EARTHQUAKE RECORDS FOR MULTI-LEVEL SEISMIC PERFORMANCE ASSESSMENT OF STRUCTURES

Rajesh P Dhakal[1], John B Mander[2] and Naoto Mashiko[3]

ABSTRACT: A method is established to identify critical ground motions that are to be used in physical testing to enable different levels of seismic performances to be assessed. The earthquake identification procedure consists of: choosing a suitable suite of ground motions and an appropriate intensity measure; selecting a computational tool and modelling the structure accordingly; performing *Incremental Dynamic Analysis* on a nonlinear model of the structure; interpreting these results into 50th (median) and 90th percentile performance bounds; and identifying the critical earthquakes that are close to these probabilistic curves at intensities corresponding to the design basis earthquake and the maximum considered earthquake. An illustrative example of the procedure is given for a reinforced concrete highway bridge pier designed to New Zealand standards. Nonlinear time-history dynamic analyses and pseudodynamic tests are performed on the pier using the three ground motions identified as: (i) a *Design Basis Earthquake* (10% probability in 50 years) with 90 percent confidence of non-exceedance; (ii) a *Maximum Credible Earthquake* (2% probability in 50 years) representing a median response; and (iii) a *Maximum Considered Event* representing 90 percent confidence of non-exceedance.

KEYWORDS: Design basis earthquake (DBE), Maximum credible earthquake (MCE), Incremental Dynamic Analysis (IDA), Pseudodynamic test, Critical ground motions.

1. INTRODUCTION

In order to experimentally assess seismic performance of structures, the loading/action to which the physical structural model is to be subjected to needs to be decided in advance. Historically, in customary experimental practice, gradually increasing reversed cyclic displacements are applied to the specimen (i.e. scaled physical model) until adverse performance or collapse is observed. The results are then used to infer the likely performance of the prototype structure when exposed to design basis and/or extreme earthquakes. Nevertheless, such controlled and patterned displacement cycles are markedly different from the actual response of structures to earthquakes which consist of irregular and random displacement reversals. To assess the seismic performance of structures, the ground motion needs to be applied directly to the physical model of the structure. This is commonly accomplished either through a pseudodynamic test or a shaking table test.

For seismic design, most codes and standards specify the *design basis earthquake* (DBE) having 10% probability in 50 years (i.e. 475 years return period) and the *maximum credible earthquake* (MCE) having 2% probability in 50 years (i.e. 2450 years return period) in terms of a single intensity measure such as the *Peak Ground Acceleration* (PGA) or the spectral acceleration ordinate at a given period.

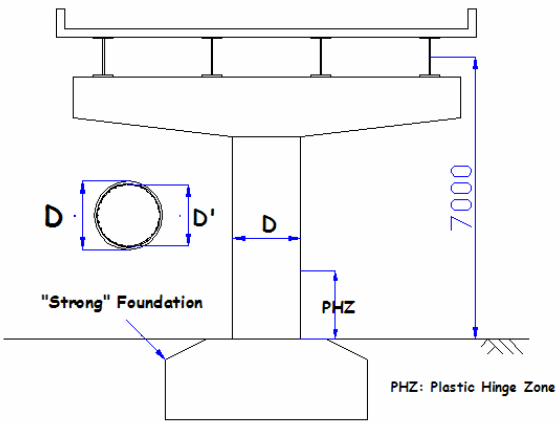
[1] *Senior Lecturer, Dept. of Civil Engineering, University of Canterbury, Christchurch, New Zealand*
[2] *Professor, Dept. of Civil Engineering, University of Canterbury, Christchurch, New Zealand*
[3] *Metropolitan Expressway Co Ltd, Tokyo Construction Department, Japan*

Codes and standards generally remain silent on other important aspects of earthquake records, such as the frequency content, duration and effective number of loading cycles. Different ground motion records with the same PGA do not induce the same level of response and also do not cause the same extent of damage on a structure [1]. Hence, the test results obtained by using one ground motion record may not provide sufficient confidence that the structure, if subjected to another ground motion record with the same PGA, will yield similar response.

Hence, a methodology that has a degree of formalism in ground motion selection for experimental use is required. The principal purpose is to select records that give the user a measurable degree of confidence in their use. That is, there is a prescribed probability of not exceeding a certain level of adverse performance when any earthquake of similar intensity strikes. This paper establishes a methodology to identify these critical ground motion records based on a thorough analytical investigation. To illustrate the proposed procedure, ground motion records that induce 90th percentile response as a DBE, and 50th and 90th percentile responses as an MCE, respectively for a typical bridge pier are identified in this paper. In order to highlight the application of the identified records in multilevel performance assessment of structures through experiments on one-off specimen, pseudodynamic tests are conducted on a scaled model of the pier with these three records arranged sequentially.

2. GENERAL METHODOLOGY AND TARGET STRUCTURE

The major steps in identifying the critical ground motions representing different levels of seismic hazard with a required level of confidence are: (i) collection of ground motion records to be investigated (the critical ground motions will be chosen from the collected suite of records); (ii) selection of a computational tool and modelling the structure (the tool must be capable of conducting nonlinear time-history analysis); (iii) performing *incremental dynamic analysis* (i.e. IDA, which involves performing nonlinear dynamic time-history analyses of the structural model under the collected suite of ground motion records, each scaled to several intensity levels designed to force the structure all the way from elastic response to final global dynamic instability); (iv) ranking of the records based on the responses at the intensity levels of DBE and MCE (a percentile confidence value can be attached to different records depending on its rank); and (v) identifying the records that are close to the desired confidence levels (usually 50% and 90%) at the DBE and MCE intensity levels.



D	1700
D'	1540
PHZ	1700
$P/A_g f'_c$	0.15
Bar	28-D32
ρ_τ	0.99%
Spiral	R20@170
ρ_σ	0.49%
T_n	0.85 sec

ρ_τ: the ratio of the longitudinal bars area to the pier's cross sectional area, ρ_σ: the ratio of the volume of the spiral to the volume of the confined concrete, T_n: natural period of the pier

Figure 1. Prototype bridge pier and its design details

To make these steps clearer, the generic methodology described above is applied to identify the ground motion records to be used in multilevel seismic performance assessment of a reinforced concrete bridge pier. The bridge pier selected for this example is designed using the seismic design standard of New Zealand [2]. The pier is 7 m high and is taken from a "long" multi-span highway

bridge on firm soil with a 40 m longitudinal span and a 10 m transverse width. The weight of the super-structure to be supported by the pier is calculated to be 7,000 kN. The bridge is considered to be located in a high seismic zone in New Zealand with the PGA of the DBE being 0.4 g. The elevation view of the pier along with its design parameters of the pier are illustrated in Figure 1.

3. CASE STUDY: REINFORCED CONCRETE BRIDGE PIER

3.1 COLLECTION OF SEISMIC GROUND MOTION RECORDS

Although New Zealand is a seismically active country, fortunately not many big earthquakes have occurred in New Zealand in the recent past, thereby creating a scarcity of regional seismic ground motion records. New Zealand has the Alpine fault passing longitudinally almost through the middle of the country and many cities in New Zealand fall as close as a few kilometres from this fault. To represent a location in New Zealand, earthquakes recorded at moderate distances from the source are needed. For this purpose, the suite of twenty ground motion records used by Vamvatsikos and Cornell [3] is adopted in this example. The details of all ground motion records in the suite are listed in Table 1. Note that the earthquakes shown in the list are all recorded in the United States on firm soil, and earthquakes recorded elsewhere on firm soils and at a moderate distance from the source could easily be added to the list. It will surely increase the amount of analysis to be done before coming to conclusion, but may not necessarily noticeably enhance the final outcome.

Table 1 Collected seismic ground motion records

No	Event	Year	Station	ϕ^1	M^{*2}	R^{*3} (km)	PGA (g)
1	Loma Prieta	1989	Agnews State Hospital	90	6.9	28.2	0.159
2	Imperial Valley	1979	Plaster City	135	6.5	31.7	0.057
3	Loma Prieta	1989	Hollister Diff. Array	255	6.9	25.8	0.279
4	Loma Prieta	1989	Anderson Dam	270	6.9	21.4	0.244
5	Loma Prieta	1989	Coyote Lake Dam	285	6.5	22.3	0.179
6	Imperial Valley	1979	Cucapah	85	6.9	23.6	0.309
7	Loma Prieta	1989	Sunnyvale Colton Ave	270	6.9	28.8	0.207
8	Imperial Valley	1979	El Centro Array #13	140	6.5	21.9	0.117
9	Imperial Valley	1979	Westmoreland Fire Sta.	90	6.5	15.1	0.074
10	Loma Prieta	1989	Hollister South & Pine	0	6.9	28.8	0.371
11	Loma Prieta	1989	Sunnyvale Colton Ave	360	6.9	28.8	0.209
12	Superstition Hills	1987	Wildlife Liquefaction Array	90	6.7	24.4	0.180
13	Imperial Valley	1979	Chihuahua	282	6.5	28.7	0.254
14	Imperial Valley	1979	El Centro Array #13	230	6.5	21.9	0.139
15	Imperial Valley	1979	Westmoreland Fire Sta.	180	6.5	15.1	0.110
16	Loma Prieta	1989	WAHO	0	6.9	16.9	0.370
17	Superstition Hills	1987	Wildlife Liquefaction Array	360	6.7	24.4	0.200
18	Imperial Valley	1979	Plaster City	45	6.5	31.7	0.042
19	Loma Prieta	1989	Hollister Diff. Array	165	6.9	25.8	0.269
20	Loma Prieta	1989	WAHO	90	6.9	16.9	0.638

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

3.2 COMPUTATIONAL MODELLING AND PERFORMING IDA

In order to perform IDA of the bridge pier, a nonlinear finite element analysis program RUAUMOKO [4] is used in this example. The RC pier is modelled as a single-degree-of-freedom system and is analysed using a modified Takeda hysteresis model [4]. Using RUAUMOKO, dynamic time-history analyses are carried out for the twenty ground motion records in the suite. The acceleration amplitude

of each ground motion record is multiplied by a scaling factor to vary the intensity measure (i.e. PGA in this case), but the time scale of the ground acceleration record is not altered. Starting from a small scaling factor, the scaling factor is gradually increased until the scaled ground motion causes collapse of the bridge pier. The critical response parameter (i.e. the engineering demand parameter) is chosen to be the maximum drift ratio experienced by the pier during the ground motion duration.

The maximum drift incurred by a scaled ground motion with a known PGA gives one point in the intensity measure versus engineering demand parameter (PGA vs. maximum drift in this example) plot and similar points corresponding to different values of PGA for the same earthquake record are joined together to yield the IDA curve for that earthquake. A typical IDA curve (i.e. PGA vs. maximum drift relationship) generated for one earthquake in the suite is presented in Figure 2 (a). Similar curves for the other earthquakes in the suite are also generated through series of dynamic time-history analyses. The generated IDA curves for the 20 earthquakes are plotted together in Figure 2 (b).

3.3 IDENTIFYING CRITICAL GROUND MOTIONS

Before selection of the critical ground motions, the intensity measure (PGA) corresponding to the seismic hazard levels for performance based seismic design (i.e. DBE and MCE) need to be determined. For the design location in New Zealand, the PGA of the DBE is 0.4g and that of the MCE is assumed to be 0.8g. For each PGA value, there are 20 (equal to the number of ground motion records collected) different values of the maximum drift, from which a median (50th percentile) and a 90th percentile values of the maximum drift for that PGA level are obtained. When these points corresponding to different PGA levels are connected, the 50th and 90th percentile IDA curves can be generated. Figure 2 (b) shows the 10th, 50th and 90th curves for the bridge pier generated based on the variability of the twenty IDA curves. The selection of the critical ground motions can now be performed by matching the corresponding individual IDA curves of different earthquakes against these fractile curves at the required PGA level. The critical ground motions to be chosen should pass through or very close to the point of intersection of the corresponding percentile IDA curve and the horizontal line at the PGA of the seismic hazard to be represented (i.e. DBE or MCE).

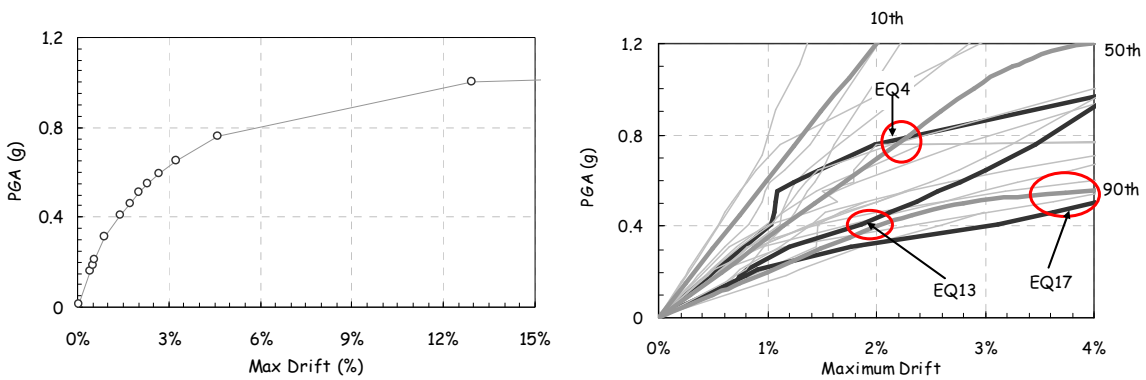


Figure 2 (a) Generation of an IDA curve; (b) Identifying the critical records

In Figure 2 (b), one such earthquake record has been identified as EQ13 which tended to be slightly weaker than 90% of all collected earthquakes records when scaled at 0.4g PGA, but it was the closest to the 90th percentile IDA curve at PGA = 0.4g. The second earthquake record (EQ4) serves as the MCE and it was chosen because it passed very close to the point of intersection of the 50th percentile IDA curve and the horizontal line at 0.8g (PGA of MCE). This earthquake record (EQ4) represents the average of the twenty earthquakes records scaled at PGA = 0.8g. The third and final earthquake record is to be chosen such that it represents 90th percentile of all records at PGA = 0.8g. Nevertheless, many records in the suite caused failure of the pier at a PGA level lower than 0.8g, and hence the 90th percentile ground motion at PGA = 0.8g could not be explicitly selected. In this example, EQ17 is

chosen to represent the destructive hazard level because it is one of the few earthquakes running very close to the 90th percentile IDA curve. Note that the ground motion records are scaled to yield PGA = 0.4g for EQ13, and PGA = 0.8g for EQ4 and EQ17, and these records are recommended to be used in seismic performance assessment of the bridge pier. What this means is: If EQ13 is scaled to PGA = 0.4g and applied to the pier, there is only a small chance (~10%) that the incurred response/damage will be exceeded in a DBE. Similarly, if EQ4 and EQ17 are scaled to PGA = 0.8g and applied to the pier, the incurred responses and damages will have respectively about 50% and 10% chance of being exceeded in a random MCE.

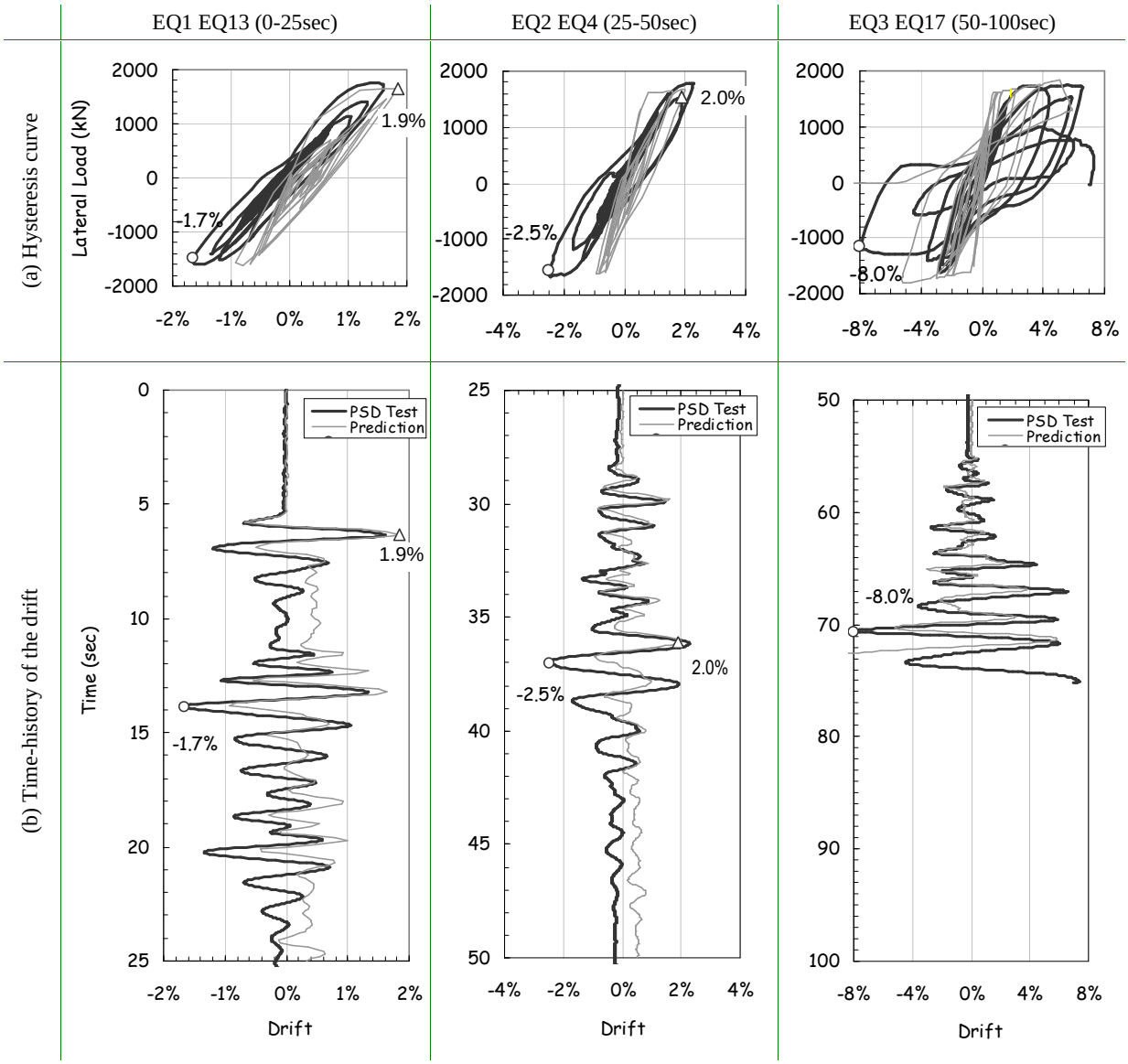


Figure 3. Analytical and experimental behaviour of the pier under the selected earthquakes

4. VERIFICATION OF THE PROPOSED PROCEDURE

Next, the validity of the procedure is verified qualitatively based on the results of time-history analysis for these three scaled records conducted earlier during the IDA and also based on the results of pseudo-dynamic tests on scaled model of the prototype pier. In order to perform multilevel performance assessment using a single specimen (i.e. to save the resources), the pseudodynamic test was carried out with the three ground motions applied sequentially. The details of the test

methodology and results can be found elsewhere [5]. The analytically predicted and experimentally measured load-displacement curves and time-history of the pier-top drift due to the three identified ground motions are presented in Figure 3.

The maximum drift is 1.9% during the DBE (EQ13) and 2.5% during the 50th percentile MCE (EQ4). This may seem a very small increase in the response given the fact that the PGA of EQ13 is 0.4g whereas that of EQ4 is 0.8g, but the confidence level these two ground motions impart are significantly different; i.e. EQ13 is one of the strongest records for PGA = 0.4g whereas EQ4 represents a median record for PGA = 0.8g. This reinforces the commonly held view that using PGA alone is an insufficient representation of the hazard level. The pier collapsed when subjected to the 90th percentile MCE (EQ17). This was expected as the DBE with PGA = 0.4g was adopted for designing the pier. As shown in Figure 3, both the test and analysis showed 8% drift, after which the analysis indicated numerical failure and the test was terminated due to imminent collapse. The lateral load vs. drift hysteresis curve also suggests that the damage sustained and the energy dissipated during EQ17 was significantly larger than during the previous two records. Hence, it is easy to conclude that EQ17 (identified as 90th percentile MCE) was much more devastating than EQ13 (identified as 90th percentile DBE), and more interestingly than EQ4 (identified as 50th percentile MCE) despite having the same PGA. This qualitatively corroborates that the three identified ground motions give conceptually logical outcomes.

5. CONCLUSIONS

This paper has proposed a systematic procedure for the identification of critical ground motions to be used in the seismic performance assessment of structures. By using Incremental Dynamic Analysis (IDA), median and upper limit response expectations can be determined and the typical ground motions that would cause such response can be identified. Confidence bounds can thus be assigned to the experimental outcomes. The efficacy of the proposed ground motion selection methodology has been demonstrated via a dual computational-experimental investigation. The proposed procedure is particularly useful in planning and conducting destructive tests that would enable multilevel performance assessment with only one specimen where the experimentalist often has only a single specimen to conduct a meaningful experiment.

6. REFERENCES

1. Iervolino, I. and Cornell, C.A., “Record selection for nonlinear analysis of structures”. *Earthquake Spectra*, Vol. 21, 2005, No. 3, pp. 685-713.
2. NZS3101-95. *Concrete Structures Standard: NZS3101*. Standards New Zealand, Wellington, New Zealand, 1995.
3. Vamvatsikos, D. and Cornell, C.A., “Applied incremental dynamic analysis”. *Earthquake Spectra*, Vol. 20, 2004, No. 2, pp. 523-553.
4. Carr, A.J. *RUAUMOKO: Inelastic Dynamic Computer Program.* Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 2004.
5. Dhakal, R.P., Mander, J.B. and Mashiko, N., “Identification of critical ground motions for seismic performance assessment of structures”. *Earthquake Engineering and Structural Dynamic*, 2006 (In Press).