



INCREMENTAL DYNAMIC ANALYSIS OF NONLINEAR STRUCTURES: SELECTION OF INPUT GROUND MOTIONS

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ABSTRACT

Incremental Dynamic analysis (IDA) is used in seismic performance assessment of nonlinear structural systems. This paper presents comparisons between two different Incremental Dynamic Analysis implementation techniques: IDA1 (classical IDA) and IDA2 (based on different return period ground motions) to assess IDA robustness. Analyses are performed for SDOF structures designed for the National Building Code of Canada (NBCC) 2005 in Eastern North America (ENA) and for an eight story braced steel frame located in Montreal, Canada. Within the IDA framework, the paper discusses the following ground motion Intensity Measures (IM) (peak ground acceleration PGA, spectral ordinate $S_a(T_1, 5\%)$, root mean square of acceleration (RMSA), Arias Intensity), and the following structural demand parameters (ductility, number of yield events...). The effect of the number of ground motion records used in a nonlinear seismic safety assessment process is also examined. Results have been shown to be sensitive to the IDA method selected and to the ground motions set selected.

Introduction

Incremental Dynamic Analysis (IDA) is a recent method developed (Vamvatsikos 2002, 2005) to attain a better knowledge of structural behaviour during earthquakes. In an IDA analysis, one or more ground motions are modified to fit different Intensity Measures (IM) using a Scale Factor (SF) (Vamvatsikos 2002). Structural models are then subjected to nonlinear dynamic time history analyses performed for each IM level until models undergo structural instability appearing after gradual damage evolution for ductile structures. Different Damage Indices (DI) can be computed to get a relation between demand and capacity.

To assess the robustness of IDA analysis, two different implementation techniques are proposed in this paper:

- The first one, IDA1, consists of a traditional incremental dynamic analysis: a structure is tested with a single ground motion record scaled to different intensity levels.
- The second one, IDA2, which is more representative of ground shaking at different

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intensities, is however dependent on the method to scale (or match) selected ground motions records of different return periods to the target spectra. Those records exhibit different characteristics such as PGA and duration as the return period is lowered.

In this paper, the ground motions selected for the study are first characterised by the description of intensity measures (IM). Structural damage indices (DI) are then described. Two types of nonlinear structures have been tested: a set of SDOF structures with 6 different natural periods of vibration and an eight-story braced steel frame designed according to 1990 NBCC. Results obtained from these analyses are discussed. Finally, a section about the number ground motions records to use for seismic safety assessment of ductile structure located in Eastern North America (ENA) is also presented.

Ground Motions and Intensity Measures (IM)

Five response spectra compatible ground motions with Return Periods (RP) ranging from 200 to 10 000 years (200, 1000, 2500, 5000 and 10 000 years) were considered in this study. Those three-dimensional ground motions were compatible for the Ottawa Valley and were spectrally matched (in the frequency domain) to the target spectra (Limoges and Léger 2009). This does not mean that all characteristics of the spectrally matched ground motions are consistent with those of the original motions. Only the longitudinal record component is considered here as preliminary analyses showed identical results for both longitudinal and transversal components. For IDA1 method, the spectrally matched 2500 years return period accelerogram has been selected and scaled using a scalar multiplier to attain the different IM values obtained for the rest of the set.

Several IM can be considered to perform IDA analyses. The most commonly used in the literature are the Peak Ground Acceleration (PGA) (Mander 2007, Dolsek 2009) and the spectral acceleration at the fundamental period: $S_a(T_1, 5\%)$ (Vamvatsikos and Cornell 2002, 2005). However, other IM can be selected, such as:

- Arias Intensity (AI), which is an estimate of the amount of energy delivered to the structures. It is defined as:

$$AI = \frac{\pi}{2g} \int_0^{t_d} [a(t)]^2 dt \tag{1}$$

where g is the acceleration of gravity and $a(t)$ is the acceleration-time history in units of g .

- The Root Mean Square of Acceleration (RMSA) which represents an average amplitude for each accelerogram. It is given by the expression:

$$RMSA = \sqrt{\frac{AI}{t_d}} \tag{2}$$

where AI is the Arias Intensity defined in Eq. 1 and t_d is the record's effective duration.

Fig. 1 presents a comparison between the ground motions obtained for PGA-based IDA1 and IDA2 for three different RP: 200, 2500 and 10 000 years. IDA1 doesn't change record's

duration when it is scaled. Tables 1 and 2 present different characteristics for the PGA-scaled record and the accelerograms used for IDA2. They show that the IDA1 method doesn't change characteristics such as D3 (time needed for the integral of the square of the ground acceleration to build up from 5% to 95% of its final value), NZC (Number of Zero Crossings), pulse duration (DP) and beginning time of the pulse (TP) (for both longest and largest pulses) as Fourier coefficients are not modified (Léger and Tremblay 2009). Other characteristics are also considered, such as the Peak Ground Velocity (PGV), the acceleration spectrum intensity (SIa), the velocity spectrum intensity (SIv), the Predominant Period of Shaking (PPS), the Pulse Area (A*) for both longest and largest pulses, a/v which is the peak acceleration to peak velocity ratio and ad/v^2 , which is the ratio representative of the energy characteristics (d is the peak displacement).

Structural Damage Indices (DI)

Literature abounds on structural Damage Indices that are based on either force, deformation or energy. Several indices have been selected among those categories:

- Ductility (μ), which is the ratio of maximal displacement over yield displacement, is one of the most recurrent one (Vamvatsikos and Cornell 2002, 2005) as it indicates the structural ability to undergo plastic deformation without collapse.
- Maximum Roof Displacement, which is useful for multi-storey buildings.
- The Number of Yield Events (NYE) for low cycle fatigue consideration.

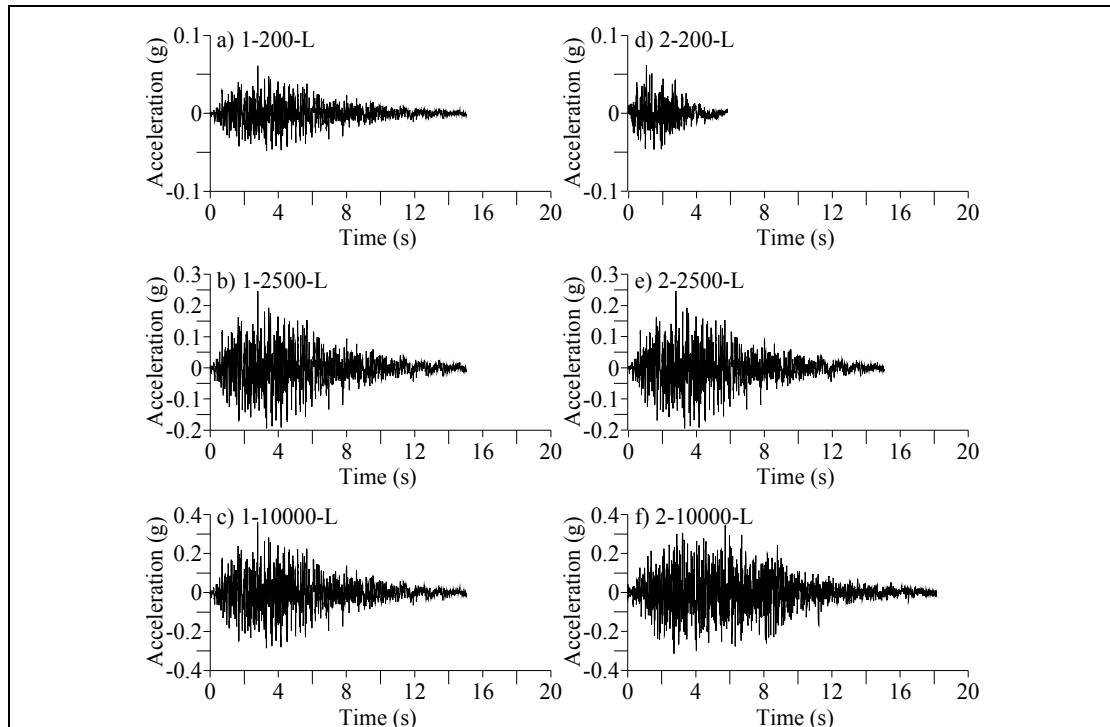


Figure 1. IDA1 PGA-based a), b), c) and IDA2 d), e), f) longitudinal ground motions for different return periods (captions are based on the following notation: IDA method used (1 or 2) - RP - Longitudinal component).

Table 1. Ground motions characteristics for PGA-based IDA1 and IDA2.

Ottawa Valley	PGA (g)	PGV (cm/s)	a/v (g*s/m)	ad/v2	D3 (s)	AI (m/s)	SIv (cm)	SIa (g*s)	RMSA (g)	PPS (s)
IDA1-PGA										
1-200L	0,06	2,03	29,57	11,09	7,75	0,03	8,51	0,03	0,01	0,05
1-1000L	0,15	5,06	29,57	11,09	7,75	0,19	21,23	0,07	0,03	0,05
1-2500L	0,25	8,18	29,57	11,09	7,75	0,5	34,3	0,12	0,05	0,05
1-5000L	0,29	9,57	29,57	11,09	7,75	0,68	40,13	0,14	0,05	0,05
1-10000L	0,36	11,99	29,57	11,09	7,75	1,06	50,28	0,17	0,07	0,05
IDA2										
2-200L	0,06	1,39	43,57	17,27	3,04	0,01	6,02	0,02	0,01	0,04
2-1000L	0,15	3,53	42,43	16,18	8,33	0,22	15,65	0,07	0,03	0,05
2-2500L	0,25	8,18	29,57	11,09	7,75	0,5	34,3	0,12	0,05	0,05
2-5000L	0,29	7,74	36,54	13,02	7,5	0,81	36,04	0,13	0,06	0,05
2-10000L	0,36	6,88	51,52	29,92	8,8	1,36	36,11	0,15	0,07	0,05

Table 2. Ground motions characteristics for PGA-based IDA1 and IDA2.

Ottawa Valley	NZC total	NZC in D3	Largest pulse in D3			Longest pulse in D3		
			A* (g*s)	DP (s)	TP (s)	A* (g*s)	DP (s)	TP (s)
IDA1-PGA								
1-200L	555	308	0,002	0,132	6,145	0,002	0,132	6,145
1-1000L	555	308	0,004	0,132	6,145	0,004	0,132	6,145
1-2500L	555	308	0,007	0,132	6,145	0,007	0,132	6,145
1-5000L	555	308	0,008	0,132	6,145	0,008	0,132	6,145
1-10000L	555	308	0,011	0,132	6,145	0,011	0,132	6,145
IDA2								
2-200L	241	144	0,002	0,074	1,517	0,001	0,074	0,67
2-1000L	683	348	0,004	0,097	2,319	0,004	0,097	2,319
2-2500L	555	308	0,007	0,132	6,145	0,007	0,132	6,145
2-5000L	559	311	0,01	0,134	5,895	0,01	0,134	5,895
2-10000L	769	385	0,008	0,102	8,718	0,008	0,115	8,04

- Cumulative Ductility Index (D_{ck}), which is based on cumulative plastic deformation (Castiglioni 2009). It takes into account the ductility μ_{si} at each cycle and is defined by:

$$D_{ck} = \sum_{i=1}^N (\mu_{si} - 1)^2 \quad (3)$$

where N represents the number of cycles and the parameter “a” ranges between 1.6 and 1.8 with a value of 1.6 for steel structures (Castiglioni 2009).

- Normalized Hysteretic Energy (NHE) (Castiglioni 2009) is given by Eq. 4:

$$NHE = \frac{E_H}{F_y \cdot u_y} \quad (4)$$

where E_H represents the total hysteretic energy, F_y is the material yield strength and u_y is the displacement obtained at F_y .

IDA of Single Degree-of-Freedom Structures

SDOF models were designed for the Ottawa Valley according with the 2005 NBCC (NRC 2005) with periods ranging from 0,1 to 3,0 s and a $R_d R_o$ value of 4,0. Those models were analysed with the NONLIN software to compare both IDA methods.

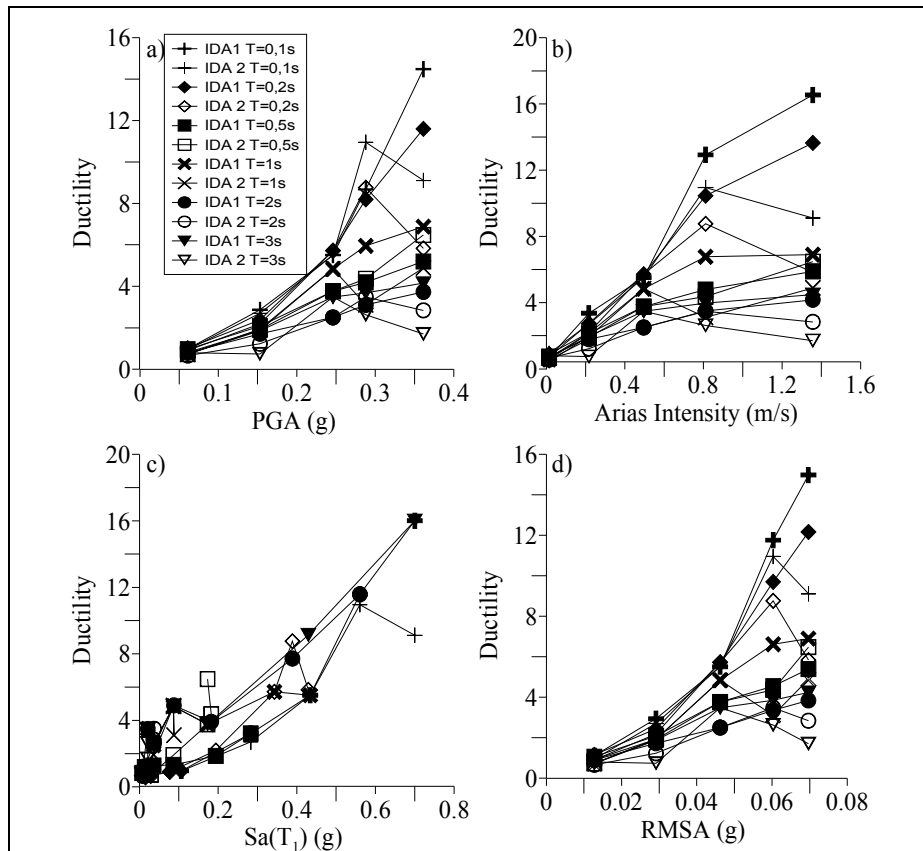


Figure 2. Ductility vs. IM for IDA 1 and IDA2: a) PGA, b) AI, c) $Sa(T_1)$, d) RMSA.

Several conclusions can be drawn from Fig. 2. The ductility demand increases as the return period increases, regardless of the IM parameter considered, because higher ground motion intensity levels result in larger peak displacements. Structures with shorter periods sustained higher ductility levels. The difference between the ductility demand from the IDA1 and IDA2 methods become more significant as IM is increased, i.e., when the ground motion return period increases. IDA1 tends to overestimate the structural demand for long return period events,

compared to IDA2. Note that the latter is considered more representative because all ground motion record characteristics are consistent with the corresponding return period.

For $S_a(T_1)$ based method, the differences between IDA1 and IDA2 results are very small for RP ranging from 200 to 5000 years. Differences only become significant for structures with short period (0,1 and 0,2 s) submitted to long return period events (10 000 years). For short period structures, the difference between the different spectral ordinates is very wide, which implies Scale Factor values greater than one to perform the IDA1 analysis. As each ground motion in the IDA2 method was spectrally matched, greater differences between IDA1 and IDA2 results are implied. For long period structures, however, the spectral ordinates are very close to each other and the factors used to scale the IDA1 record are approximately equal to one. As a consequence, the IDA1 and IDA2 ductility demand values that are obtained for long period structures are nearly identical. For the IDA2 analyses for a 10 000 years RP, ductility demand values are smaller than those obtained for a RP of 5000 years when structural period equals 0,1 s or 0,2 s. This is because a structure subjected to higher intensity ground motions is more likely to accumulate damage and a consequence of this degradation is period elongation (Vamvatsikos and Cornell 2002). The situation is not the same for the IDA1 analyses because only a few of the characteristics of the accelerogram are modified during the IDA1 scaling process. Several Damage Indices are compared in Fig. 3 for the IDA2 and PGA-based IDA1 methods. Large dispersion is observed for the residual displacements. The responses from the IDA1 and IDA2 methods for other damage indices (D_{ck} , NYE and NHE) are similar. The IDA1 method gives larger values of NHE and D_{ck} . The difference in damage indices increases with the PGA.

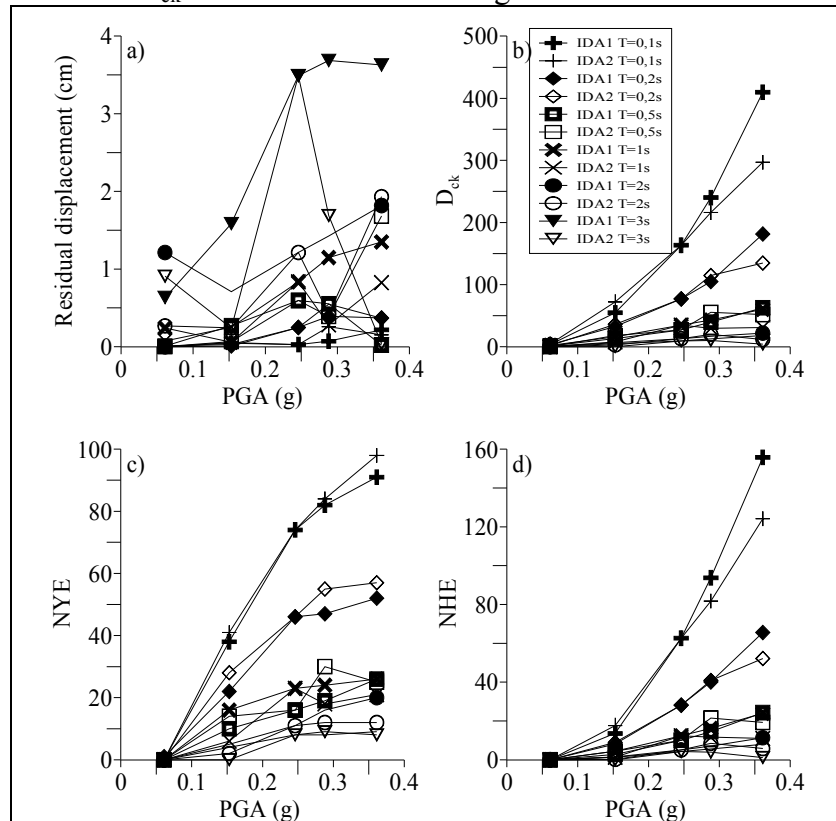


Figure 3. Damage indicators vs. PGA for IDA1 and IDA2: a) residual displ. (cm), b) Number of Yield Events, c) Cumulative Ductility index, d) Normalized Hysteretic Energy.

The analyses performed on SDOF structures show that it is useful to consider several damage indices to quantify seismic performance. They also tend to demonstrate that PGA and $S_a(T_1)$ based IDA, which are the most common in the literature, give consistent results with smaller dispersion for $S_a(T_1)$. Finally, the IDA1 method seems to slightly overestimate the structural damage, more specifically for the 5000 and 10 000 years seismic safety analyses. The IDA1 scaling process only affects some of the characteristics of an accelerogram. This has an effect on the different damage indices computed.

IDA of an Eight-story Braced Steel Frame

The eight-story steel frame (Fig. 4) has been designed accordingly to NBCC 1990 for Montreal (Kayayan 1994) with a force reduction factor equal to 4. This frame is three-bay wide and it was modified by introducing diagonal braces in the interior bay to reduce the fundamental period and develop inelastic behaviour within the structure for the different ENA accelerograms. The same structural systems are used to compare the IDA1 and IDA2 methods and avoid any bias. The analyses were performed using the RUAUMOKO program (Carr 2004).

In Table 3, the IDA1 method overestimates largely the ductility demand at every level for long return period events (5000 and 10 000 years). However, for short return period events, both methods give similar results. The different values found for the maximum roof displacement (Fig. 5) are in agreement with the previous observation (larger differences between IDA1 and IDA2 results as RP increases). Moreover, it appears clearly that a displacement peak is attained for the 2500 years return period event. For longer RP, maximum roof displacements decrease, which could be explained by the phenomenon of period elongation previously described for SDOF structures. The different conclusions that can be drawn from the analysis of the MDOF structure are similar to those obtained for SDOF systems, as structures submitted to IDA1 method undergo more damage.

Table 3. Maximum ductility values obtained in the bracing system for PGA-based IDA1 and IDA2.

	Storey							
	1	2	3	4	5	6	7	8
IDA1-PGA								
1-200L	0,90	0,56	0,71	0,71	0,86	0,74	0,98	0,93
1-1000L	2,23	1,61	2,10	2,27	2,84	2,35	3,18	3,44
1-2500L	5,12	3,91	4,98	5,36	6,56	5,57	6,29	7,48
1-5000L	5,16	3,94	5,02	5,40	6,61	5,62	6,48	7,52
1-10000L	5,00	3,80	4,86	5,23	6,40	5,43	6,27	7,34
IDA2								
2-200L	1,11	0,63	0,79	0,79	1,04	1,03	1,61	1,32
2-1000L	2,34	1,58	1,63	1,55	2,67	2,84	3,92	3,39
2-2500L	5,12	3,91	4,98	5,36	6,56	5,57	6,29	7,48
2-5000L	4,96	3,66	4,11	4,82	6,69	6,01	7,42	6,42
2-10000L	4,07	2,90	3,13	3,30	4,21	3,61	5,22	5,43

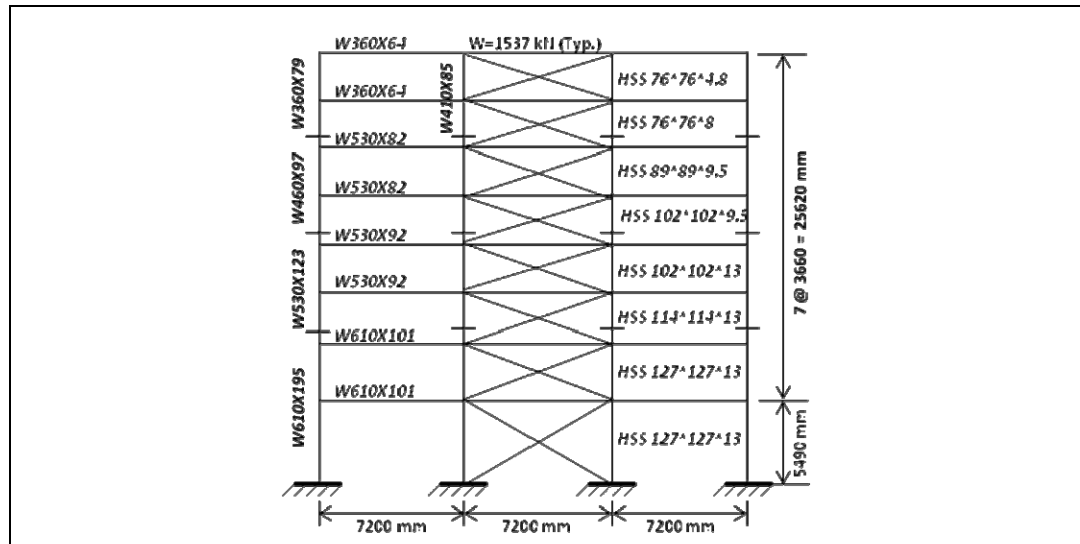


Figure 4. Dimensions and members selected for the eight story steel frame studied (W shapes for beams and columns, HSS shapes for diagonals).

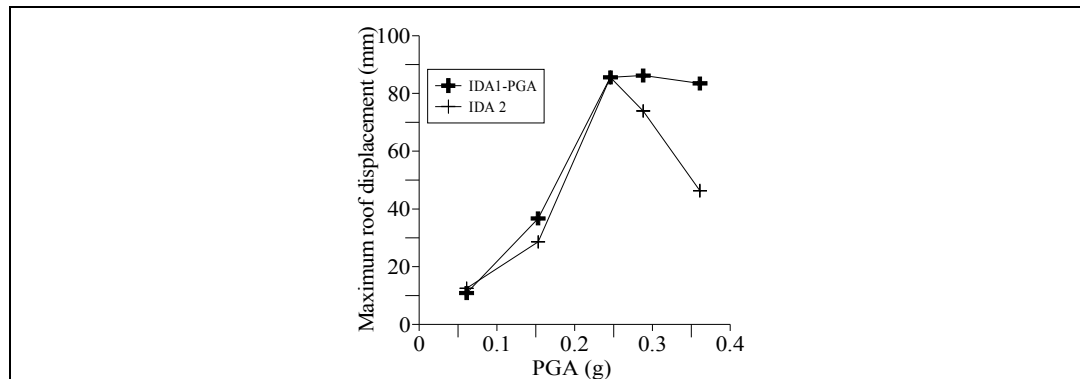


Figure 5. Maximum roof displacement vs PGA for IDA1 and IDA2.

Ground-Motion selection for ENA

For ENA, historical records are rare and simulated ground motions become particularly useful as previous studies have shown the importance of ground-motions selection for IDA methods (Vamvatsikos and Cornell 2002). The NBCC 2005 includes recommendations regarding the number of time histories to consider in dynamic analyses (the maximum response must be used for a three accelerogram set compared to the mean response if seven accelerograms are used). Four sets of three and seven independent ground motions were selected, using data provided by Atkinson (2009). Moment Magnitude M6 and M7 events with fault distances of 30 km and 70 km respectively were divided into four independent sets of three and seven accelerograms and scaled to the NBCC 2005 Montreal (Site C) response spectrum according to Atkinson (2009) for periods ranging from 0,1 s to 4,0 s. Response spectra (Fig. 6) and ductility demand computed for the different previously described SDOF structures (Fig. 7) are displayed for each time history.

Figure 7 shows that maximum ductility demand from a 3 accelerogram set is higher than the mean ductility from 7 accelerograms. The mean ductility from the seven accelerogram sets, which are more computationally intensive to use, gives more consistent results as the ductility demand approaches $R_d R_o$ value selected when the period is increased, which is in line with the principle of equal elastic and elasto-plastic displacement.

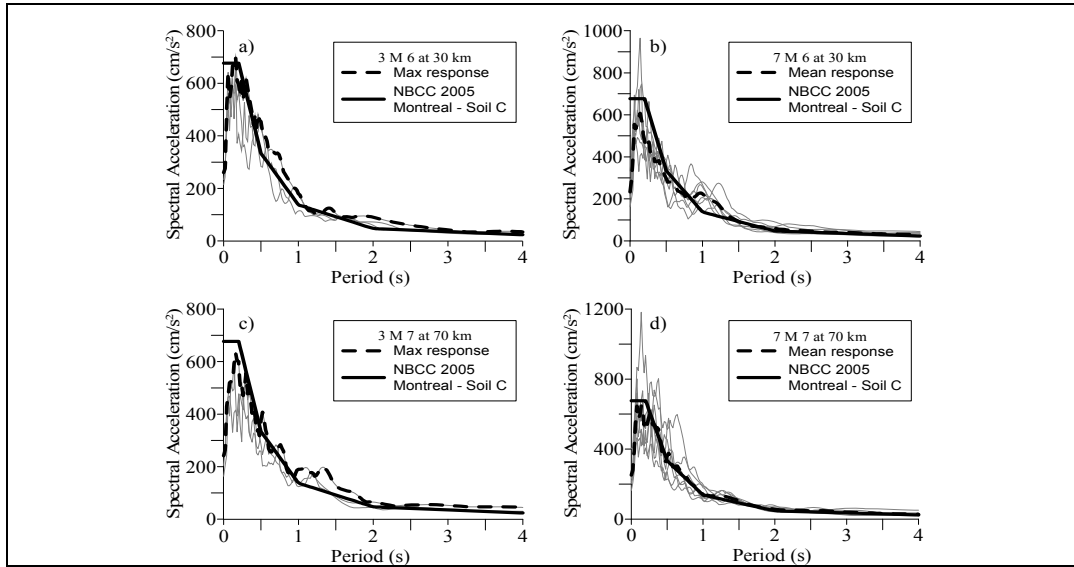


Figure 6. NBCC 2005 target spectrum and response spectra for each set: a) 3 M6, b) 7 M6, c) 3 M7, d) 7 M7.

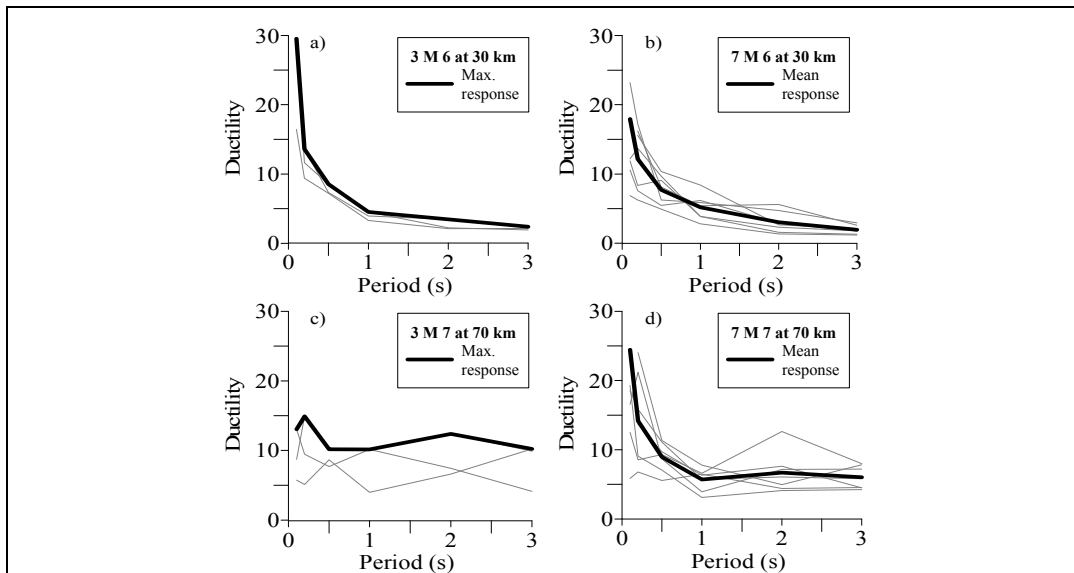


Figure 7. Ductility vs. period (s) for each set : a) 3 M6, b) 7 M6, c) 3 M7, d) 7 M7.

Conclusions

This paper presents an evaluation of the IDA robustness when the IDA1 and IDA2 methods are

scrutinized for different structures and different Intensity Measures. The following conclusions can be drawn from this study:

- For SDOF and MDOF nonlinear structures, the IDA1 and IDA2 methods give nearly identical results for short return period seismic events but the IDA1 method seems to gradually overestimate the structural response of short period structures when the return period is increased. The scaling process used in the IDA1 method (classical IDA) change only some of the accelerogram characteristics but not all (Number of Zero Crossing does not change). The analyses performed for larger RP events resulted in structural period elongation. This phenomenon was observed for both the IDA2 and IDA1 methods.
- $Sa(T_1, 5\%)$ and PGA, which are the most currently used ground motion Intensity Measures give consistent results, with less dispersion for $Sa(T_1, 5\%)$.
- The selection of ground motions must be made carefully as the envelope ductility demand computed from 3 accelerograms can give significantly larger values than the mean ductility value computed from 7 accelerogram sets.

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