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Keynote Lecture: Some Recent Developments in the Selection of Ground Motions for Design

W. D. Liam Finn
University of British Columbia, Canada

A. Wightman
BGC Engineering, Canada

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SOME RECENT DEVELOPMENTS IN THE SELECTION OF GROUND MOTIONS FOR DESIGN

W. D. Liam Finn
University of British Columbia
Vancouver, British Columbia, Canada

A. Wightman
BGC Engineering
Vancouver, Canada

ABSTRACT

This paper describes some recent developments in the selection of ground motions for design; the conditional mean spectrum approach and risk targeted ground motions. The conditional mean spectrum approach is just finding its way into practice and its application to a major dam is presented. Risk targeted ground motions are the basis for the next generation of building codes in the USA. The process of determining these motions is explained. Finally in the context of the retrofit of 800 schools in British Columbia, Canada, a performance based design procedure based on incremental dynamic analysis (IDA), with direct application to geotechnical earthquake engineering is presented. An interesting feature of this method is the segregation of hazard into subduction, sub-crustal and crustal earthquakes and the calculation of risk for each type independently and combining these risk components to obtain the total risk of violating the performance criterion.

INTRODUCTION

The selection of appropriate ground motions for design is still a controversial issue despite over forty years of experience with dynamic response analyses of buildings and earth structures. Over the years there has been a steady accumulation of recorded strong motions and the creation of large data bases accessible to designers. One of the best of these is the PEER data base with over 2700 uniformly processed strong motion records. The availability of suitable candidate records for use as input motions for analysis is no longer an issue but there are no generally accepted criteria for selecting suitable records and for deciding how many records are necessary for an analysis to yield dependable results. Typical of good current practice are the views of Shome et al. (1998), the US Army Corps of Engineers' manual, EM-1110-2-6051 (2003) and Bommer and Avecedo (2004).

Shome et al. (1998) discussed the selection of earthquake records for use in non-linear response analysis of a multi degree of freedom structure comprising a five-storey moment-resisting frame. Using three suites of 20 records each in three "bins", representing three different M and R events, they concluded that by first scaling each suite to the bin-median spectral acceleration at the fundamental period of the structure they reduced the dispersion (scatter) of results, but obtained the same median result as when the same records were used un-scaled. Furthermore, by scaling records from one bin to the median intensity level of other bins, the median results were very close to the unscaled case, leading to the conclusion that the non-linear response of a structure was not significantly dependent on magnitude, distance, or duration, but only on spectral acceleration at the fundamental frequency of the structure. In general the use of normalization to the fundamental period of the structure $S_a f_o$ reduced the number of records required to achieve a stable median response by a factor of about 4.

The US Army Corps of Engineers' engineering manual EM-1110-2-6051(2003) provides detailed guidance for the selection and scaling of earthquake records for design of hydraulic structures. It recommends selection of records on the basis of the seismological characteristics of the design event; tectonic environment, magnitude and type of faulting, distance, site conditions, response spectrum, duration of strong shaking, and pulse characteristics (directivity). Guidance is provided for the number of time histories needed; at least three for linear dynamic analyses, and at least five for non-linear analyses. This latter recommendation applies for both linearly scaled records and spectrum-matched records. Criteria are provided for spectrum fit in the period range of significance for individual time histories, and also for the aggregate spectrum fit for time-history sets. When selecting records for multi-component analyses, the preferred option is to preserve the relative amplitudes of the individual components present in the original records, but the alternative of using different scaling factors for each component is also considered acceptable.

Bommer and Acevedo (2004) discussed record selection in terms of seismological and geophysical parameters; magnitude, distance, and site classification, and compared this with matching spectral amplitude and spectral shape. They argued that strong motion records can be selected from around the world provided they come from a tectonic environment appropriate to the site under consideration, either a subduction zone, active crustal region, or stable continental region. This procedure was used in developing ground motions for the retrofit of BC schools and is described in a later section.

Bommer and Acevedo (2004) disagree with Shome et al., (1998) and insist that records should be selected from within 0.2 magnitude units of the target scenario earthquake. On the other hand they agree that distance is not such an important selection criterion. Site classification should be matched if possible, but selection from recordings made within one site class on either side of the target site class is considered acceptable.

Duration of strong shaking is hugely important for assessing the potential for liquefaction of loose saturated cohesionless soils and for determining the extent of remediation to mitigate the risk of liquefaction. Therefore for liquefaction problems, magnitude, as a surrogate for duration, becomes an important variable in the selection of appropriate ground motions for design.

Two new approaches for establishing ground motions for design which will have significant impact on practice either directly or by opening new perspectives on the issue of what are the relevant characteristics of suitable design motions will be described. These approaches are; (a) using the site-specific conditional mean spectrum-ε to define the appropriate motions and (b) selecting motions compatible with a specified risk of violating an acceptable performance criterion. A case history of performance based earthquake engineering involving the seismic retrofit of 800 schools in British Columbia over a 15-year period is presented that illustrates the use of displacement based performance criteria which has applications in geotechnical engineering. The BC project also has some other features of interest to geotechnical engineers; deaggregation of hazard by earthquake type (tectonic environment), selection and scaling of design input motion for each earthquake type, use of incremental dynamic analysis to determine risk and how to structure and access a data base of hundreds of thousands of results from nonlinear analyses to avoid having to do any further site or structure specific analysis.

CONDITIONAL MEAN SPECTRUM-E

A recent development in the selection of time-histories that are consistent with the results of a PSHA has been presented by Baker et al., (2006a). Referred to as the "conditional mean spectrum considering epsilon", CMS-ε, the procedure results in the specification of a complete scenario spectrum which is the expected (mean) spectrum, given the target spectral acceleration at the period of interest. Epsilon is the number of standard deviations by which the target spectral acceleration at the period of design interest is above (or below) the logarithmic mean provided by the ground motion prediction equation used in the hazard analysis. The target spectral acceleration is usually derived probabilistically, and for most building code and critical structural design applications such as large dams, will be significantly higher than the mean value. The period of interest might be the predominant period of the structure, or perhaps the spectral periods associated with different modes evident in the hazard de-aggregation; a short period for nearby earthquakes, and a long period for distant events. The conditional mean spectral shape is calculated using a set of epsilon correlation factors derived from analysis of 267 recordings in the PEER database, each with three components. Fig. 1, taken from Baker and Cornell (2006b) shows correlation factors for the epsilon values of spectral ordinates in the same horizontal component.

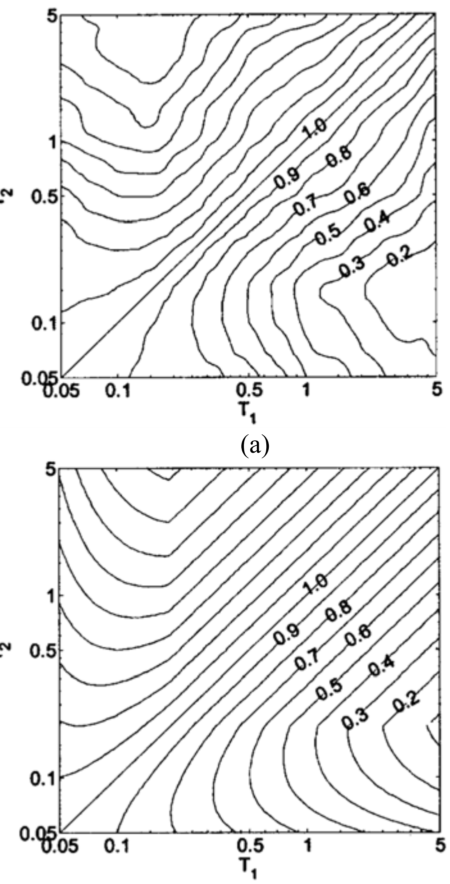

Fig. 1. Epsilon correlation factors (after Baker and Cornell, 2006b).

The diagonal in Fig. 1a shows the perfect correlation when T1 = T2, and the contours depict the decay in this correlation as T2 moves away from T1. The contours in Fig. 1a are derived from the actual data points from the analysis of the 267 recordings. These are smoothed to fit a simple mathematical expression in Fig. 1b. Another way to view this, shown by Abrahamson (2009), is reproduced in Fig. 2. Here, two slices through Fig. 1 at T1 = 0.2 and 2.0 show how the epsilons are perfectly correlated with themselves but the correlation decreases with increasing separation of spectral periods.

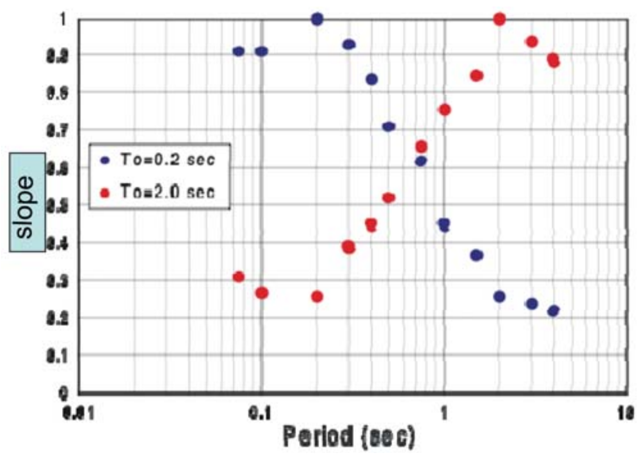

Fig. 2. Epsilon Correlations at T=0.2s and T=2.0s (from Abrahamson (2009))

Using the information for mean epsilon correlation, an example on Fig. 3 illustrates the essential features of the CMS-ε target spectrum development. As part of a dam safety assessment for a very high consequence dam on Vancouver Island, British Columbia, a probabilistic seismic hazard analysis defined the uniform hazard response spectrum (UHRS) at an annual frequency of exceedance of 1x10-4. This is the heavy black line in Fig.3. Hazard deaggregation gave mean magnitude and mean displacement, Mbar = 7.3, and Dbar = 8.3 km.

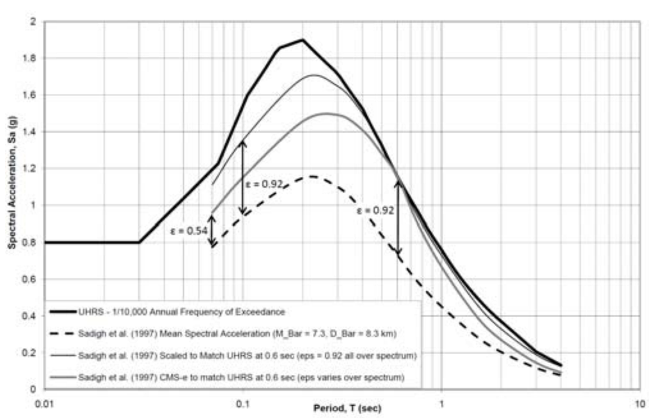

Fig. 3. Example of CMS-ε Target Spectrum Development for Dam Safety Assessment

The median spectrum returned by the ground motion prediction equation (GMPE) is shown as the dashed line in Fig. 3, and is substantially lower than the UHRS because of the low annual frequency of exceedance. For a structure, or structure component, with a natural period of significance of 0.6s, the UHRS target spectral value is 0.92 standard deviations above the median, as illustrated by the thin solid line in Fig. 3. To use this spectrum as a target for ground motion selection, as might be done in the nuclear industry, ignores the fact that on average the epsilon value at periods other than 0.6s will not be 0.92, but will be less, as illustrated by Figs. 1 and 2. The mean spectrum, conditional on the target value at 0.6s from the UHRS is based on the epsilon correlations in Fig. 1, which gives rise to the term CMS-ε spectrum. This spectrum is shown as the solid grey line on Fig. 3, where ε is seen to vary from 0.92 to 0.54, and can be used as a target for selection of acceleration records for analysis in cases where a conditional mean response is appropriate.

Baker and Cornell (2006a) argue that since seismological parameters such as magnitude, distance, epsilon etc. are already included in the development of the target spectrum, the only criterion needed for selecting records to be consistent with the CMS-ε target spectrum is for the spectral shape of the record to match the shape of this target spectrum. A database that contains response spectral information, such as the PEER NGA flat file, can be readily searched for compatible records once the CMS-ε target spectrum is defined, greatly reducing the labor involved in selecting appropriate ground motions. These would be used for analyses intended to provide a measure of the "average" non-linear response of a structure given the specified spectral ordinate. An additional advantage of the CMS-ε method is that epsilon correlation factors have been derived not only for spectral ordinates within a single horizontal component (Fig 1), but also between orthogonal horizontal components and between the horizontal and vertical components (Baker and Cornell, 2006b). The method can therefore be used to provide target spectra for the two horizontal and one vertical components of motion. These are all conditional on the horizontal spectral acceleration at the horizontal period of significance. The procedure also allows the development of a target spectrum consistent with a range of periods of significance; for instance for a non-linear analysis of a soil structure the range of significance might be from To to 1.5*To.

An example of record selection and scaling for an earthfill dam with period of significance ranging from 0.4s to 0.8s is shown in Fig. 4. This record was selected on the basis of spectral shape, and then scaled linearly to fit the CMS-ε target to satisfy the USACE criteria for fit in the target period range.

The CMS-ε procedure addresses the concern that since a probabilistically derived UHRS does not represent the spectrum of any one earthquake, selecting and scaling earthquake records to fit the entire UHRS is conservative. If there are several periods of interest for a particular structure or a combination of interacting structures, target spectra can be developed for each scenario, and the critical case(s) determined from the results of multiple analyses. With this method, spectral shape is considered the principal if not the only record selection criterion needed for structural analysis, and the use of large scaling factors is not regarded as an impediment to record selection. The geotechnical user, however, who is interested in non-linear deformation behavior, should also pay attention to the record duration and number of cycles of strong motion, in order to recover an appropriate displacement result. For example, the concept of equivalent number of uniform stress cycles at 2/3 of the peak value can be used as a second level screen for selecting suitable records for use in analyses of soil models with liquefaction potential.

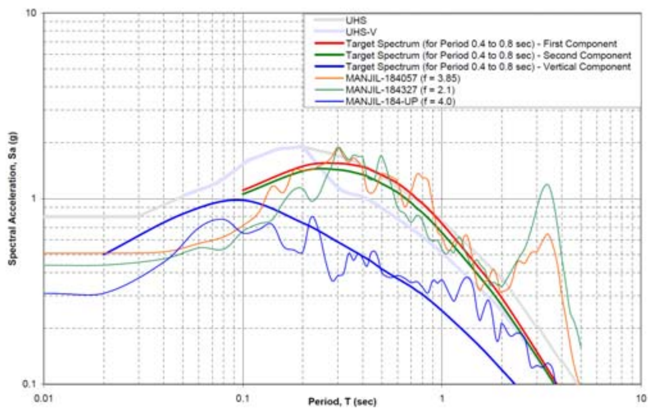

Fig. 4. Example of Record Scaling to a CMS-ε Target for Dam Safety Assessment

RISK TARGETED GROUND MOTIONS

Probabilistic ground motions for seismic design of structures in Canada and the USA have a 2% probability of being exceeded in 50 years. The seismic demand is expressed in terms of a uniform hazard spectrum and candidate motions for dynamic analysis are scaled to be compatible with the spectrum or a period range of the spectrum considered relevant for the design. This specification of motions ensures a uniform hazard in all seismic regions but not a uniform risk of collapse. Uniform hazard is not synonymous with uniform risk. There are two main reasons for this; uncertainty in the collapse capacity of buildings and differences by location in the shapes of the hazard curves from which uniform hazard estimates are derived. The next generation of US codes is proposing a shift from uniform hazard motions to motions corresponding to a uniform risk of collapse in all seismic regions of 1% in 50 years. Luco (2009) presented a very lucid and compact description of the transformation of uniform hazard motion parameters such as spectral acceleration to risk based values and his approach will be followed here. For a full formulation of the risk based approach to design, the reader is referred to ASCE7-10 and FEMA 2009.

A new generation of ground motion attenuation (NGA) models has been developed by the PEER Center at the University of California at Berkeley in cooperation with the U.S. Geological Survey and the Southern California Earthquake Center (Stewart et al. 2008). NGA formed the basis of new USA hazard maps prepared by USGS. The geometric mean of the horizontal records defines the attenuated motion. The first step in developing the risk based motions was to determine the motion in the direction of maximum response. These new maximum motions will then be modified to produce a uniform risk of collapse of 1% in 50 years.

The development of risk based motions requires specification of the risk by a fragility curve which gives the probability of the structure reaching a failure state for a given seismic demand, usually a level of spectral acceleration. The fragility curve reflects the uncertainty in predicted structural behavior. There is a great variety of fragility curves. ASCE7-10 uses a generic fragility curve that has the following properties: logarithm of the standard deviation of collapse capacity β = 0.8, and a 10% conditional probability of failure at the peak spectral acceleration corresponding to the fundamental period of the structure. The latter criterion is widely accepted in practice and is used as one of the performance criteria for retrofitting BC schools as described later.

Risk targeted ground motions with a 1% chance of exceedance that are consistent with the generic fragility curves are shown in Fig. 5 for sites in San Francisco and Memphis. The fragility curves are identical in shape but are displaced along the spectral acceleration demand axis by slightly different amounts , so that in each case the fragility curves satisfies the two controlling performance conditions; a 1% risk of collapse in 50 years and a 10% probability of collapse should the RTGM Sa value actually occur.

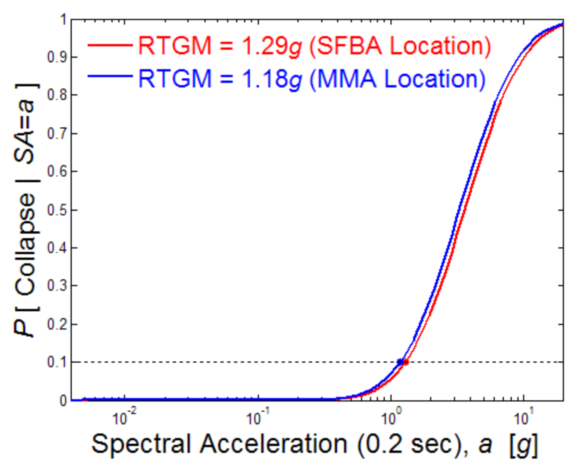

Fig. 5. Risk targeted ground motion parameters after Luco,2009.

The risk targeted ground motions (RTGM) to achieve a 1% risk of collapse in 50 years is obtained by convoluting the PDF of the fragility curve with the hazard curve for the site. The risk of collapse for a given assumed value of the RTGM is calculated using equation 1.

$$P[\text{collapse}] = \int_0^{\alpha} \frac{dP[\text{collapse} \mid SA = a]}{da} P[SA > a]\, da \qquad (1)$$

An iterative process is followed, using successive estimates of the spectral acceleration SA as the RTGM value, starting with the mapped value of SA adjusted to direction of maximum response. The fragility curve is scaled before the integration is carried out by sliding the fragility curve along the Sa axis until the conditional probability of failure is 10% for the estimated value of SA. The risk is computed for each estimate, until the 1% risk level is achieved. The process is shown by the flow chart in Fig. 6. A web based calculator will be available to evaluate the probability of collapse.

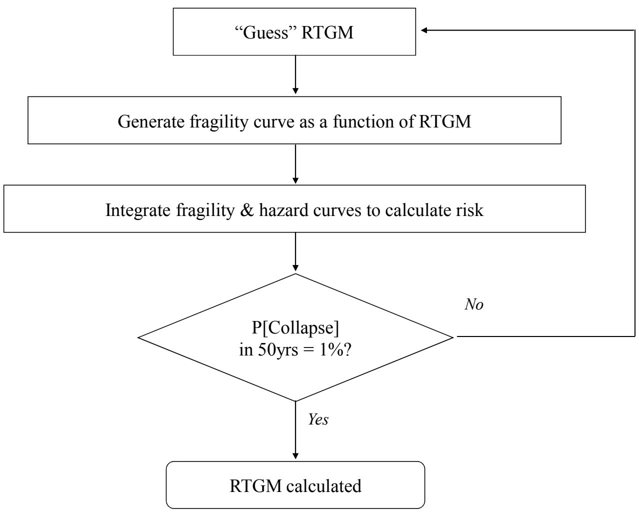

Fig. 6. Flow chart for calculating RTGM demand parameter.

GROUND MOTIONS FOR MAJOR RETROFIT PROJECT

In 2004, the British Columbia Ministry of Education initiated a $1.5 billion seismic mitigation program to make all public elementary and secondary school buildings safe. This seismic safety program is being implemented by the BC Ministry of Education (MOE) in collaboration with the Association of Professional Engineers and Geoscientists of British Columbia (APEGBC). APEGBC has been contracted by MOE to develop a set of state-of-the-art performance-based technical guidelines for structural engineers to use in the seismic risk assessment and retrofit design of low-rise school buildings. In undertaking this technical development program, APEGBC contracted the University of British Columbia (UBC) to draft the performance-based technical guidelines based on an extensive applied research program (APEGBC, 2006). Each draft of these technical guidelines has been peer-reviewed by a BC peer review committee of local consulting engineers experienced in seismic design and by an external peer review committee comprised of prominent California consulting engineers and researchers. Research on innovative retrofit methods is still being conducted and technical guidelines are issued to keep current with research developments.

The three overall objectives of the guidelines are enhanced life safety, cost effective retrofits and user-friendly technical guidance for designers.. The life safety philosophy of these guidelines is enhanced life safety through minimizing the probability of structural collapse by the use of rational performance-based engineering (PBEE) methods of earthquake damage estimation.

The performance criterion of life safety is defined by acceptable drift ratios specified for each generic school building type. The process for evaluating critical drift ratios and establishing the probability of collapse is described below.

Design Ground Motions

The seismic hazard data to schools is deaggregated by considering by considering separately the three types of seismic hazard sources that impact British Columbia; subduction, sub-crustal and crustal sources as shown if Fig. 7. Seismic hazard data for a 2% exceedance rate in 50 years for each type of earthquake was generated using the commercially available computer program EZ-RISK (Risk Engineering, 2008).

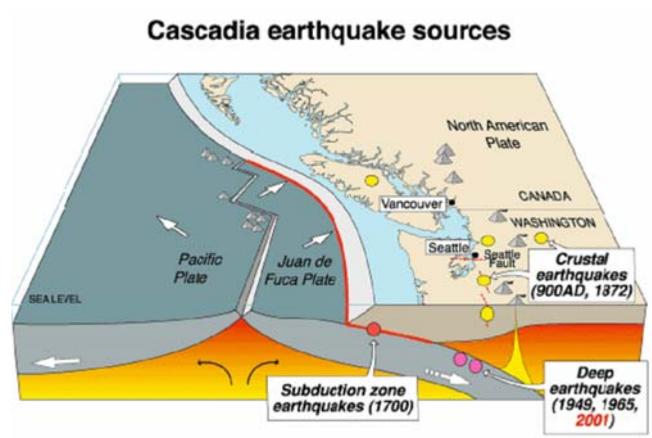

Fig. 7. Subduction, sub-crustal and crustal earthquake sources affecting British Columbia.

Ground motion at any geographic location is modeled by three ground motion suites of 10 ground motions per suite; one suite for each of crustal, sub-crustal and subduction earthquakes (Pina et al. 2010c). These records were selected on the basis of tectonic setting and appropriate magnitude, distance and site conditions. The crustal and sub-crustal suites of ground motion spectra have been scaled for Vancouver's benchmark 100% level of shaking of 2% exceedance rate in 50 years. The subduction suite of ground motion spectra have been scaled to Victoria's 100% level of shaking because of the larger cities, it is the only one affected significantly by the subduction earthquake.

Peak velocity is considered a better indicator of severe structural demand resulting in damage to a structure, than peak acceleration or displacement. Therefore ground shaking intensity is characterized by the 2% in 50 years spectral pseudo-velocity (PSV) spectrum for each geographic location and each type of earthquake. The selected ground motions are scaled linearly so that on average they match the design spectra in the period range of interest. The design ground motions for crustal earthquakes for a wood frame building are shown in Fig. 8. The motions on the average match the target spectrum in the relevant period range of interest after yield, 1s-2s. Because of the very different characteristics of each set of motions, the combined hazard derived from the individual hazard contributions of each earthquake type was less than the hazard resulting from a global hazard analysis based on considering all three types together.

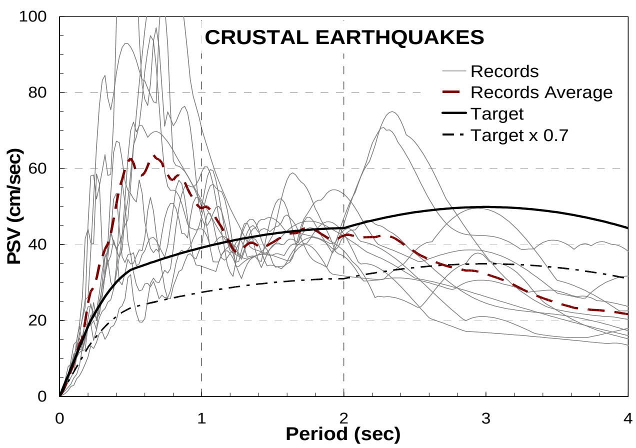

Fig. 8. Design ground motions for crustal earthquakes scaled on average to the period range of interest of 1s-2s (Pina et al. 2010c).

The full range of possible ground shaking by each scaled ground motion record is divided into a series of ground shaking increments. All levels of shaking are expressed as a percentage of a benchmark level of shaking. The "100%" intensity level is taken as the benchmark level, and it corresponds to a level of shaking with a 2% probability of exceedance in 50 years. Each ground shaking increment varies by 10% from the motions just higher and just lower.. For any geographic location, the full range of ground shaking varies from the 30% to 250% level of benchmark shaking for each selected input motion record.

Incremental Dynamic Analysis

There are 31 generic types of school buildings in BC and each is modeled for dynamic analysis by its own backbone and hysteretic curves. A typical model is shown in Fig. 9.

The performance of each generic school type is explored by nonlinear dynamic incremental analysis (Vamvatsikos and Cornell, 2001). Each building is subjected to 300 motions as each of the original 30 scaled motions are themselves scaled to intensities ranging from 30% to 250% of the 2% in 50 year motion in increments of 10% . Common types of low-rise school buildings have been analyzed for this full range of ground shaking in all regions of the province for schools on Site Class C, the reference site for the National Building Code of Canada which corresponds to firm ground. This large database of analytical results is made available to engineers assessing and retrofitting school buildings through the use of an electronic interface called the Seismic Performance Calculator, obviating the need to perform their own analyses.

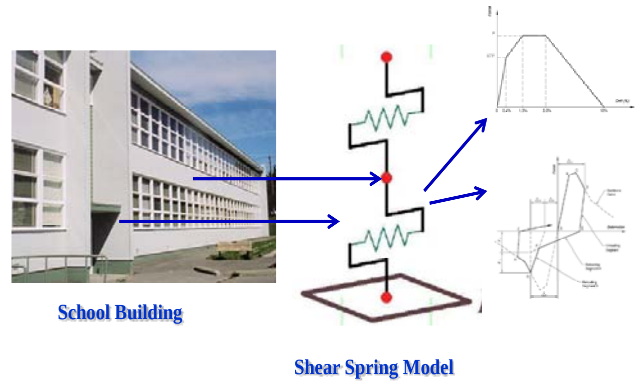

Fig. 9. Modeling a generic type school building.

There are two constraints on acceptable drift performance for a school: the absolute probability of exceeding the tolerable drift (PDE) for a specified type of school structure is 2% or less and the rate of drift exceedance (RDE), if the 100% benchmark design motions occur, is 10% or less. The latter requirement is similar to the constraint on the fragility curve to be used for the estimating risk targeted motions discussed earlier.

Calculation of PDE

The process for calculating the risk of collapse of a structural model is illustrated using the simple example of a single degree of freedom lumped mass elastic-plastic system. Collapse is defined by a maximum lateral deformation of 6 cm (equivalent to a 2% drift for a 3m-height structure). The 2% in 50 years target hazard for the site is given by the spectral acceleration at 1 second period, Sa (T=1s), of 0.2 g. Four records are selected and scaled to match the target. Incremental dynamic analyses (IDAs) of the model are performed with the four scaled records as input motions using the computer program NONLIN v. 7.05 (Charney, 1998). Each benchmark input record was scaled in intensity from 0.04g to 0.4g giving 10 records for analysis ranging from 25% to 200% for each benchmark motion. The distribution of annual frequencies of Sa(T=1s) exceedance for the site was determined by probabilistic seismic hazard analysis and are shown in Fig. 10. Note that the annual frequency of the 0.2g target hazard is around 16 × 10-4, which is equivalent to an earthquake return period of 612 years.

Table 1 shows the calculated drift values and summarizes the procedure for calculating the total risk of exceeding the collapse drift of 6 cm for this example. Lognormal distributions are assigned to the drifts specified by the mean and standard deviations of log drifts. The probabilities of exceeding the drift standard of 6 cm deformation are calculated for each level of shaking giving the conditional probabilities P(dr ≥6 cm | Sa). The frequency of occurrences of each increment, f(Sa), are derived from the annual frequencies derived from a probabilistic seismic hazard analysis. The frequencies for this example are shown in Fig. 4.

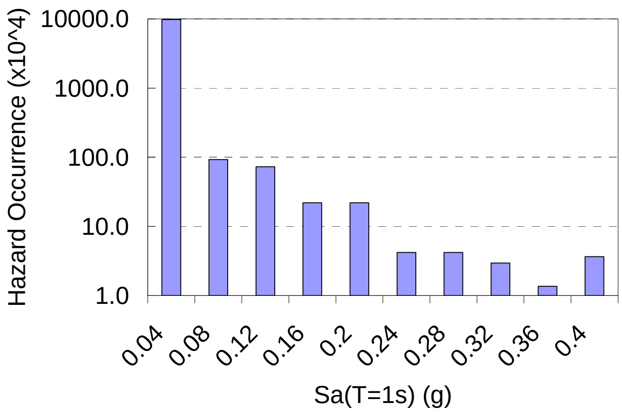

Fig. 10. Annual frequency occurrence of levels of shaking (Pina etal., 2010a).

Table 1. Example of calculation of PDE in 50 years

Sa(T=1s) g	P(dr≥6 cm| I) %	f(I) $x10^{-4}$	P(dr≥6 cm| I) x f(I) $x10^{-4}$
0	0		
0.04	0.0	9775	0.0
0.08	0.0	93	0.0
0.12	0.0	73	0.0
0.16	0.1	22	0.0
0.2	4.7	22	1.0
0.24	33.5	4	1.4
0.28	56.3	4	2.4
0.32	76.5	3	2.3
0.36	93.5	1	1.3
0.4	97.3	4	3.5
		λ	11.9
		t (yrs)	50
		PDE_{50}	6%

The last column in Table 1 shows the convolution of the conditional probability of exceeding the 6 cm deformation, P(dr≥6 cm | Sa), with the annual frequency of occurrence, f(Sa). The summation over the entire range of intensities (from 0.04g to 0.4 g) gives 11.9 × 10-4 total annual frequency, λ, of exceeding the satisfactory performance criterion of a drift of 6 cm. In other words, the 6 cm deformation will be exceeded once in the next 840 yrs. Assuming a Poisson distribution, the probability of at least one exceedance in 50 years (PDE) with an annual frequency of exceedance 11.9x10-4 is given by

$$PDE = 1-\exp(-\lambda t) = 1-\exp(\ 11.9x10^{-4} \times 50) \qquad (2)$$

This gives PDE ≈ 6%. When λ is sufficiently small the probability of at least one exceedance in t years is given by λt. This is true in the present case where λt = 11.9 x 10-4 x 50 = 0.0595 ≈ 6%.

In practice no direct calculations of PDE and RDE are necessary. PDE and RDE values for a school retrofit project are determined by the Seismic Performance Calculator operating on the data base of previous calculations. The PDE and RDE values estimated by the calculator are used to assess an existing building or check a proposed retrofit design as shown below.

Seismic Performance Calculator

The Seismic Performance Calculator, shown in Fig. 11, is the principal analytical tool of for school retrofit analysis. The tool provides the engineer access to a highly advanced, peer-reviewed analytical database without requiring the engineer to be experienced in the use of nonlinear dynamic analysis techniques. The calculator permits the engineer to quickly analyze the three principal building elements that have analytically complex behavior. These are lateral displacements resisting structures, LDRS, walls rocking out-of-plane and diaphragms. For each of these three building elements, the Calculator performs a risk assessment or a retrofit design (either basic or detailed). After making the basic parametric selections (input data), the engineer clicks on the Analysis button and the analysis results are instantly displayed. For the example shown, the performance is acceptable as the PDE is 2% and the RDE is less than 10%.

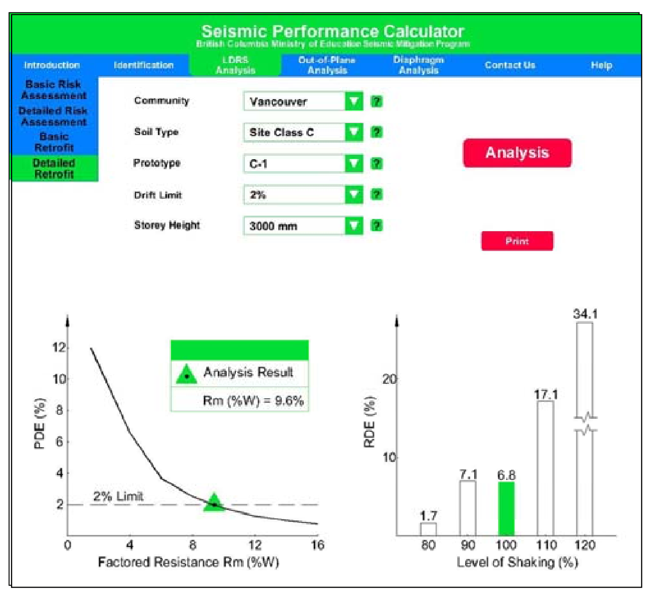

Fig. 11. Seismic Performance Calculator

Effects of Site Conditions

Site Class C (firm ground – very dense sand or soft rock) is the reference site classification used in the National Building Code of Canada and was adopted as the reference site for the retrofit program. All soils softer than firm ground are treated as one category (Site Class D / E / F) that amplifies/de-amplifies the level of shaking at the underside of the foundations relative to the response at Site Class C. The effects of these soils on structural response were evaluated for a significant number of different site conditions. The soft site responses were then expressed in terms of Site C response through the use of an Equivalent Intensity Factor (EIF) that exceeds unity for building sites in the Site Class D / E / F categories (Pina et al. 2010c). The equivalent intensity factor, EIF, is the ratio of the intensity of the motion of the reference site to ground motion intensity at the soft site to cause the same drift in the structure. Fig. 12 shows how the EIF is calculated. In this example, the reference and specific sites correspond to Site Classes C and D, respectively. IDA curves (structural Damage Measure (DM) versus input motion Intensity) are first obtained from the combined site response and structural analyses. Fig. 11 shows the incremental responses of a structural system under the j-th input motion for the two sites inputted at the level of the reference Site Class C in the site stratigraphy. For a given intensity level, the IDA curve for Site D has a larger damage measure value, which is equivalent to amplification of the structural response.

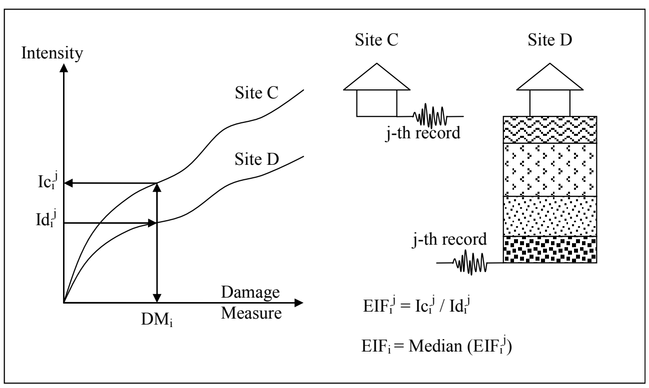

Fig. 12. Calculation process of the Equivalent Intensity Factor, EIF, for a specific site (Site D), for a given i-th intensity of the j-th record (Pina et al, 2010b).

SUMMARY

Three procedures for selecting design ground motions were presented; motions compatible with a continuous mean spectrum with ε (CMS-ε), risk targeted motions (RTGM) and motion selection based on magnitude, distance and tectonic environment.

The latter approach was used to establish ground motions for IDA analyses for assessing and retrofitting 800 schools in British Columbia. Ten ground motions were selected from worldwide records for each of the 3 types of earthquakes affecting the schools; crustal, sub-crustal and subduction motions. Because of the very different characteristics of each of these motions, the combined hazard derived from the individual hazard contributions of each earthquake type was less than the hazard resulting from a global hazard analysis based on considering all three types together as is the usual procedure.

Risk targeted motions will provide the basis for the next generation of codes in the USA. They satisfy two conditions; the probability of collapse is 1% and the conditional probability of collapse, if the design motion should occur is 10%. These probabilities are calculated using a generic fragility curve.

The uniform hazard spectrum that figures so prominently in seismic design and which has been the basis for the development of spectral compatible motions cannot approximate the spectrum of any single earthquake record. It is, in fact, the envelope of a series of individual spectra. The conditional mean spectrum scaled to a period of interest for structural response in effect singles out the individual spectrum associated with the response of interest. It is a realistic spectrum to use for selecting ground motions and scaling them appropriately. Furthermore there is evidence that CMS-ε motions give the least dispersion in the results of structural analysis.

The purpose of this paper was to acquaint the geotechnical engineer with these newer approaches to the estimation of design motions.

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