EFFECTIVENESS OF EARTHQUAKE SELECTION AND SCALING METHOD IN NEW ZEALAND

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SUMMARY

In New Zealand, time history analysis is either the required or preferred method of assessing seismic demands for torsionally sensitive and other important structures, but the criteria adopted for the selection of ground motion records and their scaling to generate the seismic demand remains a contentious and debatable issue. In this paper, the scaling method based on the least squares fit of response spectra between 0.4-1.3 times the structure's first mode period as stipulated in the New Zealand Standard for Structural Design Actions: Earthquake Actions (NZS1170.5) [1] is compared with the scaling methods in which ground motion records are scaled to match the peak ground acceleration (PGA) and spectral acceleration response at the natural period of the structure corresponding to the first mode with 5% of critical damping; i.e. $S_a(T_1,5\%)$. Incremental dynamic analysis (IDA) is used to measure the record-to-record randomness of structural response, which is also a measure of the efficiency of the intensity measure (IM) used. Comparison of the dispersions of IDA curves with the three different IMs; namely PGA, $S_a(T_1,5\%)$ and NZS1170.5 based IM, shows that the NZS1170.5 scaling method is the most effective for a large suite of ground motions. Nevertheless, the use of only three randomly chosen ground motions as presently permitted by NZS1170.5 is found to give significantly low confidence in the predicted seismic demand. It is thus demonstrated that more records should be used to provide a robust estimate of likely seismic demands.

1. INTRODUCTION

Performance based earthquake engineering (PBEE) relies on structural performance being predicted with a known and acceptable level of confidence. Steps such as hazard analysis, demand prediction, damage modelling and loss estimation affect the prediction of ultimate performance [2-3]. This paper deals with the demand prediction aspect, focusing on the variation in structural response for a given suite of ground motion records. In PBEE, structural response is presented probabilistically in an intensity measure (IM) vs engineering demand parameter (EDP) domain. Significant variation in structural responses obtained through time history analyses using different ground motion records is evident even though these records may have been subjected to a rigorous selection procedure and scaled to the same intensity level. However, the extent of this record-to-record variation in IM-EDP relationship depends very much on the selection of parameters to be used as IM and EDP. The variation of structural response for a given seismic hazard level has been shown to match reasonably with a lognormal distribution [4-5]. Therefore, if the IM is chosen to significantly reduce the lognormal standard deviation (dispersion) of the responses, fewer records and hence fewer analyses can yield the same level of confidence in the predicted seismic demand.

Peak ground acceleration (PGA) and spectral acceleration at the natural period of structure; i.e. $S_a(T_1)$, are commonly used as IM because they are either readily available or easily computable. The effectiveness of an IM is discussed in terms of its 'sufficiency' and 'efficiency'. An IM must be 'sufficient'; i.e. the structural response at a constant value of the selected IM must be independent of seismological parameters, such as earthquake magnitude and source-to-site distance. An efficient IM will reduce the variability of structural response. For most cases, $S_a(T_1)$ has been identified to be more efficient than PGA [4]. However, even $S_a(T_1)$ has been found short in 'sufficiency' and 'efficiency' in some special cases such as tall buildings where higher order modes may play a significant role in the overall response [6-8]. A similar situation arises in the case of soft-soil or near-source ground motion when the dominant frequency of the ground motion is likely to be significantly higher than the first mode frequency of a structure. The quest to overcome these shortcomings has led to investigations of other forms of scalar IMs [9-10] and vector IMs [7,11].

In contrast, the search of a more efficient EDP which correlates better with damage, although acknowledged to be equally important, has found significantly less attention from researchers. The most common EDP used by researchers to deal with structural damage is the absolute maximum interstory drift and the peak floor acceleration has been unanimously accepted as the EDP to correlate better with non-structural content damage. Apart from these, only the maximum value of the average drift of a building [12] and the average of the positive and negative interstory drift peaks [13] have been investigated as EDPs (for structural damage purpose) in the authors' knowledge.

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In this paper, using the absolute maximum interstory drift as EDP, the efficiency of three different IMs including the one recommended in the New Zealand Standard for Structural design actions: Part 5: Earthquake actions (NZS1170.5:2004) [1] is scrutinised. In New Zealand, time history analysis can be used for seismic design of all structures and is compulsory for designing torsionally sensitive structures. NZS1170.5 [1] prescribes that time history analysis shall be conducted with at least three ground motion records and the most severe demand be used in design. Each of these three ground motion records are scaled to match the design response spectrum for the target limit state and the location of the structure to be designed. The ground motions are to be selected from actual records (wherever possible) that have seismological features similar to the target design spectrum of the site. The selected records are scaled using two factors; a record scale factor k_1 and a family scale factor k_2. The record scale factor k_1 is chosen to minimise in a least squares sense the logarithm of the ratio of spectral accelerations of the scaled record spectrum to the target design spectrum over a range of period between 0.4-1.3 times the structure's first mode period T_1. The family factor k_2 is decided such that the energy content of at least one record in the family exceeds that of the design spectrum over the target period range. No ground motion records with k_1 factor outside the allowable range of 0.33-3 are permitted. Moreover, if the structure is within 20 km of a fault, then one of the three records needs to exhibit forward directivity (velocity pulse) component. The effectiveness of NZS1170.5 method of selection and scaling of ground motion records has not yet been investigated, and hence it is very timely that a study aiming to investigate the effectiveness of this approach be conducted and reported, at least in New Zealand, to ensure confidence of New Zealand designers in their design practice.

Hence, this paper compares the efficiency of three IMs namely; PGA, $S_a(T_1)$ with 5% of critical damping; i.e. $S_a(T_1,5\%)$, and the least squares fit in the period range of $0.4T_1$-$1.3T_1$ as recommended by NZS1170.5 [1] (referred to as NZS1170.5 IM hereafter). PGA based scaling is the simplest form of scaling ground motion records. Studies have shown that it produces relatively large dispersion of responses except for structures with small natural periods [14]. On the other hand, $S_a(T_1,5\%)$ produces lower dispersion of responses but it requires more effort as response spectra need to be generated prior to scaling the spectral acceleration (S_a) ordinate at the fundamental period. Least squares fit of S_a over a range of period as required by NZS1170.5 IM, which is expected to result in further lower levels of dispersion, is apparently the most cumbersome form of scaling records. The best choice of IM is disputable as a balance needs to be struck between simplicity and effectiveness in reducing record-to-record variation [2]. However, slightly increased difficulty in determining the scaling factor (which could be automated in a spreadsheet) is a far more appealing proposition than performing significantly more number of time history analyses with easy-to-scale but less efficient IMs.

2. GENERAL METHODOLOGY

The overall process of investigating the effect of various IMs on the uncertainty in structural response has been divided into the following two steps for convenience:

Step 1: Conduct Incremental Dynamic Analysis (IDA)

This involves subjecting a structural model to a suite of ground motion records scaled to a range of *im* (used hereafter to indicate the values of IM). This is analogous to increasing levels of force in pushover analysis. However, IDA provides a better indication of structural response as actual ground motion records are utilised. The *edp* (used hereafter to indicate the values of EDP) is noted at each *im* represented by a scaled record. Each time history analysis thus gives a pair of data (*edp*, *im*) which defines a point in the IM-EDP domain. Joining such points obtained from the analyses using all scaled records results in the IDA curve for that ground motion record. This curve is usually characterised by a linear elastic region, followed by a transitional phase leading to a flat line indicating collapse [15]. IDA curves are generated here for all records in the suite using the three different IMs.

Step 2: Measure and compare the dispersion of responses

From the IDA curves, several *edp* data can be extracted for any value of *im*. Using lognormal distribution to represent the variation of the *edp* data at a given *im*, the lognormal standard derivation of the *edp*s at the required *im* is calculated. Repeating this for different *im* levels, the variation of lognormal standard deviation with respect to *im* is plotted. This is repeated for all three IMs to provide a comparison of their efficiency in reducing the record-to-record variation in structural response.

3. INCREMENTAL DYNAMIC ANALYSIS (IDA)

IDA is an inelastic time history based analysis procedure that offers a relatively accurate prediction of seismic demand and capacity [14]. An inelastic time history analysis program is required for IDA, which can be conducted by following the procedure summarised below:

1. Create a computational model of the structure in an inelastic dynamic analysis program.
2. Select a sufficient number of appropriate ground motion records. For this study, a suite of 20 ground motion records is used.
3. Choose an IM and scale the ground motion records to intensities ranging from a small *im* that produces an elastic response to an *im* large enough to cause collapse.
4. Choose an EDP that represents a critical/maximum response and has a reasonable correlation with damage. In this case, maximum absolute drift angle is used.
5. Conduct time history analysis with the scaled ground motions. This means conducting as many as 20 time history analyses for each ground motion record. Extract the *edp* from the output of each analysis.
6. Locate the (*edp*, *im*) points from all analysis in the IM vs EDP plot and join these points to obtain an IDA curve for a ground motion record.
7. Scale all ground motion records and generate IDA curves for these records; i.e. repeat steps 5 and 6 for all ground motion records in the selected suite.

In this paper, inelastic dynamic analysis was conducted using the program RUAUMOKO 2D [16]. The batch file analysis mode was used extensively as a total of 5600 runs (excluding those for sensitivity analysis) had to be conducted. Analyses were conducted in batches of 400 corresponding to 20 levels of appropriately scaled *im* for all 20 records. This was repeated for 14 single degree of freedom (SDOF) systems with periods ranging from 0.3sec to 2.0sec. Records were scaled to give a range of *im* encompassing hazard levels corresponding to all limit states. The maximum *im* was intentionally set high to ensure that most records caused collapse of the structure. More efficient use of computing resources could be achieved through the use of algorithms that scale a record to capture the entire range of behaviour from elasticity to collapse. Only 12 scaling steps were found to be sufficient when using hunt and fill tracing algorithms [15]. Automatic extraction of results proved to be efficient and the potential for error was also reduced. Each batch of 400 runs required 20 minutes on a 1.6GHz processor computer. However, it should be noted that multi degree of

Table 1: Ground motion records

ID	Event	Year	Station	ϕ*	M*2	R*3 (km)	PGA (g)
Aa	Loma Prieta	1989	Agnews State Hospital	90	6.9	28.2	0.159
Bb	Imperial Valley	1979	Plaster City	135	6.5	31.7	0.057
Cc	Loma Prieta	1989	Hollister Diff. Array	255	6.9	25.8	0.279
Dd	Loma Prieta	1989	Anderson Dam	270	6.9	21.4	0.244
Ee	Loma Prieta	1989	Coyote Lake Dam	285	6.5	22.3	0.179
Ff	Imperial Valley	1979	Cucapah	85	6.9	23.6	0.309
Gg	Loma Prieta	1989	Sunnyvale Colton Ave	270	6.9	28.8	0.207
Hh	Imperial Valley	1979	El Centro Array #13	140	6.5	21.9	0.117
Jj	Imperial Valley	1979	Westmoreland Fire Sta.	90	6.5	15.1	0.074
Kk	Loma Prieta	1989	Hollister South & Pine	0	6.9	28.8	0.371
Mm	Loma Prieta	1989	Sunnyvale Colton Ave	360	6.9	28.8	0.209
Nn	Superstition Hills	1987	Wildlife Liquefaction Array	90	6.7	24.4	0.180
Pp	Imperial Valley	1979	Chihuahua	282	6.5	28.7	0.254
Qq	Imperial Valley	1979	El Centro Array #13	230	6.5	21.9	0.139
Rr	Imperial Valley	1979	Westmoreland Fire Sta.	180	6.5	15.1	0.110
Ss	Loma Prieta	1989	WAHO	0	6.9	16.9	0.370
Tt	Superstition Hills	1987	Wildlife Liquefaction Array	360	6.7	24.4	0.200
Uu	Imperial Valley	1979	Plaster City	45	6.5	31.7	0.042
Vv	Loma Prieta	1989	Hollister Diff. Array	165	6.9	25.8	0.269
Ww	Loma Prieta	1989	WAHO	90	6.9	16.9	0.638

[1] Component, [2] Moment Magnitudes, [3] Closest Distances to Fault Rupture, and Source: PEER Strong Motion Database, http://peer.berkeley.edu/smcat/

freedom (MDOF) models will take substantially longer processing times and hunt and fill algorithms may need to be used to reduce the processing time. Initially, PGA based scaling was used to conduct IDA and its result was post-processed [17] to derive IDA curves with $S_a(T_1,5\%)$ and NZS1170.5 based IM.

3.1 Ground motion record Selection

Twenty ground motion records, as shown in Table 1, were sourced from the Pacific Earthquake Engineering Research (PEER) Centre's strong motion database for this study to represent the typical range of possible earthquake scenarios. Following the current practice, the ground motions were chosen based on magnitude, distance from the nearby fault, and site conditions. These ground motions were recorded at 15-32 km from the closest point of the fault rupture and do not exhibit directivity effects. Magnitudes of these earthquakes vary from 6.5 to 6.9 and these records are from firm soil locations corresponding to USGS soil class C or D or NZS1170.5 [1] class C shallow soils.

By outlining the statistical variability of these 20 records, a more accurate measure of seismic demand imposed on the structure will be provided. The seismic response of structures subjected to a suite of records is herein assumed to have a lognormal distribution [4,5]. In order to estimate the median *edp* within a fraction (X), the number of records (n) required can be approximated using $n = 4.0\ \sigma^2 / X^2$ where σ is the lognormal standard deviation of the *edp*s for a given value of *im* [18]. For the maximum interstory ductility of a MDOF structure dominated by the first mode response, a maximum value of 0.62 was recorded by Shome et al. [4] for the lognormal standard deviation σ. Consequently, the number of records required to estimate the median response to ± 25% is $4.0 \times 0.62^2 / 0.25^2$ • 25 records. However, use of additional records will result in lower levels of margin of error as the error is inversely proportional to the square root of the number records used. Hence, reducing the error associated with 20 records by half will require 80 records. It should also

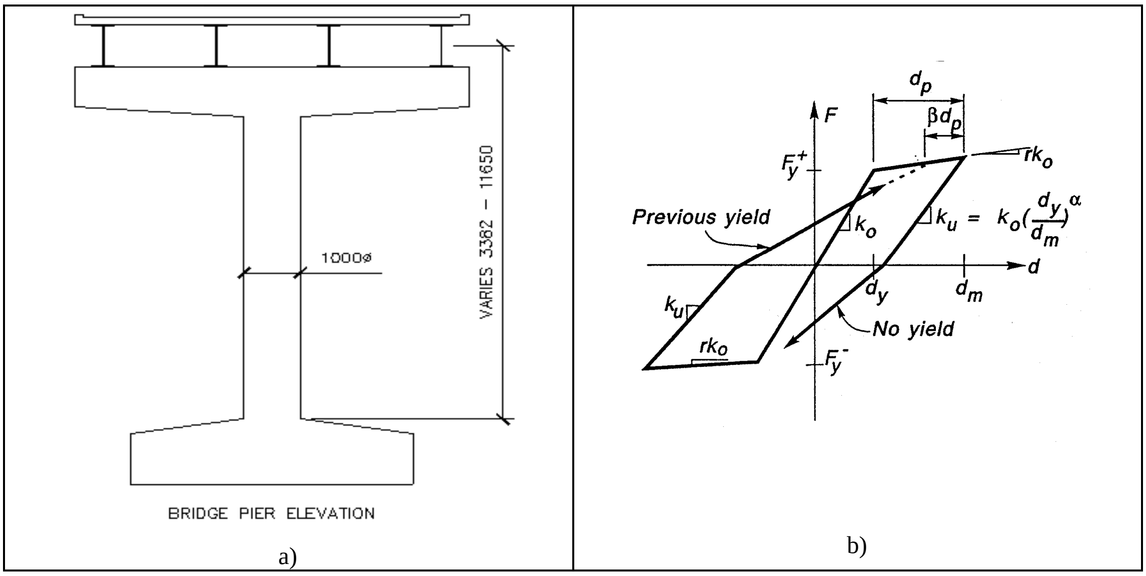

Figure 1: (a) Bridge pier elevation; and (b)Takeda hysteresis model

be noted that the use of a more efficient IM producing lower levels of dispersion should allow for fewer records to be used in the analysis.

3.2 Structural Model

A one meter diameter reinforced concrete bridge pier as shown in Figure 1a was modelled as an SDOF system for conducting the IDA. This SDOF model was chosen for its simplicity and its ability to provide an adequate representation of structures dominated by the first mode response. Also by using an SDOF model, the effect of natural period on the structural response can be investigated by varying a single parameter - in this case height. The bridge pier used here is assumed to support a combined dead and live load of 2000 kN, corresponding to 8% of its axial capacity (i.e. 0.08 f_c'A_g). As shown in Figure 1b, Takeda hysteresis loop with unloading factor of 0.3 and reloading factor of 0.5 was used to represent the nonlinear cyclic force-deformation relationship of the SDOF system. Moreover, strength and stiffness degradation in the inelastic response phase is also accounted for in this hysteresis model. Viscous damping equal to 5% of the critical was assigned. For the same model, different natural first mode periods ranging from 0.3 sec to 2.0 sec were obtained by modifying the height while maintaining the diameter and loading. While doing so, the strength was not varied. Obviously, this affected the ductility capacity of the piers. Nevertheless, as the study compares the efficiency of different IMs which does not depend on the inelastic deformability of the piers with different periods, this difference is overlooked in the results and discussions.

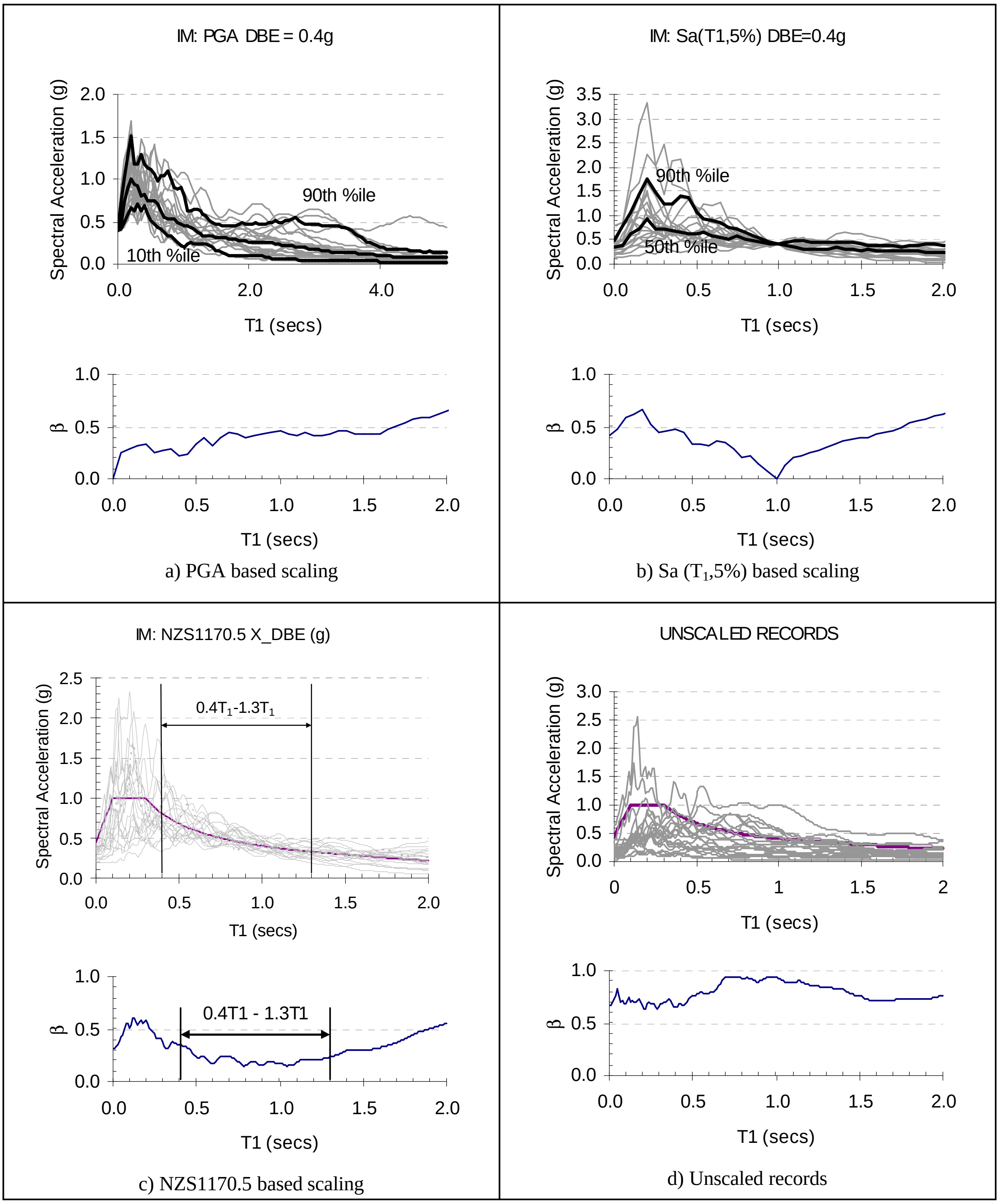

Figure 2: Elastic response spectra

4. EFFECTIVENESS OF INTENSITY MEASURES

As mentioned earlier, appropriate choice of IM (likewise with EDP) affects the dispersion of the IDA curves. More effective IMs result in lower levels of variability of *edp*; and hence provide greater confidence in the demand obtained by using the same number of ground motion records. As shown in Figure 2, three IMs; namely PGA, $S_a(T_1,5\%)$ and NZS1170.5 based IM which takes into account the spectral accelerations within a range of periods between $0.4T_1$ and $1.3T_1$ where T_1 is the period of the first mode response of the structure, are used to derive IDA curves. This enables the comparison of the effectiveness of these three IMs over the range of periods analysed.

As the variation of *edp* given *im* has been shown to conform closely to a lognormal distribution [4,5], the lognormal standard deviation (i.e. dispersion) of the *edp*s for a given *im* measures the efficiency of the IM used. For the three IMs used in this study, dispersions are calculated and compared. For the first IM used in the study (i.e. PGA), the design basis earthquake (DBE) was considered to be 0.4g which corresponds to the PGA of an earthquake that has a 10% chance of occurring in 50 years (i.e. return period of 475 years) for the design location (i.e. Wellington). The 20 original ground motion records were scaled to 0.4g PGA (i.e. DBE) and the corresponding elastic response spectra are compared in Figure 2a, which shows the variation of lognormal standard deviations of S_a at different first mode natural periods. Lognormal standard deviations (of S_a) and the response spectra for the unscaled records are also outlined in Figure 2d for comparison with the other forms of scaling.

Similar scaling was conducted for $S_a(T_1,5\%)$ after generating the elastic response spectra with 5% damping for each of the 20 original ground motion records. $S_a(T_1,5\%)$ based scaling involves generating the elastic acceleration response spectrum of a record with 5% of critical damping and scaling the record to yield a constant value of S_a at the first mode period T_1. Hence, response spectra of all records were scaled at the first mode period of the structure (used as 1 second for this figure) to the S_a corresponding to the DBE. Following the commonly used period-dependent interrelationships between PGA and S_a for different ranges of the design response spectrum, the S_a values at DBE (and any other hazard intensity for that matter) can be calculated for a structure with a given period when the corresponding PGA values are known. As $S_a(T_1,5\%)$ based scaling is conducted at a single period corresponding to the natural first mode period calculated based on elastic stiffness, it fails to capture higher order response modes and period elongation (due to softening of structure) effects. As shown in Figure 2b, the elastic response spectra for the structural model with the first mode period of 1.0 sec show a large variation in S_a ordinates at smaller and larger periods. Consequently, the lognormal standard deviation plot for $S_a(T_1,5\%)$ based scaling outlines a trough with zero dispersion at T_1 and rapidly increasing dispersion levels away from T_1.

Next, NZS1170.5 IM based scaling is conducted by performing least squares fitting to the logarithms of S_a over the period range of $0.4T_1$ – $1.3T_1$ for the 20 records. NZS1170.5 IM based scaling, which is done not only at the first mode period T_1 but over a range of periods, is intended to account for higher mode effects and decrease of stiffness (i.e. softening) in the inelastic phase of the structural response. In other words, including responses over a period range covering both sides of T_1 in the scaling method accounts for period elongation as the structural response is forced into the inelastic range (due to softening) as well as period shortening due to participation of higher frequency modes. As shown in Figure 2c, this results in a relatively low level of dispersion, which fluctuates around • = 0.2 over the abovementioned period range.

A study addressing the issue of selection based on principal seismic characteristics and scaling have shown that there may not be a need for careful site specific process for record selection by magnitude, distance and scaling [19]. However, it is still believed that factors such as the type of faulting, soil type and velocity pulses associated with near field effects would need to be considered prior to selecting ground motion records. In fact, it is often argued that scaling should be minimised as much as possible by using ground motion records with response spectra that match the elastic design spectrum as closely as possible. To this end, ground motion records are being categorised to enable ease of selection for given locations. Accordingly, NZS1170.5 [1] also recommends avoiding very large and very small scaling factors by eliminating records that need to be scaled by a factor $k_1<0.33$ or $k_1>3$. An addition, records that did not provide a good fit after scaling to the target spectra were also eliminated. This resulted in 5 records being rejected from the original suite of 20 records.

5. RESULTS: EFFECT OF IM IN IDA CURVES

Figure 3 shows IDA curves (IM vs EDP plots) for the structural model with 0.8 sec natural period. The three IDA curves are for PGA, $S_a(T_1,5\%)$ and NZS1170.5 IM, respectively. Note that the vertical axis is normalised with respect to the DBE; i.e. a normalised value of 1 represents the DBE. Note that the maximum considered earthquake (MCE) with 2% chance of occurring in 50 years has approximately 0.8g PGA for the assumed location. Hence, a value of 2 for the normalised intensity scale in the vertical axis represents the MCE (i.e. 0.8g/0.4g = 2). Normalisation of the results is done to allow for a meaningful comparison of IDA results. While $S_a(T_1,5\%)$ is almost equal to the PGA at 1 sec period based on S_a=PGA/T which is valid in the constant velocity range of elastic response spectra that normally encompasses 1 sec, the same is not true for other periods. For instance, as shown in Figure 4, in the case of the 0.5 sec period structural model the S_a ordinate equals 0.68g, but it corresponds to a PGA (i.e. S_a at 0 sec) of 0.4g. Thus comparing IDA results from a record scaled to $S_a(0.5,5\%)$ = 0.4g and the same record scaled to PGA = 0.4g will produce erroneous conclusions. Values of S_a are normalised with respect to the S_a at the first mode period (T_1) and are represented as fractions and multiples of DBE; e.g. for the 0.5 second period structure $S_a(0.5,5\%)$ = 0.68g is nominated as DBE whereas 0.4g represents DBE for PGA based IM.

In addition to the IDA curves for the 20 ground motion records, Figure 3 also plots the 10^{th}, 50^{th} (median), and 90^{th} percentile IDA curves drawn based on the true variation of *edp*s at each level of *im*. The difference in the 10^{th} and 90^{th} percentile IDA curves is representative of the variability in *edp*. PGA based scaling shows the greatest difference at DBE (0.4g) between the 10^{th} and 90^{th} percentile curves. The 10^{th} and 90^{th} percentile PGA based IDA curves give *edp*s of 0.84% and 3.73%, respectively at *im*=DBE; i.e. a ratio of 4.4 between the 90^{th} and 10^{th} percentile *edp*s. On the other hand, $S_a(T_1,5\%)$-based IDA curves give 0.89% and 2.66% *edp*s corresponding to the 10^{th} and 90^{th} percentile response (i.e. a ratio of approximately 3). Similarly, the ratio of the 90^{th} to 10^{th} percentile *edp*s at *im*=DBE for the NZS1170.5 IM based IDA curves is 2.5. This simple comparison indicates that the hierarchy of the three IMs based on their capability to reduce record-to-record variation in structural response (i.e. efficiency) is: (i) NZS1170.5 based IM; (ii) $S_a(T_1,5\%)$; (iii) PGA.

It is normally acknowledged that the lognormal standard deviation of the *edp*s at different *im* levels provides the best indication of the efficiency of the IM used. Figure 5 plots the lognormal standard deviations (dispersions) of the three IMs to compare their efficiencies. Note that there are four curves, one each for the three IMs and the fourth represents the case where three ground motion records are randomly selected (and scaled) according to the current NZ design practice. At DBE, the PGA based scaling shows a larger dispersion in comparison to $S_a(T_1,5\%)$ and NZS1170.5 IM based scaling. This is in line with the differences in the 10th and 90th percentile *edp*s at *im*=DBE obtained from the IDA curves

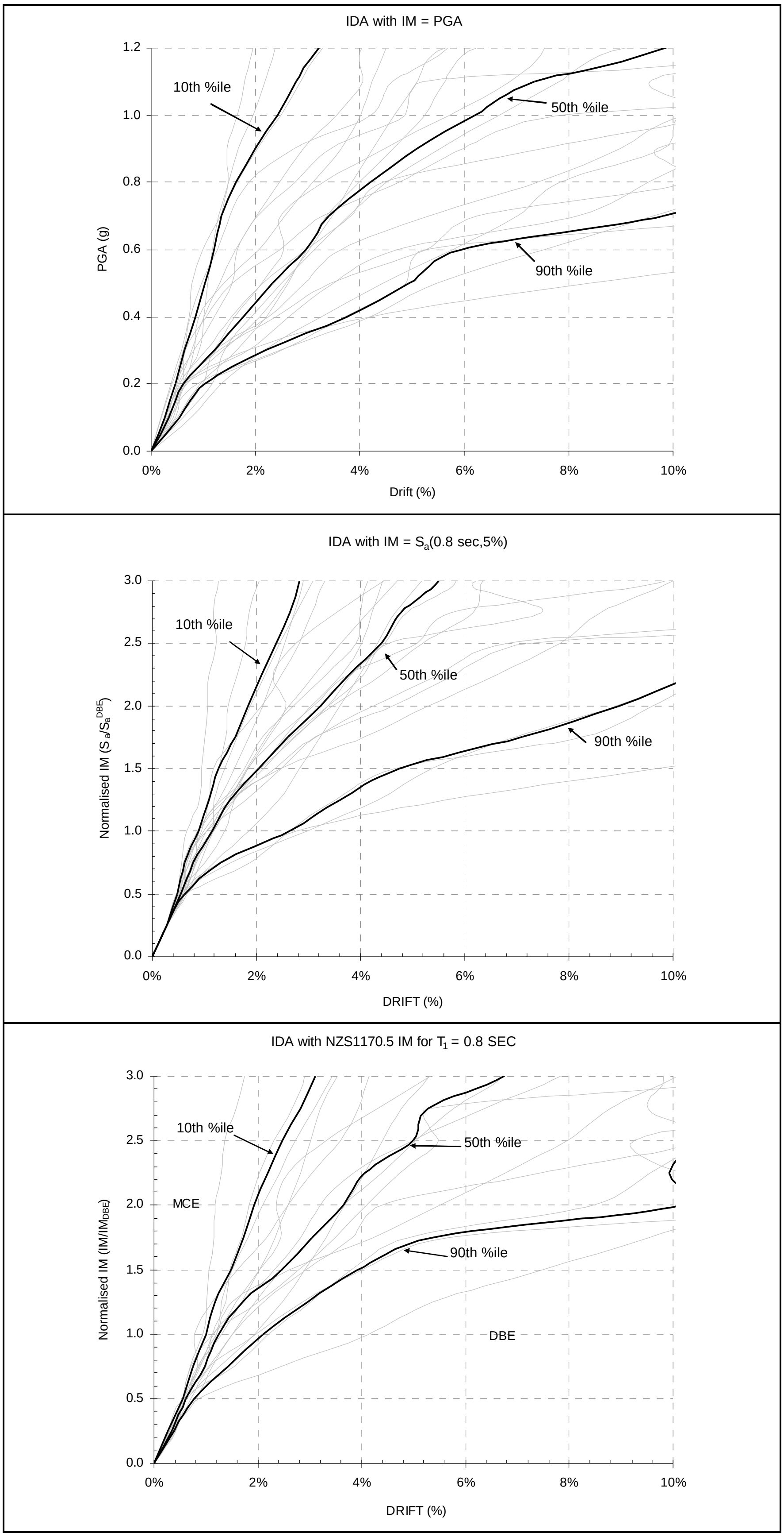

Figure 3: IDA curves for 0.8 sec period structure with different IMs: a) PGA; b) $S_a(T_1,5\%)$; c) NZS1170.5 IM

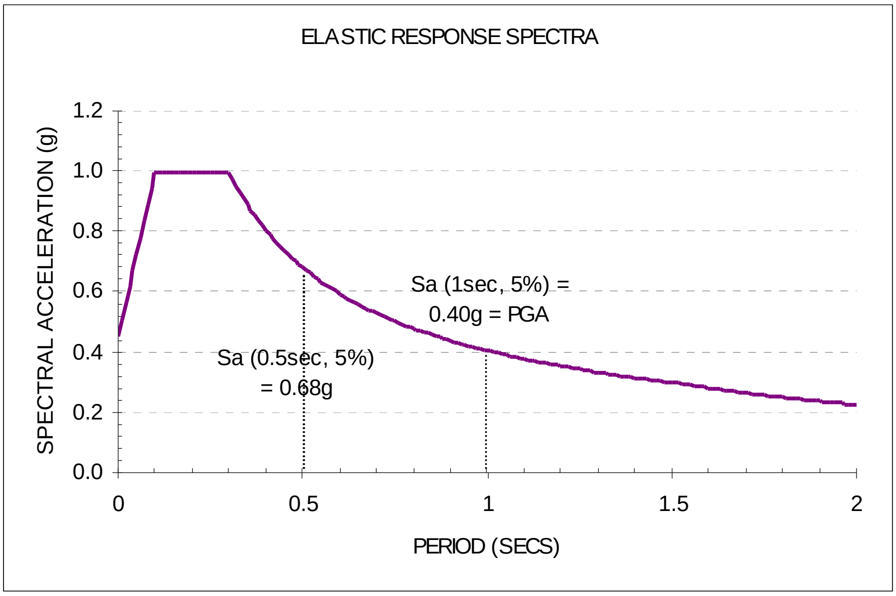

Figure 4: Normalisation of IM

generated by using the three different IMs. Interestingly, the dispersion profiles indicate little difference between the $S_a(T_1,5\%)$ and NZS1170.5 IM based scaling for this structure.

To investigate further the relative merits of these two IMs, IDA was conducted for structural models with different natural periods. Figure 6 compares the lognormal standard deviations of the three different IMs for structural models with natural period of 0.5, 1, 1.5, and 2 sec. As can be observed in all four plots, PGA is shown to consistently produce the highest level of dispersion in all structural models. Moreover, $S_a(T_1,5\%)$ exhibits relatively small levels of lognormal standard deviation for *im*<DBE range, and appears to be the best IM in terms of effectiveness up to approximately *im*=0.5DBE. This is because as long as the response is elastic (which is the case with smaller *im*), the response of SDOF system (where higher frequency modes do not exist) is completely characterised by the elastic stiffness that controls the natural period, which is the basis of $S_a(T_1,5\%)$ based scaling.

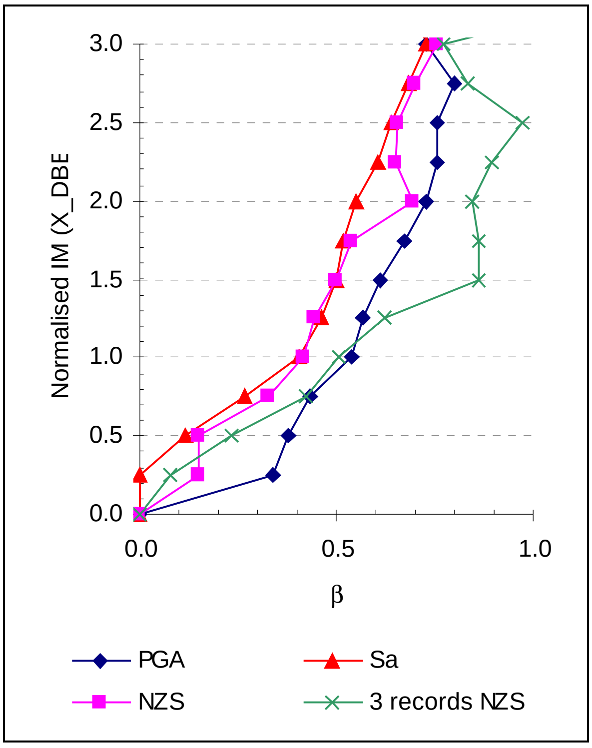

Figure 5: Dispersion for T_1 = 0.8 sec

Also understandably, $S_a(T_1,5\%)$ proved to be as efficient as (if not more than) the NZS1170.5 IM for the 0.5 sec period structure because the extent of inelastic response in the *im* range considered for this stiff structure is less likely to be significant. Expectedly, as the *im* increased the softening due to inelastic effects (which are taken into account in the analyses) became more prominent, and the dispersion associated with $S_a(T_1,5\%)$ increased noticeably. On the other hand, NZS1170.5 IM based scaling is found to produce relatively low levels of dispersion in large *im* ranges as well. However, when using only 3 randomly selected ground motion records as stipulated in NZS1170.5 [1], the variation of the dispersion in various structural models is apparently inconsistent and unpredictable, indicating that this may not be an appropriate option. Designers could certainly have more confidence on the outcome if they use a suite of some 20 records instead. Note that once a reliable computational model is developed and verified, little additional time is needed to perform further analysis with additional records.

As outlined earlier, lower levels of dispersion provide higher confidence in the structural response. Take for instance the structural model with 1 sec natural period shown in Figure 6b. The lognormal standard deviations (β) at *im*=DBE for PGA, $S_a(T_1,5\%)$ and NZS1170.5 IM are 0.63, 0.46 and 0.42, respectively. Based on these values, the confidence in the median response can be approximated using $n = 4.0\ \beta^2 / X^2$. Therefore, the number of records required to estimate the median *edp* within a factor X (say 0.1; i.e. ±10%) is 159, 85 and 71 for PGA, $S_a(T_1,5\%)$ and NZS1170.5 IM, respectively. The corresponding standard errors of estimation as percentage of the median can be calculated using β×100/• n, which results in 14.1%, 10.3% and 9.4% for PGA, $S_a(T_1,5\%)$ and NZS1170.5 IM, respectively. Therefore, it is important to use IMs that produce lower levels of dispersion in the response predicted using different ground motion records.

Following yet another approach of comparing the effectiveness of different IMs, Figure 7 compares the number of records required to estimate median *edp* response within a factor of ±20%. In addition to the four SDOF periods discussed in Figure 6, Figure 7 includes one more case (with

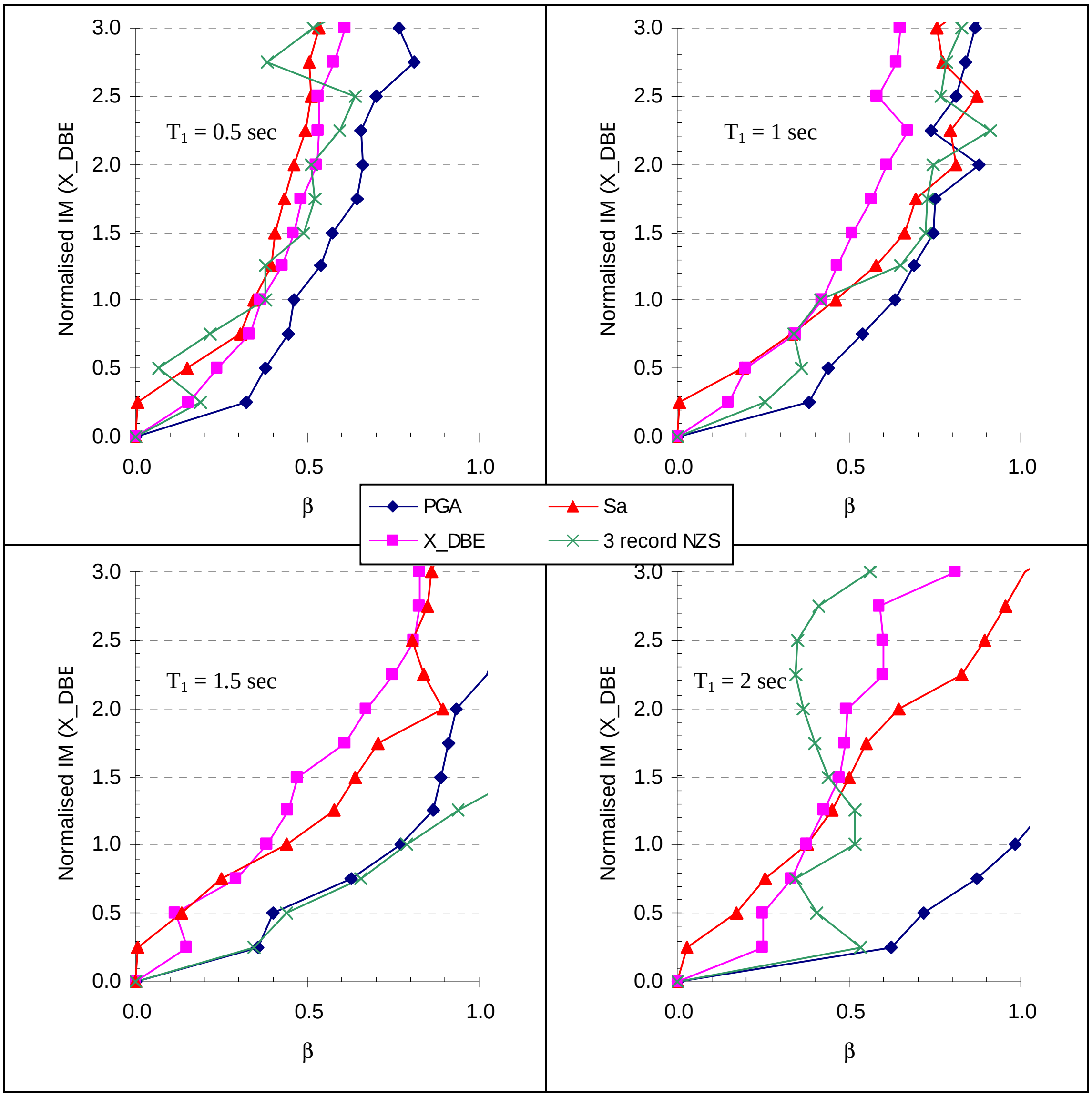

Figure 6: Comparisons of dispersions for structures with 0.5, 1.0, 1.5 and 2. 0 sec natural periods

T_1 = 0.3 sec) which represents a very stiff structure. Interestingly, but perhaps not surprisingly, Figure 7a shows that PGA based scaling required the fewest records to predict median response for the T_1 = 0.3 sec period structure, which indicates that PGA is the best IM for such short period structures. To be more specific, using PGA based scaling requires 18 and 34 records to predict the median response within a factor of ±20% at *im*=DBE and *im*=MCE, respectively, whereas the numbers are 33 and 42 for $S_a(T_1,5\%)$ and 39 and 53 for NZS1170.5 IM, respectively. This is because this structure is so rigid that the response is close to the applied ground motion itself and as the scaling of PGA hence also means scaling of the peak response.

For larger structural periods though, PGA based scaling is found to require invariably the most number of records to generate the same level of confidence in the outcome. To be more blunt, the results suggest that PGA based scaling should not be used in case of longer period structures. Agreeing with Figure 6a, Figure 7b also shows that $S_a(T_1,5\%)$ is the most effective IM for the 0.5 sec period structure. For structures with 1.0 sec and longer period, the NZS1170.5 IM proves to be the most effective at reducing the dispersion of structural response, which is reflected by the consistently lowest number of ground motion records required to predict the median response. Furthermore, Figure 7 also highlights the need for a greater number of records to predict median response with the same level of confidence at MCE compared to that at DBE. This is because the structural response at MCE is invariably more inelastic than at DBE and none of the three IMs explicitly use inelastic spectra for scaling the records.

Although Figures 6 and 7 are plotted based on analyses of SDOF systems that deform in only one mode, these figures can be interpreted to qualitatively extrapolate the effect of higher frequency modes in the overall response of MDOF systems. Typically, the higher frequency modes are of concern in tall buildings, whose fundamental periods are in the range of a few seconds and whose higher order modes are closer to the predominant period of ground motions (say around 1 sec). Using information in Figures 6 and 7, one can clearly see that the overall response contributed mainly by the fundamental mode (T_1>>2 sec) and significantly by the higher order modes (T_n~1 sec) will be predicted more efficiently by NZS1170.5 IM than by $S_a(T_1,5\%)$.

6. NZS1170.5 PROVISIONS: NUMBER OF RECORDS AND EFFECTIVENESS OF THE K_2 FACTOR

As mentioned above, NZS1170.5 [1] recommends at least three records be used for estimating the seismic demand if

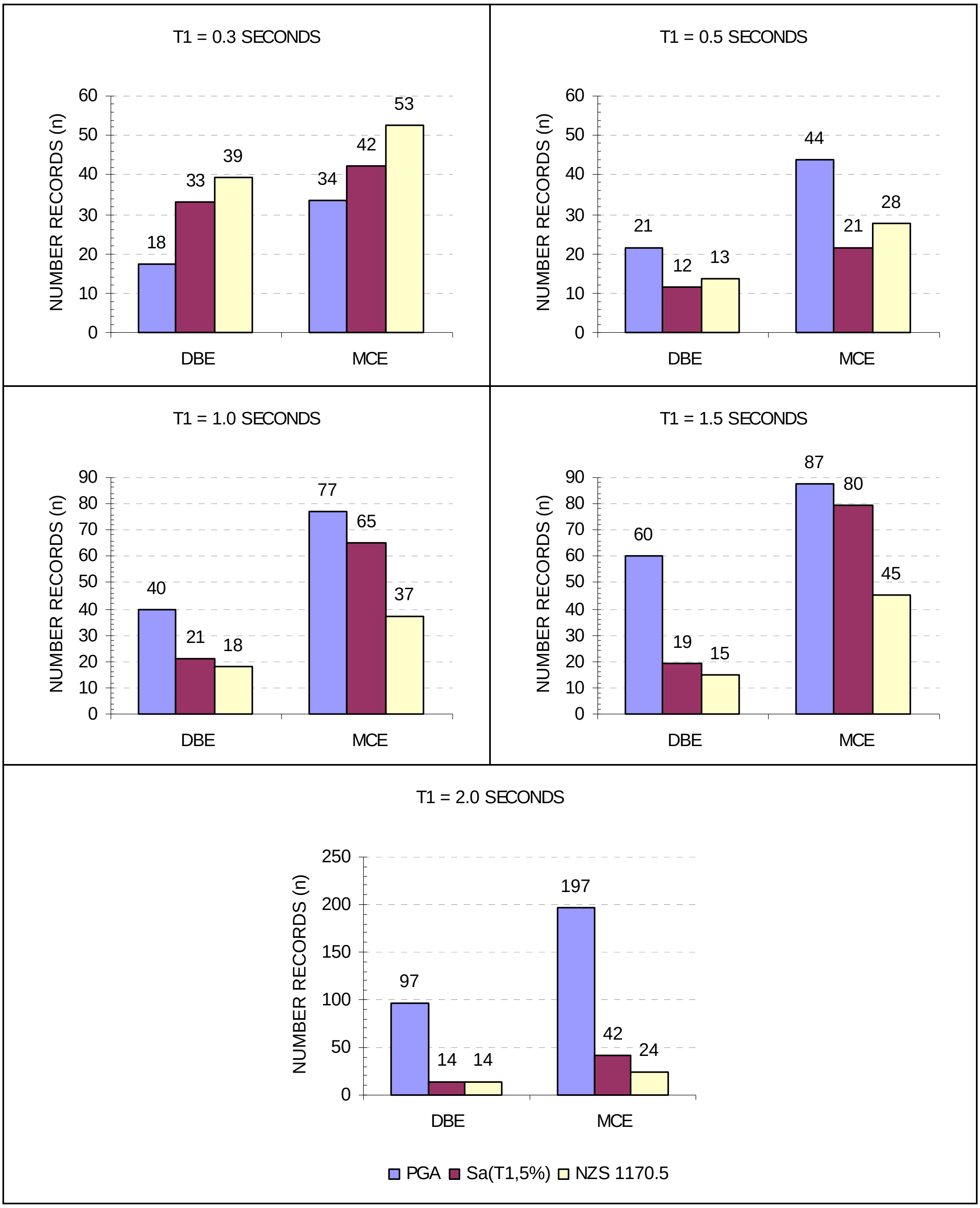

Figure 7: Number of records required to estimate the median response within a factor of ±20%

nonlinear time history analysis be adopted for design purposes. It requires scaling the records using a scaling factor k_1 which minimises in a least mean square sense the function $\log(k_1 S_a^{record}/S_a^{target})$ over the period range of interest between $0.4T_1$ and $1.3T_1$ where T_1 is the natural first-mode period of the structure being designed. The three records could be randomly chosen; the only guideline specified in the standard for selection of ground motion records is that the scaling factor must lie between 0.33 and 3. Hence, from the 20 ground motion records used in this study, any three among the 15 records that satisfy this criterion could be chosen. To explore the likely variation in the seismic demand due to random selection of three records, three different scenarios are investigated herein. These scenarios include using:

(i) all 20 records;
(ii) all 15 records that satisfy the acceptance criteria $0.33<k_1<3$; and
(iii) only three eligible records selected randomly from the suite.

The variations of dispersion of the *edp*s at different values of NZS1170.5 *im* for these three different scenarios are compared in Figure 8. In this figure, 'NZS' denotes using all eligible records for which $0.33<k_1<3$; 'NZS all records' denotes using all 20 records; and '3 records NZS' denotes using 3 eligible records randomly selected from the suite. The comparisons are shown for three structures with natural periods of 0.5, 1.0, and 1.5 sec. It can be seen in the plots that for smaller period structures the dispersion is low (because

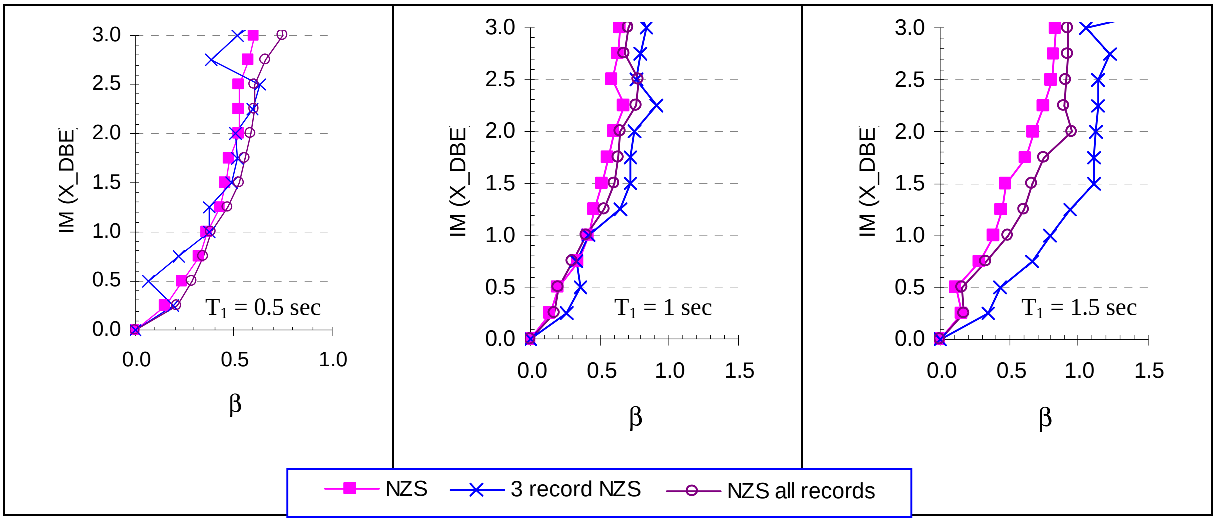

Figure 8: Comparisons of dispersions for T_1 = 0.5 sec, 1 sec and 1.5 sec

the response of these relatively stiff structures is closer to the applied ground motion) and there is little difference between the three scenarios. When only 3 records are used, at some intensities the structural responses are clustered and indicate less variability in response resulting in artificially low dispersions. Hence, the only apparent observation from the 0.5 and 1.0 sec structures, perhaps, is the zigzag nature of the dispersion profile of the "3 records" scenario, thereby hinting at its lack of dependability.

The dispersions for the 1.5 sec structure are consistently higher than those for the two stiffer structures. The plot also shows that using only 3 records gives dispersions which are roughly twice the dispersions obtained by using all 15 eligible records from the suite. As can be seen from Figures 5 and 6, using only 3 records for NZS1170.5 IM gives consistently higher dispersions than using $S_a(T_1,5\%)$ and in many cases also higher than using PGA. It follows that the whole idea of using the complex IM suggested in NZS1170.5 [1] cannot be defended if only 3 records are to be used.

On the other hand, using all 20 records (including those for which the scaling factor is outside the range 0.33-3) slightly increases the dispersion compared to using only the eligible records. In fact, regardless of the structural period, using only the eligible records from the suite is always found to yield the least dispersion. Hence, the approach of screening the ground motion records as stipulated in NZS1170.5 [1] is undoubtedly beneficial. This also indicates that the efficiency of NZS1170.5 IM could be further increased (albeit slightly as indicated by the difference between using all 20 records and only the eligible records in Figure 8) by narrowing the allowable range of the scaling factor k_1, say to 0.5-2.

In addition to the component scaling factor k_1, NZS1170.5 [1] also prescribes using a family factor k_2 for each record (the value of which is not allowed to be less than 1) in the family of three earthquakes so that for every period in the range of interest, the response of at least one record is larger than the design/target spectrum. While doing so, all three records can be assigned k_2 factors, but for simplicity only one record is rescaled here using the k_2 factor to ensure that its S_a ordinates are at least equal to the design spectra ordinates within the range of $0.4T_1$-$1.3T_1$. This is equivalent to $k_2 = 1.0$ (less than actual) for the other two records and more than needed for the rescaled record, thereby artificially inflating the dispersion compared to the actual NZS1170.5 approach. Nevertheless, as dispersion is not the topic of discussion here, this simple approach is adopted.

Figure 9 shows the IDA curves (using NZS1170.5 IM) of three randomly selected ground motion records that satisfy

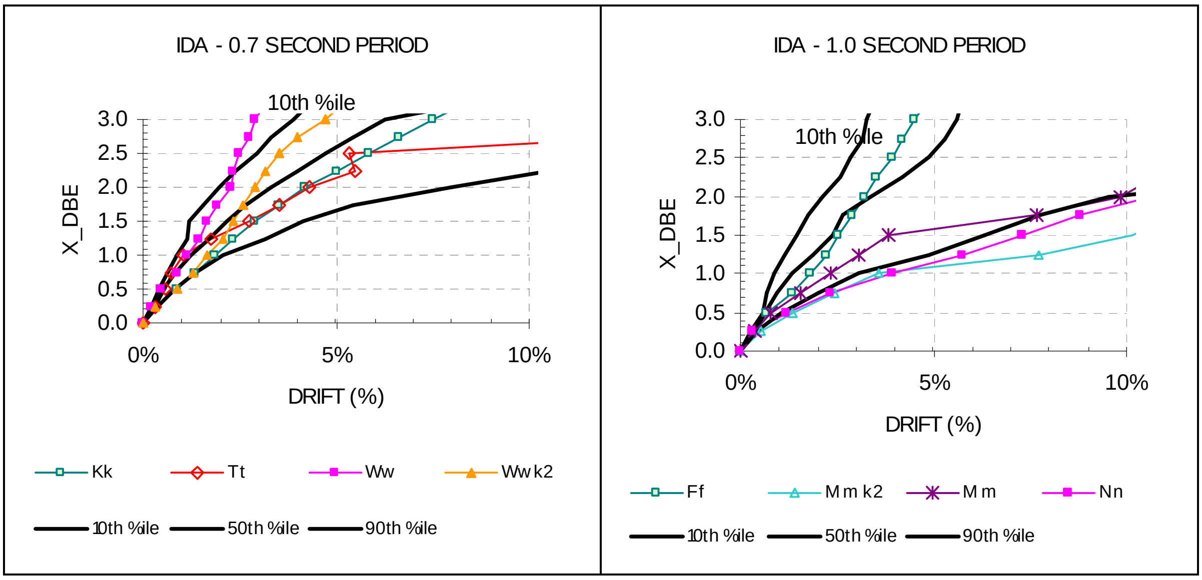

Figure 9: Effect of the family scale factor k_2 on the confidence of the predicted seismic demand

the $0.33<k_1<3$ criteria for structures with 0.7 sec and 1.0 sec natural periods, respectively. The 10^{th}, 50^{th} and 90^{th} percentile IDA curves generated from the IDA data of all acceptable records from the suite are also superimposed on the figure to facilitate the discussion. In the $T_1 = 0.7$ sec case, the three ground motion records are Kk, Tt and Ww. As shown in the figure, record Ww produces the smallest response, even less than the 10^{th} percentile values at higher IMs. The other two ground motion records produce almost similar responses, consistently between the 50^{th} and 90^{th} percentile IDA curves. Applying the k_2 factor to record Ww produces a response closer to the 50^{th} percentile response, still noticeably less than the response of the other two records without being inflated by the k_2 factor, outlining that a single record modified by k_2 factor to exceed the design spectra throughout the $0.4T_1$-$1.3T_1$ range does not necessarily result in it producing the largest response of the 3 records.

Among the three records (Ff, Mm, and Nn) randomly selected for the $T_1 = 1$ sec structure, record Mm falls between the other two and also between the 50^{th} and 90^{th} percentile responses. Applying the k_2 factor to Mm results in the response easily exceeding that of the 90^{th} percentile IDA curve. If the k_2 factors were applied to all three records as suggested in NZS1170.5 [1], record Nn which is already (without k_2 factor) beyond the 90^{th} percentile line, would definitely give a very high response. As the recommendation is to adopt the maximum among the three predictions, the design demand in this case would have been extremely conservative. These two cases clearly indicate that applying factor k_2 to all three records will produce a maximum response which is highly likely to exceed the 90^{th} percentile response, and that too by a big margin in many cases. Note that in a typical probability distribution function such as Gaussian or lognormal which have been used commonly to represent the variation in *edp*s [5], a small increase in the cumulative probability beyond 90% corresponds to a big increase in the value of the *edp*. As this level of confidence can hardly be justified for any performance requirement (perhaps, except for the life safety criteria), the use of k_2 factor to scale all three records as currently specified in NZS1170.5 [1] significantly overestimates the seismic demand thereby leading to an overly conservative design if the nonlinear time history analysis method is used in seismic design.

7. CONCLUSIONS

Based on the computational investigation described in this paper, the following conclusions can be drawn:

1. Significant record-to-record variation in structural response will invariably occur even though the ground motion records have been scaled to the same level of intensity. Incremental dynamic analysis (IDA) can be conducted to assess the degree of this record-to-record variation that can be quantified using dispersion factors (i.e. standard deviation calculated assuming lognormal distribution).
2. By using more 'efficient' intensity measures (IM), the dispersions can be significantly reduced. It is found that the NZS1170.5 IM (based on least squares fit of logarithms of spectral accelerations at periods within a range of 0.4-1.3 times the natural period of the structure) is found to be slightly more efficient than the 5% damped spectral acceleration at the natural period; i.e. $S_a(T_1,5\%)$, and somewhat more efficient than the peak ground acceleration (PGA). The reason for the NZS1170.5 based IM being the most efficient is its ability to incorporate the effects of softening ($T>T_1$) due to inelastic response and the contribution of higher order modes ($T<T_1$) in the scaling process.
3. Although $S_a(T_1,5\%)$ is found to provide a similar level of dispersion as NZS1170.5 IM up to 0.5DBE and at all intensity levels for lower period structures. Given the small difference between the dispersions using $S_a(T_1,5\%)$ and NZS1170.5 IM in other cases, too, and given that processing the records using the NZS1170.5 IM is significantly more cumbersome, it is a close call to select the IM for SDOF systems. Nevertheless, the SDOF system analytical results were scrutinised to infer that the difference in efficiency is likely to be more in larger period MDOF systems where higher order modes are likely to play a significant role in the overall response.
4. The aforementioned gain in efficiency is true only if a significant number of records (20 in this study) are used. If only three records (allowed in NZS1170.5) are used, the efficiency of NZS1170.5 IM is consistently less than $S_a(T_1,5\%)$, thereby making the use of the complex NZS1170.5 IM difficult to defend. On the other hand, the guideline to select records based on the $0.33<k_1<3$ criteria has been found helpful in further enhancing the efficiency of the NZS1170.5 IM, and further narrowing the allowable range of k_1 will be beneficial.
5. The use of the family scale factor k_2 to scale all three records is found to result in significant overestimation of the demand leading to an unintended overly conservative design if time history analysis is used for seismic design purpose.

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