d}T. \tag{6}$$

The significant fundamental period range (*Tn*) for the present study is 0.6–1.0 s as mentioned earlier. Hence the definition of ASI is modified as

$$\text{Modified ASI}(\xi) = \int_{0.6}^{1.0} Sa(\xi, T)\mathrm{d}T. \tag{7}$$

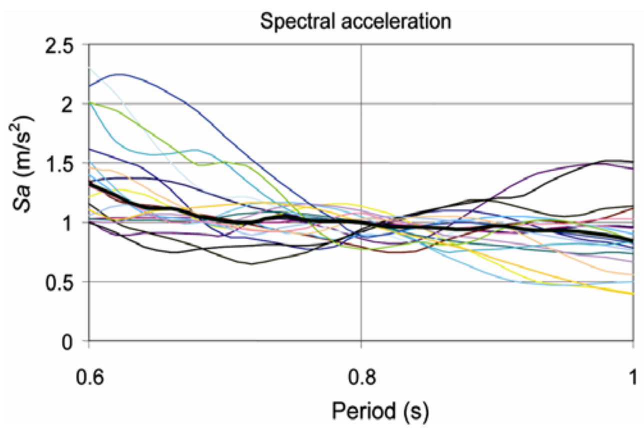

Figure 4. Variation of *Sa* when scaled to *Sa*, median, $0.6 \text{ s} \leq Tn \leq 1.0$ s for individual ground motions (thick line shows median values).

Table 5 displays the variation of response spectrum intensities when scaled to unit PGA.

Amplitude parameters

The most common way of describing a ground motion is with a time history. The motion parameter may be acceleration, velocity or displacement, or all three may be displayed. Typically, only one of these quantities is measured directly, with the others computed from it by integration and/or differentiation.

Peak ground acceleration: This is the most commonly used measure of the intensity of shaking at a site and is taken to be the largest absolute value of the horizontal acceleration recorded at a site. PGA for a given component of motion is simply the largest (absolute) value of horizontal acceleration obtained from the accelerogram of that component.

$$\mathrm{PGA} = \max|a(t)|. \tag{8}$$

The variation of recorded PGA for the time histories selected is shown in Table 6. The range of PGA recorded is agreeable with that expected on moderate seismic zone, thus avoiding unreasonable scaling.

Ground motions with high peak accelerations are usually, but not always, more damaging than those with lower peak acceleration. The duration of the peak excitation is also an important consideration in estimating the damage potential of a ground motion. Very high peak accelerations that last for only a very short period of time may cause little damage to many types of structures. A number of earthquakes have produced peak accelerations in excess of 0.5 *g*, but have caused no significant damage to structures because the peak accelerations occurred at very high frequencies and the duration of the earthquakes was not long. Although peak acceleration is a useful parameter, it provides no information on the frequency content or duration of motion; consequently, it must be supplemented with additional information to characterize a ground motion accurately.

Table 5. Variation of response spectrum intensities when scaled to unit PGA

Spectral intensity	Minimum	Maximum	Median	Dispersion
ASI	0.404	1.192	0.932	0.220
VSI	15.378	46.775	24.992	0.249
MASI	0.175	0.689	0.393	0.355
MVSI	2.255	8.473	4.818	0.343
MASI when scaled to median *Sa*	0.374	0.551	0.420	0.101

ASI, Acceleration spectrum intensity; VSI, Velocity spectrum intensity; MASI, Modified ASI; MVSI, Modified VSI.

Table 6. Variation of recorded PGA, PGV, PGD, PGV/PGA and PGD/PGA of the bins of ground motions

Parameter	Minimum	Maximum	Median	Dispersion
PGA (g)	0.067	0.067	0.095	0.402
PGV (cm/s)	3.475	18.909	6.997	0.469
PGD (cm)	3.667	87.655	21.679	0.874
PGV/unit PGA (cm/s)	5.155	10.826	6.936	0.214
PGD/unit PGA (cm)	3.195	108.371	27.251	0.986

PGV, Peak ground velocity; PGD, Peak ground displacement.

Peak velocity: The peak horizontal velocity (PGV) is another useful parameter for characterization of ground-motion amplitude. Since the velocity is less sensitive to the higher frequency components of the ground motion, PGV is more likely than PGA to characterize ground-motion amplitude accurately at intermediate frequencies. For structures or facilities that are sensitive to loading in this intermediate frequency range (e.g. tall or flexible buildings, bridges, etc.), PGV may provide a more accurate indication of the potential for damage than PGA.

$$\mathrm{PGV} = \max|v(t)|. \tag{9}$$

Peak displacement (PGD): This is generally associated with the lower frequency components of an earthquake motion. It is, however, often difficult to determine accurately, due to signal processing errors in the filtering and integration of accelerograms and due to long-period noise. As a result, peak displacement is less commonly used as a measure of ground motion than is peak acceleration or peak velocity.

$$\mathrm{PGD} = \max|d(t)|. \tag{10}$$

In the present study PGV/PGA and PGD/PGA have been calculated for all time histories scaled to 1.0 m/s^2 (unit) PGA, as displayed in Table 6.

Other amplitude parameters

Various parameters describe only the peak amplitudes of a single cycle within the ground-motion time history. In some cases, damage may be closely related to the peak amplitude, but in others it may require several repeated cycles of high amplitude to develop. Newmark and Hall[12] described the concept of an effective acceleration as 'that acceleration which is most closely related to structural response and to damage potential of an earthquake. It differs from and is less than the peak free field ground acceleration. It is a function of the size of the loaded area, the frequency content of the excitation, which in turn depends on the closeness to the source of the earthquake, and to the weight, embedment, damping characteristic, and stiffness of the structure and its foundation'.

Sustained maximum acceleration and velocity: Nuttli[13] used lower peaks of the accelerogram to characterize strong motion by defining the sustained maximum acceleration (SMA) for three (or five) cycles as the third (or fifth) highest (absolute) value of acceleration in the time history. The sustained maximum velocity (SMV) was defined similarly.

In the present study SMA/PGA and SMV/PGA have been calculated for all time histories scaled to 1.0 m/s^2 (unit) PGA as displayed in Table 7. The dispersion in SMA is indicative of small-duration peaks in PGA.

Effective design acceleration: The notion of effective design acceleration (EDA), with different definitions, has been proposed by at least two researchers. Since pulses of high acceleration at high frequencies induce little response in most structures, Benjamin *et al.*[14] proposed that an effective design acceleration be taken as the peak acceleration that remains after filtering out accelerations above 8–9 Hz. Kennedy[15] proposed that the effective design acceleration be 25% greater than the third highest (absolute) peak acceleration obtained from a filtered time history.

A95 parameter[16]: This represents the acceleration level below which 95% of the total Arias intensity (*Ia*) is contained. In other words, if the entire accelerogram yields a value of $Ia = 100$, the A95 parameter is the threshold of acceleration such that integrating all the values of the accelerogram below, one gets an $Ia = 95$. Table 8 displays the variation of EDA and A95 parameters. The variation of EDA/PGA and A95/PGA is less. These parameters may be applied in combination, along with other major scaling parameters, to reduce the variation of response to some extent.

Root mean square acceleration (RMSa): A single parameter that includes the effect of amplitude and frequency content of a strong motion record is the RMS (root mean square) acceleration, defined as

$$a_{\mathrm{rms}} = \sqrt{\frac{1}{t_{\mathrm{d}}}\int_0^{t_{\mathrm{d}}} [a(t)]^2\,\mathrm{d}t} \qquad (11)$$

where t_{d} is the duration of strong motion and $a(t)$ the acceleration intensity.

Root mean square velocity (RMSv): This can be defined similar to eq. (11) as

$$v_{\mathrm{rms}} = \sqrt{\frac{1}{t_{\mathrm{d}}}\int_0^{t_{\mathrm{d}}} [v(t)]^2\,\mathrm{d}t}. \qquad (12)$$

Root mean square displacement (RMSd): This can also be defined similar to eq. (11) as

$$d_{\mathrm{rms}} = \sqrt{\frac{1}{t_{\mathrm{d}}}\int_0^{t_{\mathrm{d}}} [d(t)]^2\,\mathrm{d}t}. \qquad (13)$$

In the present study RMSa/PGA, RMSv/PGA and RMSd/PGA have been calculated for all time histories scaled to 1.0 m/s^2 (unit) PGA, as displayed in Table 9. It is clear from Table 9 that the dispersion for RMSv and RMSd increases compared PGV and PGD. For PGA-based scaling this is not an effective parameter for the dataset to converge the response.

Arias intensity: A parameter closely related to the RMSa is the Arias intensity[17] which is defined as

$$I_{\mathrm{a}} = \frac{\pi}{2g}\int_0^{t_{\mathrm{d}}} [a(t)]^2\,\mathrm{d}t. \qquad (14)$$

The Arias intensity has units of velocity and is usually expressed in metres per second. Here, it is obtained by integration over the duration of strong motion. Figure 5 displays the build-up of Arias intensity over significant duration for the time histories selected.

Specific energy density (SED): This is defined as

$$\mathrm{SED} = \int_0^{t_{\mathrm{r}}} [v(t)]^2\,\mathrm{d}t, \qquad (15)$$

Table 7. Variation of SMA/PGA and SMV/PGA of ground motions

Parameter	Minimum	Maximum	Median	Dispersion
SMA/unit PGA (cm^2/s)	0.472	0.967	0.752	0.209
SMV/unit PGA (cm/s)	3.667	7.671	5.197	0.187

SMA, Sustained maximum acceleration; SMV, Sustained maximum velocity.

Table 8. Variation of EDA/PGA and A95/PGA of ground motions

Parameter	Minimum	Maximum	Median	Dispersion
EDA	0.585	1.071	0.997	0.133
A95	0.967	0.988	0.982	0.008

EDA, Effective design acceleration.

Table 9. Variation of RMS acceleration (RMSa), velocity (RMSv) and displacement (RMSd)

Parameter	Minimum	Maximum	Median	Dispersion
RMSa	0.128	0.227	0.170	0.174
RMSv	0.838	3.431	1.891	0.378
RMSd	1.381	55.581	12.414	1.009

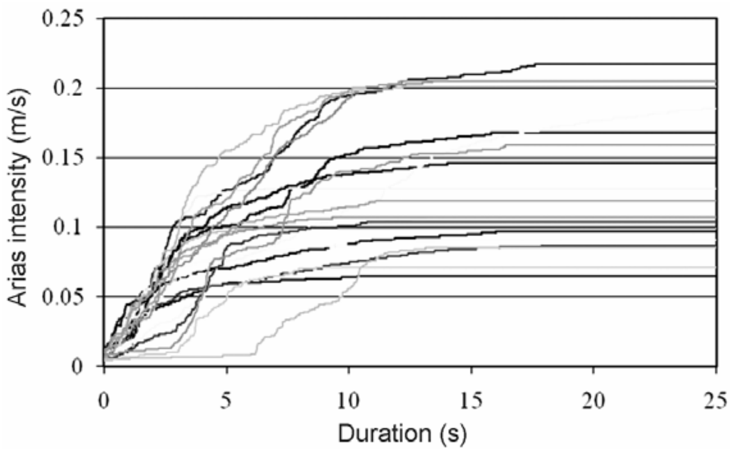

Figure 5. Build-up of Arias intensity for individual ground motions.

Table 10. Variation of Arias intensity (AI), specific energy density (SED) and characteristic intensity (CI)

Parameter	Minimum	Maximum	Median	Dispersion
AI	0.065	0.217	0.113	0.375
SED	28.009	419.972	105.605	0.773
CI	0.238	0.552	0.359	0.255

and has units of m^2/s. Here it is obtained by integrating velocity square over effective duration of an earthquake. This parameter captures the variation in kinetic energy input to the structure during significant duration of the earthquake.

Characteristic intensity (CI): This is defined as

$$I_c = (a_{rms})^{3/2} \cdot \sqrt{t_d}. \tag{16}$$

It is related linearly to an index of structural damage due to maximum deformation and absorbed hysteric energy. I_a/PGA, SED/PGA and CI/PGA are calculated for all time histories scaled to 1.0 m/s^2 (unit) PGA, as displayed in Table 10.

Frequency content parameters

The dynamic response of structures is sensitive to the frequency at which they are loaded. Earthquakes produce complicated loading with components of motion occurring at a broad range of frequencies. The frequency content describes how the amplitude of a ground motion is distributed among different frequencies. Since the frequency content of an earthquake motion will strongly influence the effects of that motion, characterization of the motion cannot be complete without consideration of its frequency content.

Fourier spectra: Any periodic function (i.e. any function that repeats itself exactly at a constant interval) can be expressed using Fourier analysis as the sum of a series of simple harmonic terms of different frequency, amplitude, and phase. Using Fourier series a periodic function, $x(t)$, can be written as

$$x(t) = c_0 + \sum_{n=1}^{\infty} c_n \sin(\omega_n t + \phi_n). \tag{17}$$

The Fourier series provides a complete description of the ground motion, since the motion can be fully recovered by the inverse Fourier function.

A plot of Fourier amplitude versus frequency is known as a Fourier amplitude spectrum. The Fourier amplitude spectrum of a strong motion shows how the amplitude of motion is distributed with respect to frequency (or period). A narrow band spectrum implies that the motion has a dominant frequency which can produce a smooth, almost sinusoidal time history. A broad spectrum corresponds to a motion that contains a variety of frequencies that produce a jagged, irregular time history.

Power spectrum: This is an alternative representation of the frequency content of a time history. It is closely related to the Fourier amplitude spectrum of the records as

$$s(\omega) = \frac{1}{2\pi T} \,|\, X(\omega) \,|^2, \tag{18}$$

where $s(\omega)$ denotes the power spectrum, $|X(\omega)|$ the Fourier amplitude spectrum, and T the duration of the record.

Duration

The duration of strong ground motion can have a marked influence on earthquake damage. The degradation of

Table 11. Parameters of the ground motion unaffected by scaling

Record id	V_{max}/A_{max} (s)	Peak Fourier amplitude	Peak power amplitude	Bracketed duration (s)	Effective duration (s)
3 LMSR	8.10	0.314	0.028	42.34	16.32
4 LMSR	6.18	0.213	0.010	42.34	17.72
5 LMSR	7.96	0.308	0.036	39.92	6.88
6 LMSR	6.02	0.356	0.062	39.92	6.88
7 LMSR	6.01	0.685	0.199	24.78	16.28
9 LMSR	9.30	0.234	0.027	24.78	16.28
10 LMSR	7.25	0.244	0.033	24.80	18.08
1 LMLR	7.95	0.223	0.017	31.74	15.14
4 LMLR	6.18	0.277	0.018	33.70	14.18
5 LMLR	5.64	0.220	0.026	35.70	13.60
6 LMLR	5.29	0.203	0.011	41.10	25.40
7 LMLR	6.51	0.500	0.113	31.96	14.16
8 LMLR	6.13	0.216	0.011	26.18	12.38
9 LMLR	5.16	0.240	0.014	26.32	12.48
10 LMLR	10.08	0.394	0.070	23.78	9.96
1 SMSR	6.99	0.313	0.066	16.34	10.08
2 SMSR	7.73	0.653	0.198	16.10	11.00
3 SMSR	10.85	0.639	0.124	25.50	16.48
1 SMLR	9.43	0.586	0.139	25.86	11.22
5 SMLR	6.89	0.295	0.050	24.34	13.32
Minimum	5.16	0.20	0.01	16.10	6.88
Maximum	10.85	0.68	0.20	42.34	25.40
Median	6.94	0.30	0.03	26.25	13.88
Dispersion	0.21	0.41	0.98	0.29	0.32

stiffness and strength of certain types of structures are sensitive to the number of load or stress reversals that occur during an earthquake. A motion of short duration may not produce enough load reversals for damaging response to build-up in a structure, even if the amplitude of motion is high. On the other hand, a motion with moderate amplitude but long duration can produce enough load reversals to cause substantial damage.

The duration of a strong ground motion is related to the time required for the release of accumulated strain energy by rupture along the fault. As the length, or area, of the fault rupture increases, the time required for rupture also increases. As a result, the duration of strong motion increases with increasing earthquake magnitude. The duration of strong motion has been studied by interpretation of accelerograms from earthquakes of different magnitudes.

Bracketed duration: This is the total time elapsed between the first and the last excursions of a specified level of acceleration (default is 5% of PGA). Bracketed duration of selected time histories ranges between 16 and 42 s.

Significant duration: This is defined as the interval of time over which a proportion (interval between the 5% and 95% thresholds) of the total Arias Intensity is accumulated. Median significant duration is 13.9 s.

Table 11 displays PGV/PGA, peak Fourier amplitude, peak power amplitude, bracketed duration and significant duration for all the selected ground motions which are independent of scaling.

Results and discussion

All selected time histories were scaled to unit PGA and various ground motion parameters evaluated. Parameters like PGV/PGA, peak Fourier amplitude, peak power amplitude, bracketed duration, and significant duration were unaffected by scaling of time histories.

Table 12 shows the spectral parameters for the selected time histories. The dispersion in *Sa* for different periods is different. Considering spectral parameters within a narrow band of fundamental periods makes the analysis more accurate.

Various ground-motion parameters evaluated showed variation for different ground motions. In order to determine the efficiency of some of the ground-motion parameters as IMs, lognormal variation in maximum displacement response observed for inelastic, constant-ductility (=3) SDOF has been considered as the engineering demand parameter (EDP). Ground-motion parameter which gives converged response of EDP is considered as efficient IM. Table 13 shows the dispersion of responses. Lesser dispersion indicates higher efficiency of the IM.

Table 12. Variation of spectral parameters when scaled to unit PGA

Record id	*Sa* at Tn = 0.8 s (m/s^2)	*Sa* at Tn = 0.6 s (m/s^2)	*Sa* at Tn = 1.0 s (m/s^2)	Median *Sa* for Tn = 0.6–1.0 s (m/s^2)	ASI (m/s)	VSI (cm)	MASI (m/s)	MVSI (cm)
3 LMSR	0.91	1.23	0.88	0.91	0.90	31.30	0.42	5.18
4 LMSR	1.06	1.09	1.01	1.06	1.01	23.85	0.44	5.55
5 LMSR	1.22	1.30	0.43	1.08	1.19	27.33	0.41	4.97
6 LMSR	1.15	1.67	0.55	1.10	1.01	24.76	0.41	4.99
7 LMSR	0.68	0.79	1.14	0.79	0.76	23.22	0.35	4.62
9 LMSR	0.63	1.07	0.88	0.79	0.92	29.20	0.33	4.14
10 LMSR	1.05	1.09	0.77	1.05	0.75	26.67	0.41	5.14
1 LMLR	0.74	1.32	0.64	0.82	0.95	24.93	0.35	4.41
4 LMLR	0.60	0.94	0.60	0.67	1.01	23.85	0.28	3.57
5 LMLR	0.47	1.21	0.49	0.52	0.85	21.25	0.26	3.17
6 LMLR	0.40	0.50	0.49	0.43	0.40	15.38	0.17	2.25
7 LMLR	0.75	0.80	1.21	0.80	0.94	28.65	0.35	4.61
8 LMLR	0.69	0.70	0.51	0.67	1.10	21.98	0.27	3.45
9 LMLR	0.68	0.82	0.65	0.63	0.96	22.14	0.27	3.43
10 LMLR	1.86	1.70	1.13	1.69	1.00	33.88	0.68	8.47
1 SMSR	0.93	1.33	0.50	0.91	0.83	25.05	0.38	4.67
2 SMSR	2.22	2.49	0.96	1.16	0.92	36.22	0.64	7.72
3 SMSR	1.36	2.74	1.14	1.36	1.01	46.78	0.64	7.80
1 SMLR	1.08	2.80	1.18	1.39	0.92	39.41	0.69	8.41
5 SMLR	1.16	1.28	0.45	1.16	0.79	22.12	0.43	5.30
Minimum	0.40	0.49	0.43	0.43	0.40	15.38	0.17	2.25
Maximum	2.22	2.80	1.21	1.69	1.19	46.78	0.69	8.47
Median	0.92	1.22	0.71	0.91	0.93	24.99	0.39	4.82
Dispersion	0.43	0.45	0.37	0.34	0.22	0.25	0.35	0.34

Table 13. Variation of engineering demand parameter when scaled to various parameters

Parameter	$\beta_{\ln}$ (maximum displacement of SDOF with $\mu = 3$)
Sa, median, 0.6 s ≤ Tn ≤ 1.0 s	0.29
PGV/PGA	0.30
VSI	0.30
MVSI	0.32
ASI	0.32
MASI	0.33
A95	0.34
SMV	0.37
Acc. RMS	0.39
Sa at Tn = 0.8 s	0.41
SMA	0.45
Vel. RMS	0.53
Arias intensity	0.85
PGD/PGA	1.07

Conclusion

Based on the present work it can be concluded that:

- The random nature of earthquakes requires using more than one ground-motion record to capture variability in response from the probable randomness of input motion.
- The selection and scaling of earthquake ground motions is an important step in defining the seismic loads that will be applied to a structure during structural analysis, and serves as an interface between seismology and engineering.
- As scaling only changes the magnitude, certain minimum number of records are required to cover the variability associated with frequency parameters.
- Twenty records having magnitude range 5.4–6.5 and radius range 10–90 km are sufficient to cover the variability of ground-motion parameters without any compromise in the results.
- PGA alone always does not give true information about the damage potential of an earthquake; some time histories are characterized by single-cycle peak amplitude.
- PGV/PGA, *Sa* (median, 0.6 s ≤ Tn ≤ 1.0 s), MVSI, and MASI are efficient IM with reference to a narrow band of fundamental period of a class of population of structures.
- It is proposed that instead of considering scaling of time histories to *Sa* at specific fundamental period, the time histories can be scaled to median *Sa* for the considered period range to reduce the dispersion of EDP.
- It is proposed based that the median *Sa* for Tn ranging from 0.6 to 1.0 s for each ground motion is the most efficient IM. The effectiveness of this measure is proved as dispersion in inelastic, constant-ductility (=3), displacement response (EDP) is reduced significantly. The parameter giving least dispersion for inelastic displacement response (EDP) and high

dispersion of S_a/PGA (IM) is considered as the most effective IM.

- *Sa* varies for different fundamental periods for a given time history. At the same time, *Sa* also varies for different time histories at a given fundamental period. Spectral accelerations considered for narrow range of fundamental period give more reliable results instead of wide-period bands.
- The dispersion in the various ground-motion parameters is effective in capturing the probabilistic variation in the future ground motion.

A final set of 20 recorded time histories is made available for performing IDA for structures having fundamental period in the range 0.6–1.0 s. Although the final set of selected ground motions covers variability of future ground motions for earthquake events having magnitude between 5.4 and 6.9, it does not cover variability for events having magnitude more than 6.9. Also this study does not consider variability due to near fault ground motions.

1. Vamvatsikos, D. and Cornell, A. C., Incremental dynamic analysis. *Earthquake Eng. Struct. Dyn.*, 2002, **31**(3), 491–514.
2. Shrikhande, M., *Atlas of Indian Strong Motion Records*, CD, Department of Earthquake Engineering, IIT Roorkee, 2001.
3. Dhakal, R. P., Mander, J. B. and Li, L., Identification of critical ground motions for seismic performance assessment of structures. *Earthquake Eng. Struct. Dyn.*, 2006, **35**(8), 989–1008.
4. Mander, J. B., Dhakal, R. P., Mashiko, N. and Solberg, K. M., Incremental dynamic analysis applied to seismic financial risk assessment of bridges. *Eng. Struct.*, 2007, **29**(10), 2662–2672.
5. Chopra, A. K. and Chintanpakdee, C., Inelastic deformation ratios for design and evaluation of structures: Single-degree-of-freedom bilinear systems. *J. Struct. Eng.*, *ASCE*, 2004, **130**(9), 1309–1319.
6. Ibarra, L. F. and Krawinkler, H., Global collapse of deteriorating MDOF systems. In Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, BC, Canada, 2004, Paper No. 116.
7. Shome, N. and Cornell A., Probabilistic seismic demand analysis of nonlinear structures. Report No. RMS-35, Department of Civil Engineering, Stanford University, USA, 1999.
8. Sewell, R. T., Damage effectiveness of earthquake ground motion: Characterizations based on the performance of structures and equipment. Ph D dissertation, Department of Civil and Environmental Engineering, Stanford University, USA, 1989, p. 619; http://www.stanford.edu/group/rms/
9. Shome, N., Cornell, A., Bazzuro, P. and Carballo, J. E., Earthquake records and nonlinear responses. *Earthquake Spect.*, 1998, **14**(3), 469–500.
10. Housner, G. W., Behavior of structure during earthquakes. *J. Eng. Mech. Div., ASCE*, 1959, **85**, 109–129.
11. Von Thun, J. L. and Rochim, L. H., Earthquake ground motion for design and analysis of dams. In *Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation*, Geotechnical Special Publication 20, ASCE, New York, 1988, pp. 463–481.
12. Newmark, N. M. and Hall, W. J., *Earthquake Spectra and Design*, EERI Monograph, Earthquake Engineering Research Institute Berkley, California, 1982, p. 103.
13. Nuttli, O. W., The relation of sustained maximum ground acceleration and velocity to earthquake intensity and magnitude. In Miscellaneous Paper S-71-1, Report 16, US Army Corps of Engineers, Waterways Experiment Station, Mississippi, 1979.
14. Benjamin, J. R. *et al.*, A criterion for determining exceedance of the operating basis earthquake. EPRI Report NP-5930, Electric Power Research Institute, Palo Alto, California, 1988.
15. Kennedy, R. P., Ground motion parameter useful in structural design. Presented at the Conference on Evaluation of Regional Seismic Hazards and Risk, Santa Fe, New Mexico, 1980.
16. Sarma, S. K. and Yang, K. S., An evaluation of strong motion records and a new parameter A95. *Earthquake Eng. Struct. Dyn.*, 1987, **15**, 119–132.
17. Arias, A., A measure of earthquake intensity. In *Seismic Design for Nuclear Power Plants*, MIT Press, Cambridge, 1970, pp. 438–483.
18. Chopra, A. K., *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, Prentice-Hall, 1995.
19. Seismosignal, Version 3.2.0, Seismosoft Corporation.

Received 12 July 2009; revised accepted 22 November 2010