Choice of Intensity Measure in Incremental Dynamic Analysis

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Abstract

Incremental Dynamic Analysis (IDA) is a powerful method for evaluating the seismic performance of structural systems. When undertaking IDA, ground motion records are scaled in accordance with a certain seismic intensity level up until the limit of collapse of the structure is reached. Peak ground acceleration (PGA) is often used as intensity measure (IM) for scaling purposes whilst peak ground velocity and response spectral accelerations can also be used as IM parameters. One of the challenges with the IDA methodology is that the performance (IDA) curve obtained for a structural system is non-unique meaning that it can be sensitive to changes in the frequency properties of the input excitations. Thus, there is a practice of holding the frequency content of the excitation unchanged (or constrained to a model design spectrum) when applying the scaling. The shortcomings with this modelling approach is that real conditions can be misrepresented given that frequency contents of the earthquake changes with magnitude and distance. This paper presents findings from an investigation which was aimed at determining what ground motion parameters are most suitable for use as IM whilst allowing the frequency content of the earthquake to vary in an IDA. This paper presents findings of an investigation wherein nonlinear dynamic time-history analyses involving the use of both recorded and artificial accelerograms were undertaken on models representing buildings constructed of cold-formed steel. For systems with natural period of less than 0.4s; PGA and the peak acceleration demand (PAD) have been found to be desirable choices of IM parameters given that the associated IDA curves are generally insensitive to the choice of the accelerogram ensemble used. For systems possessing longer natural periods of vibration, other ground motion parameters have been found to be more desirable choices.

Keywords: Incremental dynamic analysis (IDA), intensity measure (IM), peak ground acceleration (PGA), peak acceleration demand (PAD)

1. INTRODUCTION

Engineers have used over the years several different analysis techniques to estimate the seismic performance of new or existing structures located at a specific site. With the advent of performance-based earthquake engineering and the increasing availability of sophisticated structural analysis software and faster computers, nonlinear dynamic time-history analysis has become more widely used for design and evaluation of structures. Vamvatsikos & Cornell (2002) proposed a computational-based methodology called incremental dynamic analysis (IDA) which involves a series of non-linear dynamic time history analyses to estimate structural performance under seismic loads. One of the biggest issues for performing IDA is the selection of appropriate ground motion records. The input ground motions to such analyses are usually selected to be either representative of earthquake scenarios that control the site hazard or consistent with a target elastic response spectrum. In both cases, the desired input ground motions are usually very intense. The scarcity of real ground motion records with the right characteristics has often forced practitioners to manipulate recorded accelerograms. The manipulation involves scaling the input time histories from a limited recorded strong motion data to achieve a reliable date with a very gradual change in the intensity of ground shaking. One of the most common scaling approaches is to multiply the amplitude of a record by the constant scalar factor necessary to reach a target spectral acceleration level at the fundamental natural period of the structure with a damping ratio of 5%, denoted as S_A (T_1). However simulations using such scaling method might misrepresent the response of the structure as the frequency content and the intensity of ground motion are not independent.

Another major issue with IDA is the selection of ground motion parameter as intensity measure (IM). PGA is often used as an intensity measure as it is a common measure to represent seismic hazard in most standards. Spectral acceleration (S_A (T_1)) as an intensity measure was also used by some researchers (Shome *et al.*, 1998; Bradley *et al.*, 2008). According to Shome *et al.* (1998), spectral acceleration served as a more consistent intensity measure especially for a simple structure represented by a single-degree-of-freedom system. However S_A (T_1) has significant deficiencies when used as IM as it does not include the effects of changes in the period of vibration in an inelastic response. Also, the single parameter S_A (T_1) does not reflect many of the aspects of earthquake ground motions that affect inelastic stiffness and strength degradation. Comprehensive research has been conducted in recent years to develop several new intensity measures. For example, Cordova *et al.* (2001) introduced an IM that takes into account the elongation of the first modal period of vibration as a result of nonlinear behaviour. The analysis results showed that this IM is more effective than S_A (T_1).

To investigate these issues, several bins of ground motion records for generating IDA curves have been employed in this study. Both recorded and artificial records including scaled and unscaled accelerograms have been used for performing IDA. The variation/deviation of IDA curves obtained from different bins of ground motion records sets were compared when various ground motion parameters were used as IM.

2. GROUND MOTION RECORDS

One of the major problems with IDA is that a curve showing "Drift versus IM" may vary for the same structure depending on the frequency content (response spectral shape) and nature of the ground motions applied. Thus, it is very difficult to interpret IDA curves which are based on certain ensemble of accelerograms given that different accelerograms may give very different IDA curves. Because of these variabilities, software such as SeismoMatch V2.1 (SeismoSoft, 2013) has been developed to pre-condition accelerograms in order that their response spectral behaviour matches with a certain pre-defined target spectrum. In this study, five different bins of ground motion records (as listed in Table 1) including artificial and recorded ground motions along with modified accelerograms using SeismoMatch have been employed in the parametric studies. Ground motion records in each bin were generated using program GENQKE (Lam, 1999) or collected (PEER ground motion database) for rock site keeping similar moment magnitude (M) and epicentral distance (R) except Bin 1 in which epicentral distance varied from 13 to 150km. As the sub soil conditions vary from site to site, the idea is to collect the ground motion records for rock site of the considered region and then extended to cover for conditions on soil sites (consistent with Site Class D as per AS1170.4, 2007) using program SHAKE (Idriss & Sun, 1992).

2.1 Artificial earthquakes (Bins 1 & 2)

Bin 1 contained a large number of accelerograms (some 18 scenarios with constant magnitude M=6.5 and epicentral distance ranging from R=13 to 150km) to perform IDA with a very gradual change in the intensity of ground shaking. Unlike other considered bins, no scaling was applied to accelerograms in bin 1 for performing IDA. Moreover 12 accelerograms were simulated for each M-R earthquake scenarios in order to include some randomness inherent in the ground motions.

Bin 2 contained a specific set of earthquake scenario from bin 1 that matched with the target spectra on rock site. Simulations based on earthquake scenario M=6.5 & R=40km on rock site seem to match best with the design response spectra stipulated by the Standard Australia (AS1170.4, 2007) for Melbourne rock conditions.

2.2 Recorded earthquakes (Bin 4)

Due to the paucity of strong ground motion database in Australia, all recorded earthquakes considered in this study were taken from the Pacific Earthquake Engineering Research Center (PEER) database (http://peer.berkeley.edu/peer_ground_motion_database/). In order to maintain the consistency between the artificial (bin 2) and recorded earthquakes, the selection of recorded earthquakes were constrained by magnitude (close to M=6.5), epicentral distance (close to 40km) and the average shear wave velocity of top 30 meters of the site (Vs30 greater than 500m/s consistent with rock conditions of Australia) such that the resulting response spectra fairly matched with the target spectra on rock site (similar to bin 2 earthquakes). A list of 12 unscaled recorded accelerograms on rock site is provided in Table 2 which is further simulated using program SHAKE to obtain accelerograms on soil sites.

2.3 Modified earthquakes (Bins 3 & 5)

Each accelerograms from bins 2 and 4 (on rock site) were modified to closely match with the rock target spectra to generate accelerograms on rock outcrops for bins 3 and 5 respectively. Program SeismoMatch was used for adjusting accelerograms to match the design target response spectrum on rock site, using the wavelets algorithm proposed by Abrahamson (1992) and Hancock *et al*. (2006). Further simulations were carried out for sub-soil condition using program SHAKE and the input motions taken from rock outcrop for each set of earthquake scenarios.

Samples of median response spectra on rock and soil sites resulting from ensembles of accelerograms from source bins 2 to 5 are plotted in Figure 1.

Table 1 List of considered earthquake scenarios for IDA

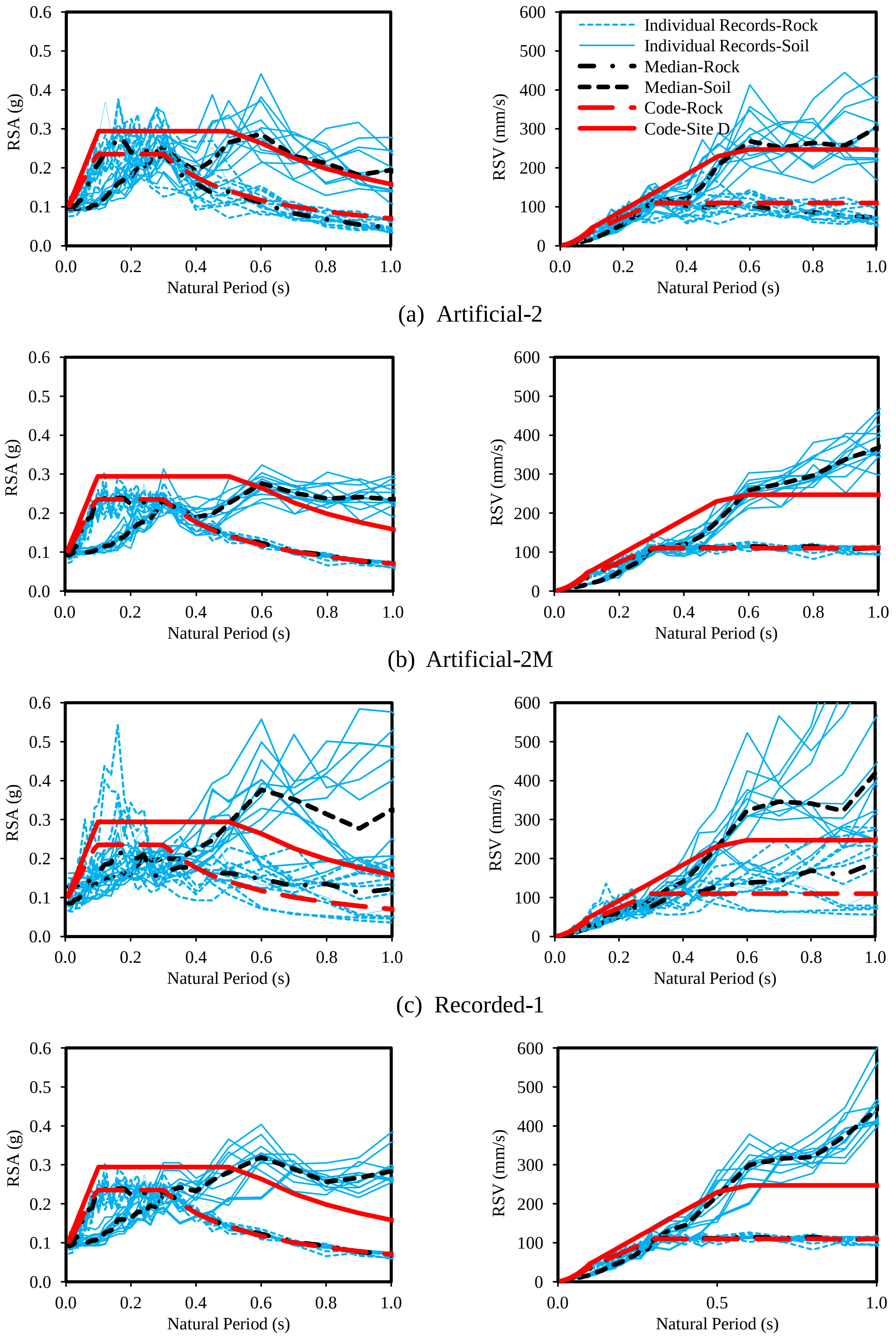
Bin	Type	Modification[1]	M[2]	R[3] (km)	Scaling[4]	No. of accelerograms[5]	No. of IMs[6]	Nomenclature
1	Artificial	No	6.5	13-150	No	12	18	Artificial-1
2	Artificial	No	6.5	40	Yes	12	18	Artificial-2
3	Artificial	Yes	6.5	40	Yes	12	18	Artificial-2M
4	Recorded	No	6.3 to 6.7	30-60	Yes	12	18	Recorded-1
5	Recorded	Yes	6.3 to 6.7	30-60	Yes	12	18	Recorded-1M

[1]Modified to match with target spectra on rock site, [2]Moment magnitude, [3]Epicentral distance, [4]Scaling to desired intensity of ground shaking, [5]Total number of accelerograms for each earthquake scenario defined by M-R combinations, [6]Total number of IMs to scale the records for performing IDA

Table 2 Collected recorded earthquakes

No.	Event	Component	Year	Station	M[1]	R[2] (km)	Vs30[3] (m/s)	PGA[4] (g)	PGV[5] (mm/s)
1	San Fernando	FN*	1971	Pearblossom Pump	6.61	39.0	529	0.135	46.5
2	San Fernando	FP+	1971	Pearblossom Pump	6.61	39.0	529	0.130	46.1
3	Northridge-01	FN*	1994	Alhambra-Fremont School	6.69	36.8	550	0.100	81.3
4	Northridge-01	FP+	1994	Alhambra-Fremont School	6.69	36.8	550	0.080	99.0
5	Northridge-01	FN*	1994	Leona Valley #1	6.69	37.2	685	0.108	78.9
6	Northridge-01	FP+	1994	Leona Valley #1	6.69	37.2	685	0.046	63.4
7	Northridge-01	FN*	1994	Leona Valley #3	6.69	37.3	685	0.099	82.6
8	Northridge-01	FP+	1994	Leona Valley #3	6.69	37.3	685	0.068	93.5
9	Northridge-01	FN*	1994	Palmdale-Hwy 14 & Palmdale	6.69	41.7	552	0.074	64.3
10	Northridge-01	FP+	1994	Palmdale-Hwy 14 & Palmdale	6.69	41.7	552	0.069	107.4
11	Chi-Chi-Taiwan-06	FP+	1999	CHY029	6.30	41.4	545	0.118	108.4
12	Chi-Chi-Taiwan-06	FP+	1999	CHY087	6.30	56.3	505	0.085	91.7

[1]Moment magnitude, [2]Closest distance to fault rupture, [3]Average shear wave velocity of top 30 meters of the site, [4]Peak ground acceleration, [5]Peak ground velocity, *Fault-normal, +Fault-parallel
Source: PEER Strong Motion Database, http://peer.berkeley.edu/peer_ground_motion_database/



(a) Artificial-2

(b) Artificial-2M

(c) Recorded-1

(d) Recorded-1M

Figure 1 Response spectra (5% damping) of considered earthquake scenarios on rock and soil sites

3. COMPUTATIONAL MODELLING OF CONSIDERED STRUCTURE AND ANALYSES RESULTS

The structure selected in this study was a 2.4mx2.4m cold-formed steel (CFS) framed wall panel sheathed with fibre cement boards. The details of wall panel configurations and connections are shown in Figure 2a. The CFS frame was made of 89x36x0.75mm C-shaped lipped studs (with web stiffened) and 91x40x0.75mm plain channel sections for plates and noggings. Studs were placed at 600mm spacings and two identical fibre cement boards of 5mm thickness were used as the sheathing boards, attached vertically on one face of the wall panel. All CFS members were grade G550 and the connections between them were made using 15mm long M6 GX® Frame Screws (Buildex). The sheathing boards were connected to the framing members at 100mm spacings along the periphery of the board and at 150mm spacings for the middle portion of the board. All sheathing screws were 20mm long M5-16TPI CSK FibreZips self drilling screws (Buildex). The wall panel was tested under quasi-static loading conditions using a loading protocol (Shahi *et al*., 2013) which had been developed based on the seismic conditions of Australia (AS 1170.4, 2007). Refer to Shahi *et al*. (2014) for detail test set up and experimental results. The hysteretic curve of the wall panel from racking test is shown in Figure 2b.

In order to perform IDA of the considered wall panel, computer program SAPWood V2.0 (Pei & van de Lindt, 2010) was used. Most researches modelled wall panel as single-degree-of-freedom (SDOF) system with an equivalent dynamic characteristics to that of detailed wall panel model for performing dynamic analysis (Folz & Filiatrault, 2001; Filiatrault & Folz, 2002; Rosowsky 2002; Folz & Filiatrault, 2004). In this study, the wall panel was also modelled as an equivalent single-degree-of-freedom (SDOF) system using a nonlinear hysteretic spring element SAWS (Folz & Filiatrault, 2001) for performing dynamic analysis. SAWS parameter that closely fits the hysteretic curve of the tested wall panel is provided in Figure 2b. Using SAPWood, nonlinear dynamic time history analyses were carried out on SDOF system using a nonlinear SAWS spring with critical damping ratio of 5% and varying masses to cover a range of vibration periods (T_1 = 0.25 to 1.0s). Each model was subjected to various bins of earthquake records listed in Table 1. As described in Section 2, each ground motion records were scaled to vary the intensity measure except for earthquake records from bin 1 in which no scaling was applied.

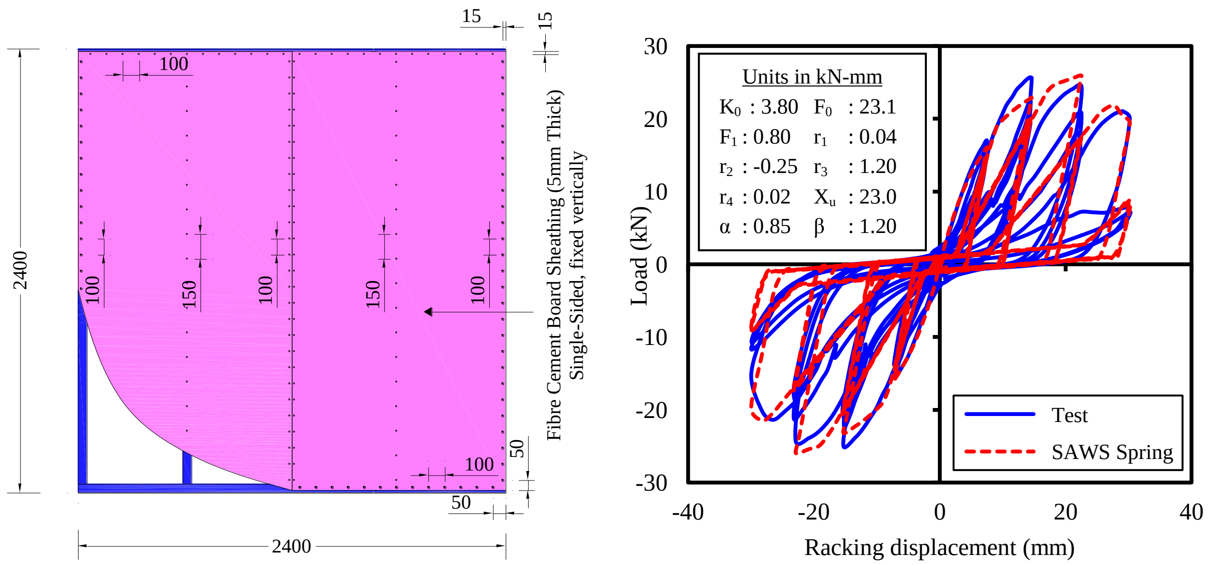
Most researches used PGA as an intensity measure (PGA is used as a common measure to represent seismic hazard in most of the standards), whereas spectral acceleration at first structural natural period (S_A (T_1)) as an intensity measure was also used by some researchers (Shome *et al*., 1998; Bradley *et al*. 2008). According to Shome *et al*. (1998), spectral acceleration served as a more consistent intensity measure especially for a simple structure represented by single-degree-of-freedom system. Nevertheless, this study considered several IMs (PGA, PGV, S_A (T_1), PAD, PVD and PDD) to identify the variations in IDA curves for the considered structure subjected to different bins of earthquake records. New parameters namely PAD, PVD and PDD are introduced herein as alternative IM parameters. These parameters have the attributes of addressing response behaviour of the structure and are not biased to a particular natural period of vibration. PAD, PVD and PDD refer to the maximum response spectral acceleration, velocity and displacement respectively (i.e. highest point on the corresponding response spectra) (Lam *et al*., 2000) which are illustrated in Figure 3.

IDA curve for one earthquake record was constructed by plotting the range of IM values against corresponding engineering demand parameter (EDP). In this study, maximum racking displacement from each non-linear time history analyses was used as an EDP. A typical IDA curve (PGA vs racking displacement) generated for one earthquake record from source bin 2 is shown in Figure 4a. IDA curves for other earthquake records from different sources of bins were generated through series of non-linear time history analyses. Illustration of the generated IDA curves for earthquake ensemble records from source bin 2 is shown in Figure 4b. Variations in the IDA curves resulted from the random nature of ground motion records (of same magnitude-distance combination) is well demonstrated in Figure 4b. For comparison of IDA curves resulting from different bins of earthquake records, this study considered moderate (50th percentile or median) as well as extreme (95th percentile) IM values.

Sample comparison of median IDA curves (median values plotted against different IMs for considered structure with vibration period of 0.40s) obtained from various bins of earthquake records is shown in Figure 5. Similar trend was observed with 95th percentile values. From Figure 5, it is shown that the median IDA curves can be sensitive to the choice of accelerograms being adopted for the analyses. The sensitivity depends on what parameter has been used as IM and also on the initial natural period of vibration of the structure. For example, with a structure having an initial natural period of vibration of 0.40s, the use of artificial accelerograms and recorded accelerograms give very different IDA curves when parameters such as PGA or S_A (T_1) has been used as IM (Figure 5). The use of Seismomatch to constrain the frequency content of the accelerograms has resulted in a much lower value of PGA, PGV or S_A (T_1) value to impose a certain amount of drift on the structure. PGA is well known for its limitations as its value can be sensitive to high frequency content of the vibrations in the ground and such high frequency content can have little effects on the response behaviour of the structure. S_A (T_1) addresses response behaviour of the structure but is based on the notion that the natural period of vibration stays constant during the course of the excitations. In reality, the natural period of vibration can vary as the effective stiffness of the structure decreases with increasing displacement amplitude. It is seen that the use of PGV or PVD as IM has resulted in very similar IDA curves across all accelerogram ensembles used. However, the behaviour of these parameters depends on the natural period of vibration of the structure.

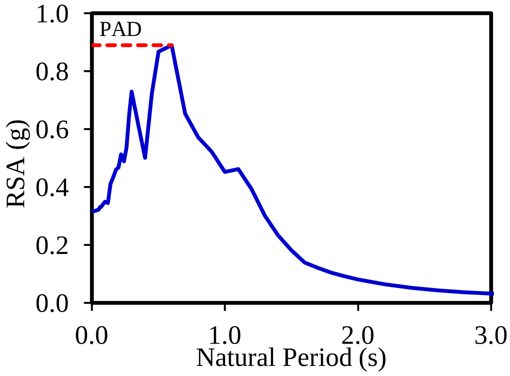
A more detailed illustration of the results for different choices of parameters as IMs and for different vibration periods is plotted in Figure 6. The plot shows limiting values of IMs at which the considered structure reached its ultimate limit state (ULS) condition. Refer to Figure 7 for the definition of limit state conditions. Serviceability limit state (SLS) refers to the displacement at H/300 (Experimental Building Station, 1978) and ULS is the displacement corresponding to 80% of strength at the declining portion of the back bone curve. Average value from the last cycle backbone curves was used for obtaining limit state conditions using the Equivalent Energy Elastic-Plastic (EEEP) principle (AISI Standard, 2007). It is shown in Figure 6 that the behaviour of the IM parameter depends on the vibration period of the structure. IDA curves using IMs as PGA and PAD provide consistent result regardless of the earthquake records used for vibration periods in the range of 0.25 to 0.40s. For structure with vibration periods of 0.40 to 0.70s, and longer than 0.7s; PGV & PVD, and PDD respectively generate consistent IDA curves. The complimentary attributes of the newly introduced parameters PAD, PVD and PDD are well demonstrated. IDA curves plotted using S_A (T_1) as IM showed larger deviation in the results irrespective of the vibration period of the structure.

Variation in the results obtained from different bins of earthquake records using various IMs had been expressed by lognormal standard deviation or dispersion (β) which is plotted in Figure 8 (median values for ULS). Since the dispersion is expressed in natural log, a value of β = 1.0 refers to no dispersion and a value of β = 1.2 refers to a dispersion of 18.23%. A dispersion of less than 18% was noticed using IMs PGA & PAD; PGV & PVD; and PDD for structures with vibration periods less than 0.4s; between 0.40 to 0.70s; and greater than 0.7s respectively. Similar findings were observed for SLS condition with both median and 95th percentile values. A maximum dispersion of 1.70 which corresponds to nearly 50% deviation was noticed if deficient IM is selected.

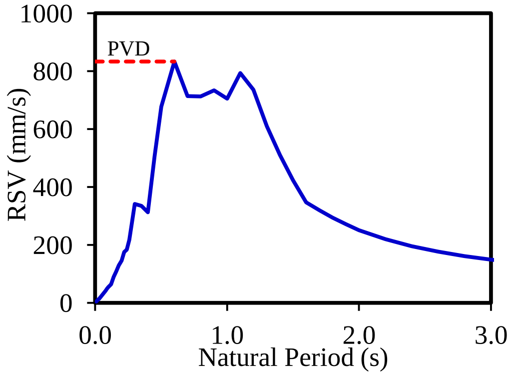


(a) Configuration of wall panel (all units in mm) (b) SAWS parameter fitting the hysteretic curve

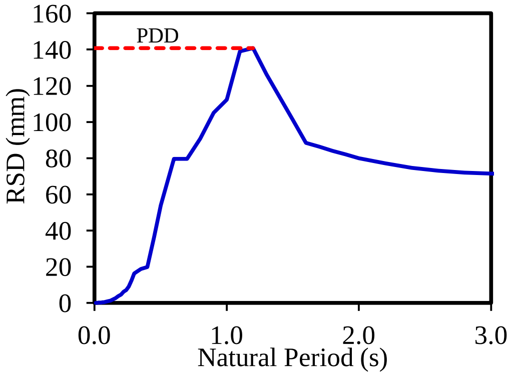
Figure 2 Considered structure (2.4mx2.4m cold-formed steel-framed wall panel)



(a) PAD

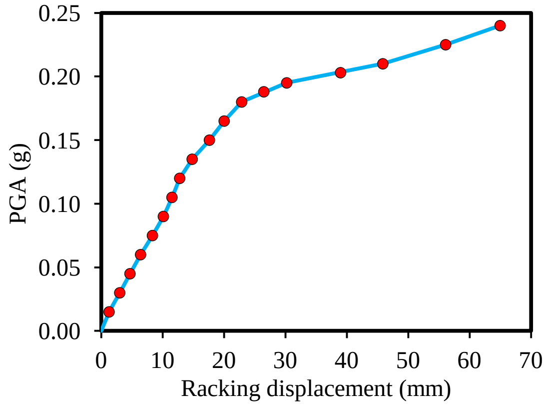


(b) PVD

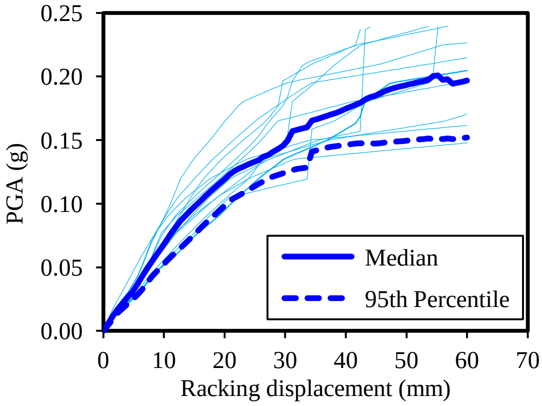


(c) PDD

Figure 3 Illustration of PAD, PVD and PDD (Sample response spectra taken from Artificial-1 accelerogram on soil (M=6.5 & R=15km)

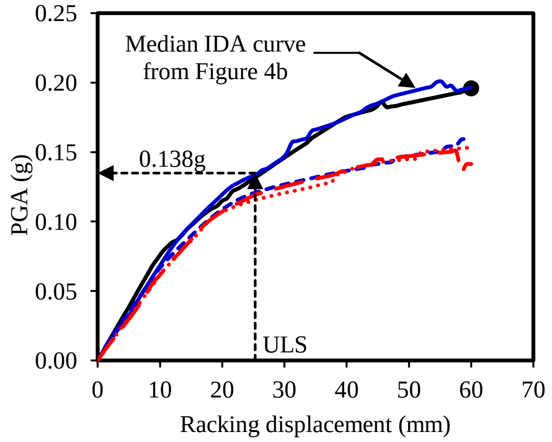


(a) IDA curve for one earthquake record

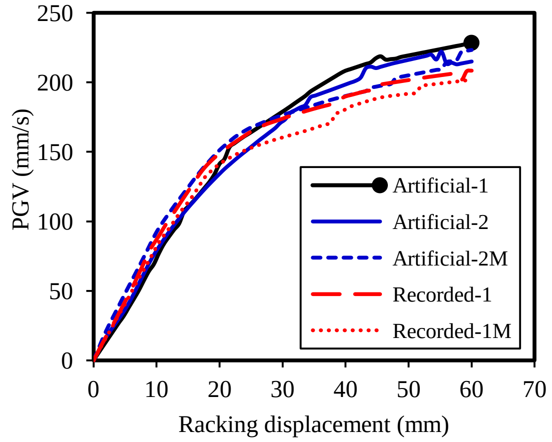


(b) IDA curve ensemble for 12 earthquake records showing median and 95th percentile curves

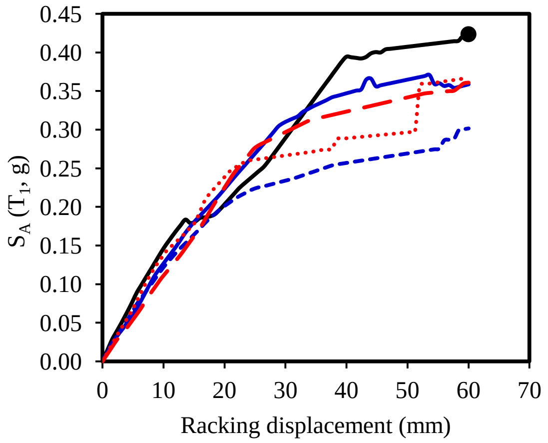
Figure 4 IDA results from source bin 2 for considered structure with vibration period of T_1 = 0.40s



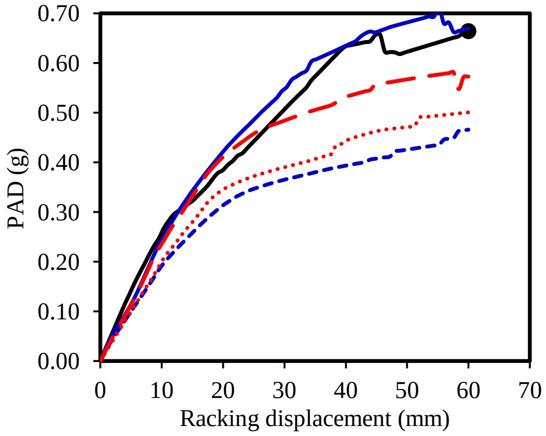
(a) PGA as IM



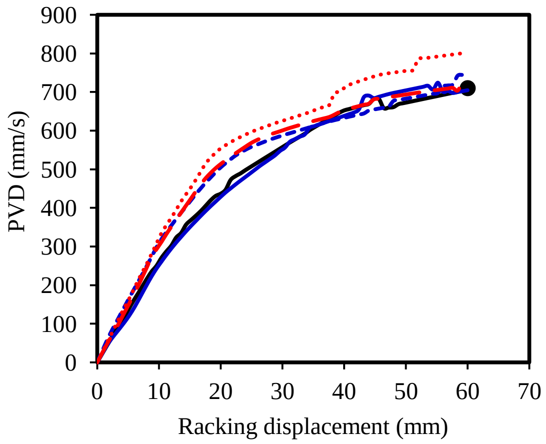
(b) PGV as IM



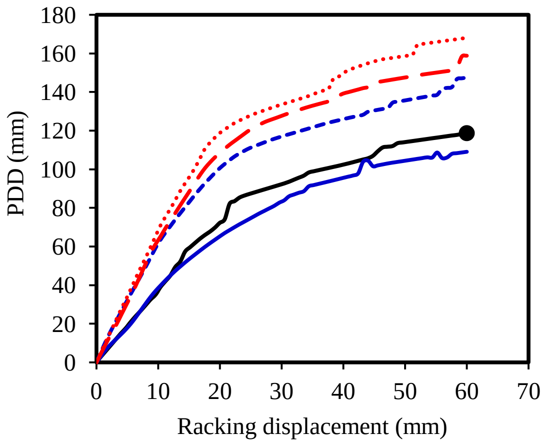
(c) S_A (T_1) as IM



(d) PAD as IM

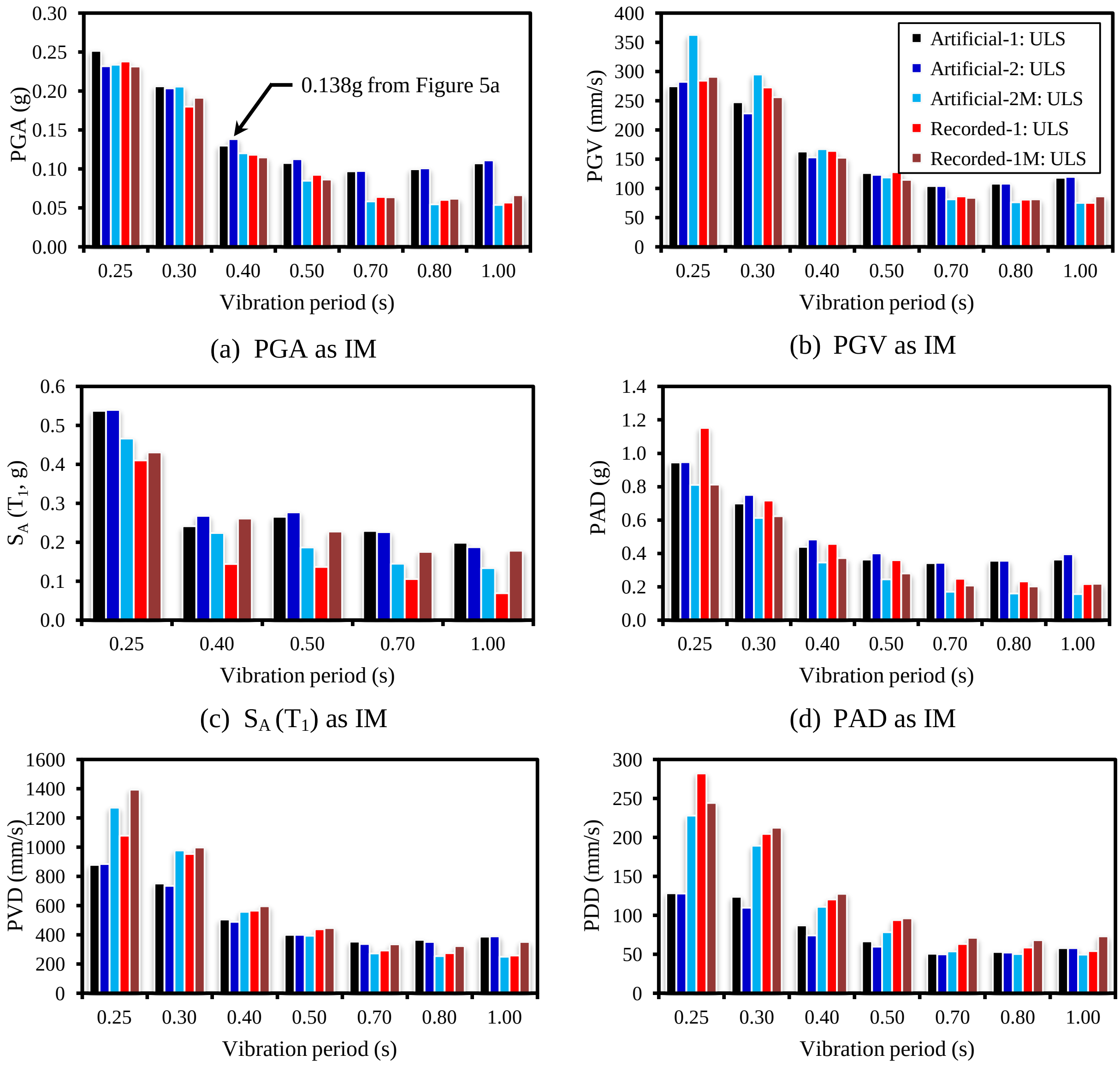


(e) PVD as IM



(f) PDD as IM

Figure 5 Median IDA curves plotted against various IMs for considered structure with vibration period of $T_1 = 0.40$s



(a) PGA as IM

(b) PGV as IM

(c) $S_A (T_1)$ as IM

(d) PAD as IM

(e) PVD as IM

(f) PDD as IM

Figure 6 Limiting values (median) plotted against various IMs at which the considered structure reached its ULS condition (all vibration periods are not shown for clarity)

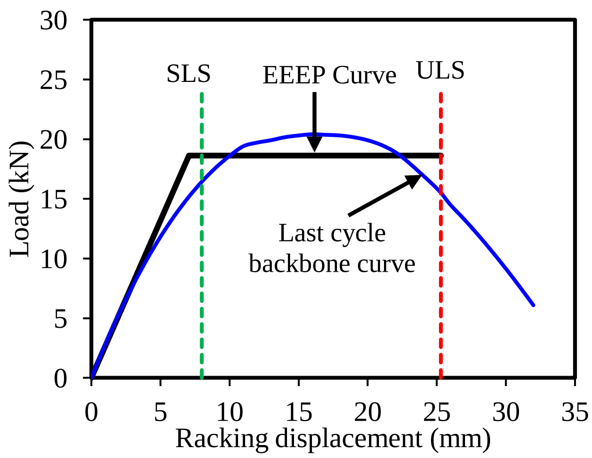


Figure 7 Limit state conditions of considered structure

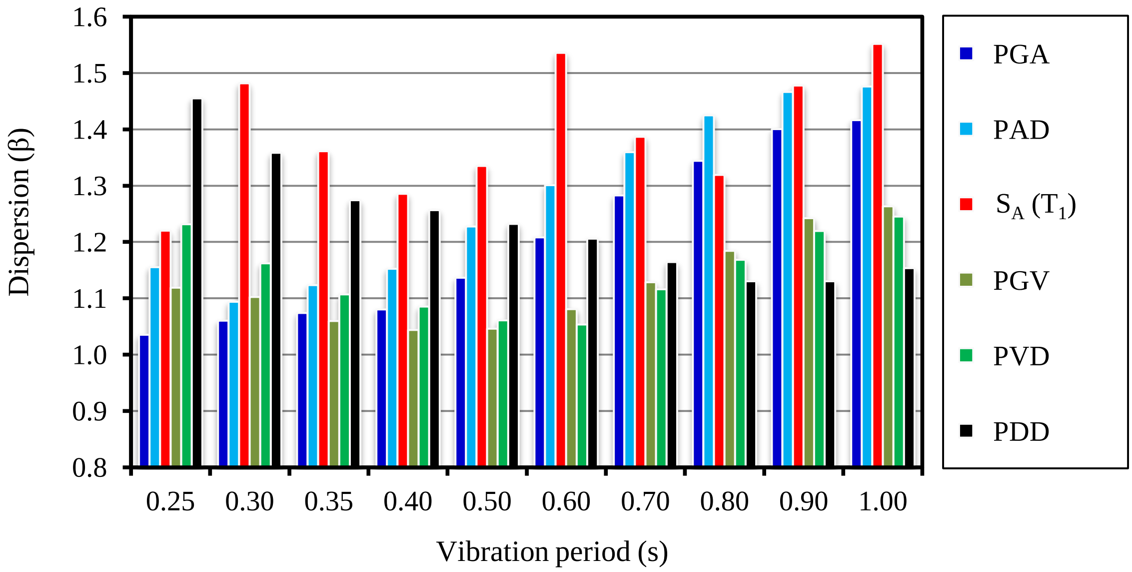


Figure 8 Dispersion of the limiting values (median) obtained from 5 different bins of earthquake records at which the considered structure reached its ULS condition

4. CONCLUDING REMARKS

This paper presented the generation of incremental dynamic analysis (IDA) curves for the seismic assessment of cold-formed steel-framed wall panel. 5 different bins of ground motion records including both recorded and artificial accelerograms were considered for generating IDA curves. First bin of earthquake records contained artificial records in which no scaling of ground motion record was applied for generating IDA curves. However, the remaining bins of earthquake records contained a limited range of intensity level and hence scaling of each ground motion records was undertaken to generate IDA curves. Several non-linear dynamic time history analyses were conducted for a single-degree-of-freedom (SDOF) system with an equivalent non-linear hysteretic behaviour obtained from racking test and for vibration periods ranging from 0.25 to 1.0s. IDA curves are constructed using 6 different IMs such as PGA, PGV, S_A (T_1), PAD, PVD and PDD plotted against racking displacement as an engineering demand parameter (EDP). Median (50^{th} percentile) and 95^{th} percentile IDA curves were obtained from each bins of earthquake records and corresponding curves were compared to find the variation in the result expressed in terms of log normal standard deviation or dispersion (β). It was found with a dispersion of less than 20% if IMs PGA & PAD; PGV & PVD; and PDD were used respectively for structures with vibration periods less than 0.4s; between 0.40 to 0.70s; and greater than 0.70s. It was observed that IDA curves plotted using S_A (T_1) as IM showed larger deviation in the results irrespective of the vibration period of structure. A maximum dispersion of 1.70 which corresponds to nearly 50% deviation was noticed if deficient IM is selected for plotting IDA curve. This study did not consider earthquake scenarios with different moment magnitude and soil type nor structures with vibration period longer than 1.0s and possessing different hysteretic behaviour. The objective of the study is to investigate the variations in IDA curves generated from different ground motion records (recorded and artificial; scaled and unscaled). A good choice of ground motion parameters as IM was found to give better confidence (less dispersion and discrepancy) in the result regardless of what ground motion ensembles were used.

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