Draft Report of CSRN Theme 1 Meeting
Apr. 25-27
Listel Hotel, Vancouver

Present: Gail Atkinson, Karen Assatourians, Najib Bouanaani, Luc Chouinard, Liam Finn, Pierre Leger, Dariush Motazedian, Kristy Tiampo, Robert Tremblay, Carlos Ventura

See appended presentations for details; some useful references and websites are also listed below.

Microzonation and Liquefaction:

Presentations were made of progress in microzonation and liquefaction studies in Ottawa, Montreal and Vancouver/Victoria. Maps are being made of Vs30 and amplification for the study regions, as well as liquefaction potential. A standard format may emerge to present the results as interactive layers in Google map (UBC is following this approach and provided a demonstration).

A question was raised concerning the role and potential utility of mapping fundamental site period. We will revisit this question in Sept. Other questions concern the type of access to the maps that can be made available (what can be downloaded? By whom?); this also to be revisited. Along with the maps, we will compile an anthology of terms/techniques so that we can be clear about what is being plotted and its meaning (eg. Fundamental period based on 4Vs30/H may differ from that based on H/V; there are several ways to plot liquefaction potential, etc.).

Ground motions and time histories:

An overview of the state of practice in methods of selecting/modifying/scaling/simulating time histories for nonlinear analysis was conducted, with much stimulating discussion. On simulated records, it was agreed that true 3-component records, having the correct inter-component and intra-component (frequency-to-frequency) statistical correlations would be useful (including vertical component records). Further work will be done on this (Atkinson, Motazedian, Assatourians to report plans in Sept).

Over all, there was a feeling that ultimately we may wish to place less emphasis on "matching a target UHS" and more emphasis on selecting/simulating scenario records at an appropriate probability (change the focus of the target from the UHS to the time histories). In the present context of matching a UHS, there are many approaches, from selecting/scaling records to match a UHS (or portion thereof) or CMS, to modification of records in the time or frequency domains, to simulations. These methods range from simple to complex, and often involve subtle but critical decisions in their implementation. Our group aims to understand these methods and boil them down into simple guidance for practitioners. As a product of the CSRN, we aim to deliver a set of general (but non-prescriptive) guidelines for time histories that could improve greatly on the current

NBCC Commentary. These guidelines will include a hierarchy of the available methods and their pros and cons, with key references, and include worked examples. Leger will prepare a draft Table of Contents for these guidelines for discussion in Sept.

From Hazard to Risk:

An overview of seismic risk studies in Vancouver/Victoria and Montreal was held. Google streetview and other online and GIS tools are making inventory easier, but this is still a challenge. Tiampo to investigate insurance industry models for probabilistic treatment of inventory to fill in missing information. Some inventory information (ie. Utilities) will likely not be made available to our studies, for security reasons, potentially limiting risk applications to a focus on buildings/bridges and available information.

Inventory and risk studies are ongoing this summer in Montreal and in Richmond/North Vancouver. At present, MMI is the most useful "scenario" ground motion parameter in risk studies, but if suitable fragility curves are available, spectral ground motions could also be adopted (methodology updated as appropriate). Discussion on software platform (HAZUS?) still ongoing in the East, while the West has tools that are largely already developed from previous applications.

The current focus on MMI motivates us to explore a new avenue of collaboration within CSRN. Atkinson/Tiampo to look into feasibility of developing online Did You Feel It (DYFI) system for Canada (GSC was planning this years ago, but it has not progressed); Atkinson/Tiampo to report on DYFI at Sept. mtg. We may be able to import and make suitable modifications to USGS system to enable real-time mapping of intensity across Canada from all felt earthquakes (from citizen responses). This could be web-hosted (and mirrored) at several CSRN Universities, in London, Vancouver, Montreal, in both English and French, providing redundancy. DYFI could potentially be interfaced with real-time instrumental systems in Vancouver to aid in interpolation of intensities between monitored locations.

Some Ground-Motion References (see also www.seismotoolbox.ca)

Assatourians, K., and G. Atkinson (2010). Database of processed time series and response spectra for Canada: An example application to study of the 2005 MN5.4 Riviere du Loup, Quebec earthquake. Seism. Res. L., submitted.

Atkinson, G. (2009). Earthquake time histories compatible with the 2005 NBCC Uniform Hazard Spectrum. Can. J. Civ. Eng., **36**, 991-1000.

Atkinson, G. (2010). Impact of recent developments in ground motion prediction equations on probable ground motions for Canadian cities. Proc. 9thU.S./10thCdn.Conf.Earthq.Eng., Toronto, July 2010 (in press).

Goda, K., H. Hong and G. Atkinson (2010). Impact of using updated information on seismic hazard in Western Canada. Can. J. Civil Eng., in press.

Goda, K. and G. Atkinson (2010). Intra-event spatial correlation of ground-motion parameters using SK-net data. Bull. Seism. Soc. Am., in press.

Goda, K. and G. Atkinson (2010). Seismic performance of wood-frame houses in southwestern British Columbia. Earthq. Eng. Struct. Dyn., in press.

Goda, K. and G. Atkinson (2010). Quantitative seismic risk assessment of wood frame buildings in Richmond, B.C. 9thU.S./10thCdn.Conf.Earthq.Eng., Toronto, July 2010 (in press).
Goda, K. and G. Atkinson (2010). Impact of key uncertainties on seismic hazard assessment for Canadian cities. Bull. Seism. Soc. Am., submitted.
Goda, K., G. Atkinson, J. Hunter, H. Crowe and D. Motazedian (2010). Probabilistic liquefaction hazard analysis for Canadian cities. Bull. Seism. Soc. Am., submitted.

Contributed Presentations follow

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Funded by NSERC / *Subventionné par le CRSNG*

Project 1.2
Microzonation

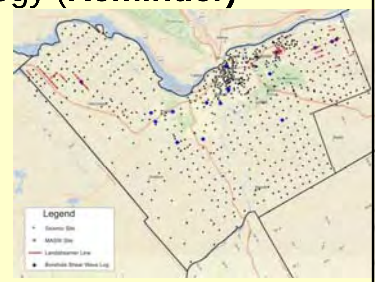
Overview of Ottawa-area studies and methodology

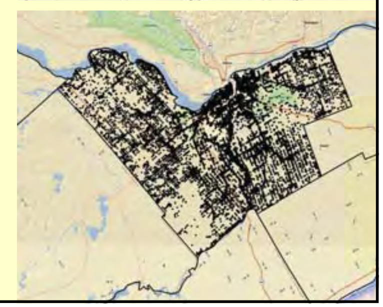
Focus Group Meetings, April 25-27, 2010, Vancouver, BC

Carleton UNIVERSITY
Canada's Capital University

Studies and methodology (**Reminder)**

- We have covered
  - 700 seismic sites
  - 25 line-km landstreamer
  - 11 borehole sites
  - 400 H/V sites
  - 43 MASW
- Compiled ~21,000 GSC Borehole Database
- Two Broadband Seismic Stations





Studies and methodology

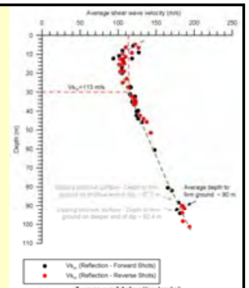
- Features of the three main geological units:

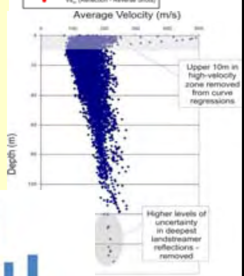
1. **Champlain Sea sediments (covers about 65% of the city)**
   – Velocity-depth function for each site was compiled

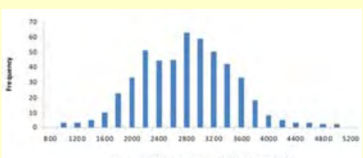
   – Velocity-depth function for all sites was compiled
     - **Vs_{av}=123.86 + 0.88z ±20.3 m/s**

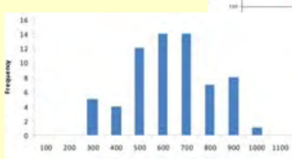
2. **Glacial till Vs: 580 ± 174 m/s**

3. **Bedrock Vs: 2700 ± 675 m/s**







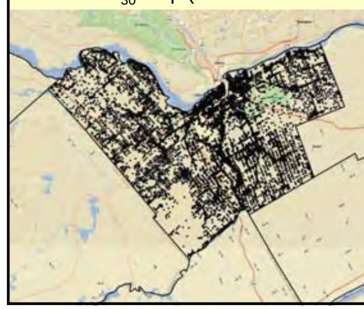


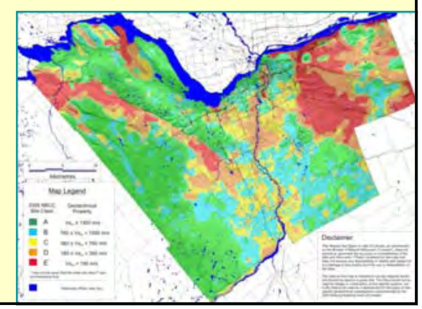
Studies and methodology (Vs30 map)

- We applied to ~21,000 GSC borehole database:
  – The velocity-depth function for Champlain Sea sediments
    - **Vs_{av}=123.86 + 0.88z ±20.3 m/s**
  – average Vs for Glacial till: **580+-174 m/s**
  – average Vs for bedrock : **2700 ± 675 m/s**

- 700 seismic sites: **site specific Vs values**

- Final Vs_{30} map (based on 2005 NBCC)

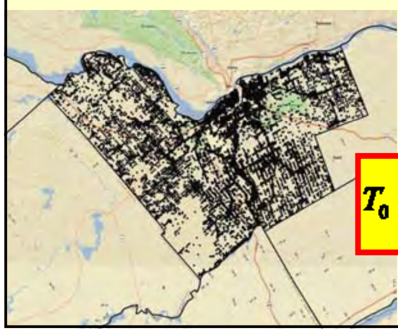




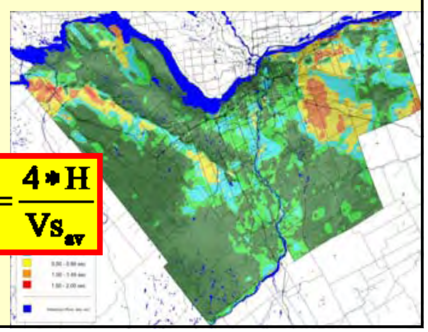
Studies and methodology: Preliminary T_0 map

- Obtained Vs_{ave} for all **700 seismic** sites and **~21,000** old boreholes
- Applied **T_0=4H/ Vs_{ave}** to all sites
- Preliminary T_0 map

- **Preliminary(?) Will be discussed later**



$$T_0 = \frac{4 * H}{Vs_{av}}$$



Studies and methodology: bedrock Vs

- Borcherdt (1992, 1994) soil amplification factors are based on the analysis results of records mainly form Loma Prieta earthquake, 1989.
    - $F_a = (1050/ Vs_{30})^{a}$
    - $F_v = (1050/ Vs_{30})^{b}$
- Note : **1050** (in m/sec) is the average shear wave velocity for bedrock **(Franciscan bedrock in California).**

- NEHRP; similar to Borcherdt approach is based on real or mapped input ground motion data (mainly from records of Loma Prieta earthquake).

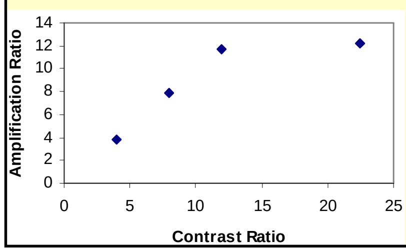
- Average Vs for Ottawa's bedrock
    - **2700 m/d (+- 650 m/s)** based on 505 measurements
        - Compare it with 1050 m/s
    - **Does this high Vs make a difference?**

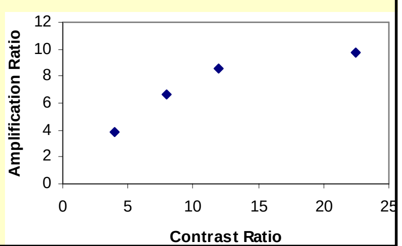
- Ottawa's Leda clay is too loose
    - **Does Q or damping of Leda clay make a difference**
    - Is Q (or damping) for Leda clay following the general equation mainly based on a database from west?

Studies and methodology: bedrock Vs

- **Does this high Vs contrast make a difference?**
- Sensitivity of Amplification factor to **shear wave contrast** ratio between the soil and bedrock
  - ORHO, 23 Gal
  - ORHO, 41 Gal

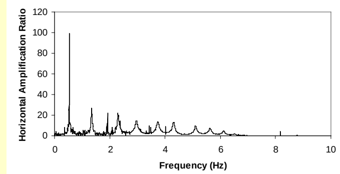
– Vs contrast ratio does make a difference!

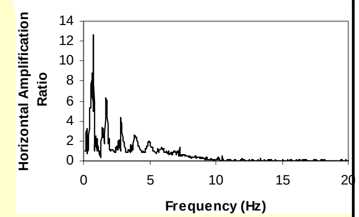


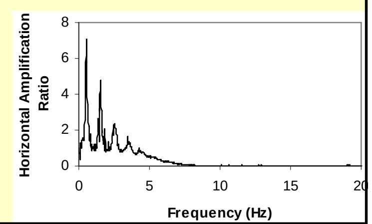


The effect of nonlinear soil

- **Sensitivity of amplification factor to input PGA**
- Weak motion
- 12 recordings with PGA of **208 Gal**, on average
- 12 recordings with PGA of **349 Gal**, on average







Studies and methodology: Soil damping

- We need to measure damping or Q which causes the nonlinearity
- Measuring Q, or Soil Damping
  - In Situ, **Spectral Ratio Method for Mono-frequency Source Approach:** 10Hz, 15 Hz, 20 Hz...120 Hz
  - Lab Tests – Resonant Column Testing

Studies and methodology: Soil damping

- Comparison with other regions
- **We do need your help for higher levels of strain**

Location	Material	Depth Range	Method	% Strain	Velocity (m/s)	Damping (%)	Q	Source
		30-50m	in situ - monofreq spectral ratios	$<10^{-6}$	225	0.36%	139	MSc thesis of H.Crow, Carleton University, current
		30-70m	in situ - monofreq spectral ratios	$<10^{-6}$	248	0.20%	250	

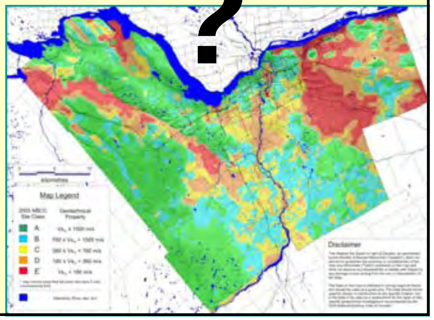
Studies and methodology: F_a, F_v, F_{f0}

- We are working on soil Amplification factor for different site class for the **Ottawa area**
  - Using Finite element method (FEM)
  - Finite difference method (FDM) is a future approach
- To obtain
  - F_a :similar to NBCC 2005 (Finn and Wightman, 2003) at 5 Hz
    - Fourier spectra analysis
    - Response spectrum analysis
  - F_v :similar to NBCC 2005(Finn and Wightman, 2003) at 1 Hz
    - Fourier spectra analysis
    - Response spectrum analysis
- F_{f0} :Amplification factor at Fundamental frequency of site !?

Studies and methodology: Gastineau

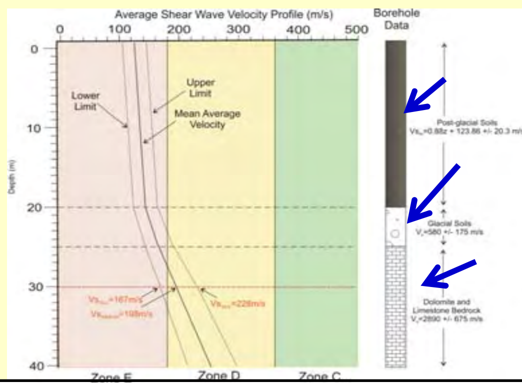
- Extending microzonation activities to Gastineau in summer 2010
  - In touch with the City
  - Hired one summer student
- Should we do the same thing?
  - Vs30 map?
  - T0 map?

?



Studies and methodology: Effects of Error

- Working on the effects of error associated with Vs30 and possibly T0 on site classification



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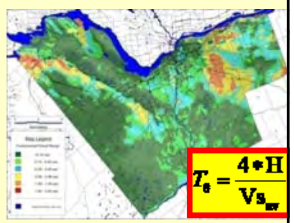
Thank You All

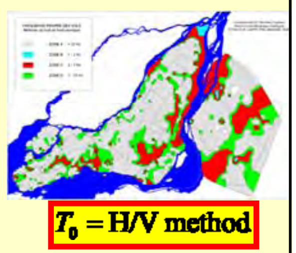
CANADA 1842 GEOLOGICAL SURVEY · COMMISSION GÉOLOGIQUE

Carleton UNIVERSITY
Canada's Capital University

Issue #1 : calibration of T0 methods

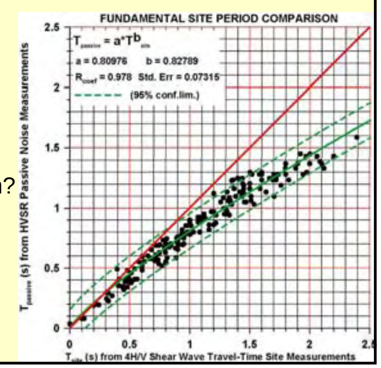
- T_0 map for Ottawa
  - **T_0** obtained based on **T_0=4H/ Vs_{ave}** for boreholes and sites with accurate H and **Vs_{ave}** (based on first arrival time)
- T_0 map for Montreal or Vancouver
  - H/V method
- Both methods are commonly used
- **But results are different especially for thick soil deposits**
- We do not know which one is better yet!
- This why we are here!





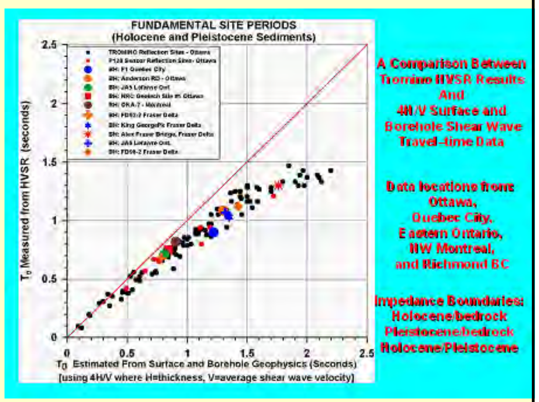
Issue #1 : calibration of T0 methods

- **Both methods were applied to about 200 sites**
  - T_0 based on **H/V, Vertical axis**
  - T_0 based on **T_0=4H/ Vs_{ave} , Horizontal axis**
- **Deviation for 1:1 line !**
- **This is problematic for thick soils**
- **Is H/V** underestimates **T_0**?
- Or
- **T_0=4H/** Vs_{ave} overestimates **T_0** ?
- Which methods gives a better estimation?



Issue #1 : calibration of T0 methods

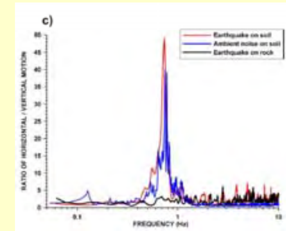
- **It is not just Ottawa !**
  - boreholes from
    - Quebec City
    - Eastern Ontario
    - NW Montreal
    - Richmond , BC
    - Ottawa



- What are the reasons?
  - Nonlinear soil?
  - Velocity gradient?
  - ???

Issue #1 : calibration of T0 methods

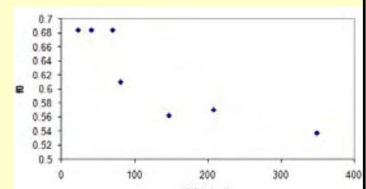
- **Is nonlinearity an important factor?**
- We applied a few methods to one of our broadband seismic station (ORHO) which we know it very well
  - 91 of soil with **accurate Vs and H**
    - Borehole data
    - Seismic refraction reflection
    - There is nearby rock seismic station



- F0 based on
  - Background noise H/V; **~0.8 Hz**
  - Earthquake H/V **~0.8 Hz**
  - Ratio to near by rock station **~0.8 Hz**

  - **4H/ Vs_{ave}** **~0.6 Hz**
  - multilayer soil profile **~0.5Hz**
  - **Which one?**

  - **A pilot FEM analysis** **~0.5-0.7Hz**



A look at relative liquefaction hazards for eastern vs. western Canada

Katsu Goda & Gail Atkinson
University of Western Ontario
Jim Hunter & Heather Crow
Geological Survey of Canada
Dariush Motazedian
Carleton University

Objectives

- Investigate relative severity of liquefaction hazard in eastern vs. western cities, based on probabilistic liquefaction hazard analysis (PLHA)
- This is based on combining reliability-based liquefaction potential evaluation using shear-wave velocity (V_s) data and probabilistic seismic hazard analysis
- Utilize updated seismic hazard models for eastern and western Canada
- Conduct PLHA for several cities across Canada and investigate the effects of regional seismic hazard characteristics on liquefaction assessment.

Probabilistic Liquefaction Hazard Analysis

Updated seismic hazard models
- **New seismic rates**
- **New ground motion prediction equations** with distance conversion
- **Probabilistic Cascadia subduction events**

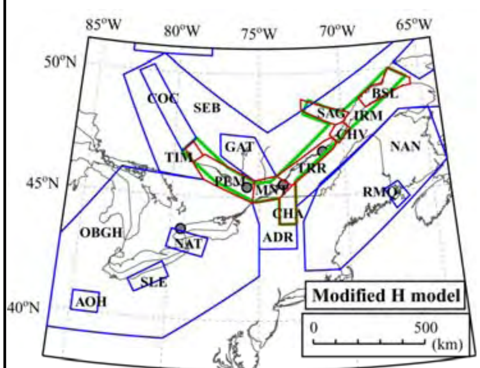
V_s-based liquefaction assessment
- Reliability-based method by incorporating **parametric uncertainty of input variables** and **model uncertainty**

Joint probability distribution of peak ground acceleration and magnitude is directly taken into account

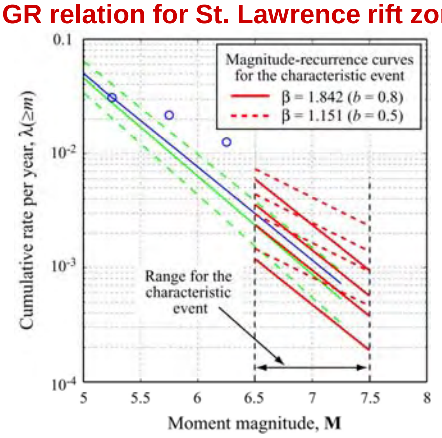
Liquefaction hazard curve
- Use the **liquefaction potential index** to account for thickness, proximity, and extent of liquefied soil layers

Updated Seismic Hazard Model (1/2)

Modified seismic source zones



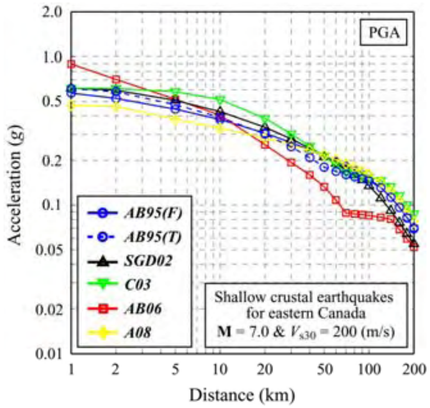
GR relation for St. Lawrence rift zone



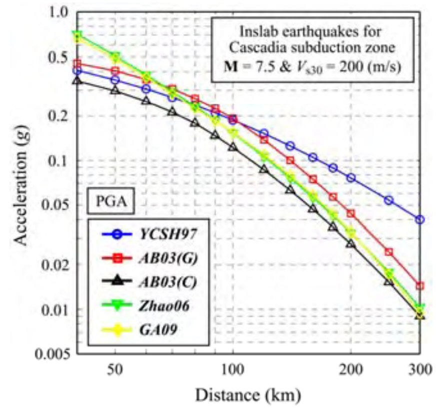
- Seismic rates are reevaluated using a longer and homogeneous *CCSC09* earthquake catalog compiled by Macias et al.
- For the **St. Lawrence rift region** (IRM, **green color**), small-to-moderate events are characterized by several GR relations for smaller zones, whereas large events are characterized by a semi-characteristic model.

Updated Seismic Hazard Model (2/2)

Eastern Canada

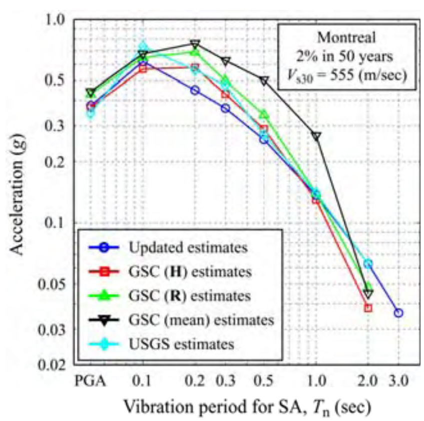


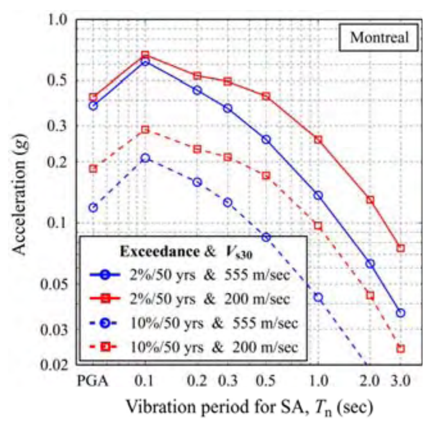
Western Canada (inslab events)



- Ground motion prediction equations have most significant impact on seismic hazard estimates.
- We consider multiple recent ground motion prediction equations to account for epistemic uncertainty regarding their selection.

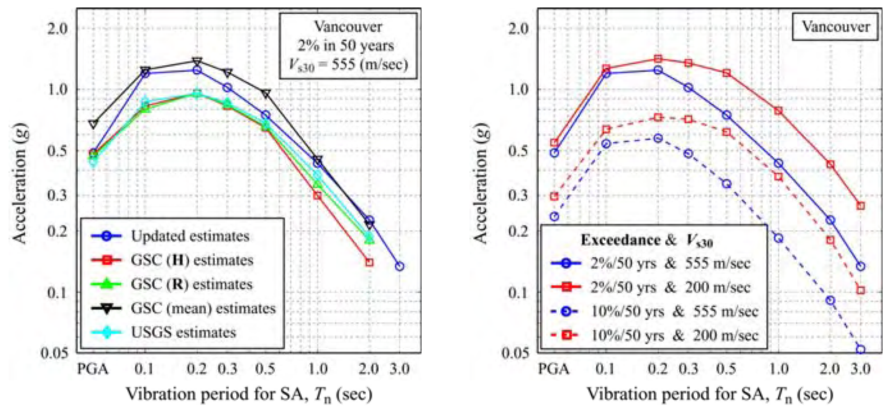
Seismic Hazard Assessment - Montreal





- Updated seismic hazard estimates for Montreal are lower than mean and median estimates based on the current GSC model.
- Soft soil condition increases seismic hazard estimates for longer vibration periods significantly.

Seismic Hazard Assessment - Vancouver

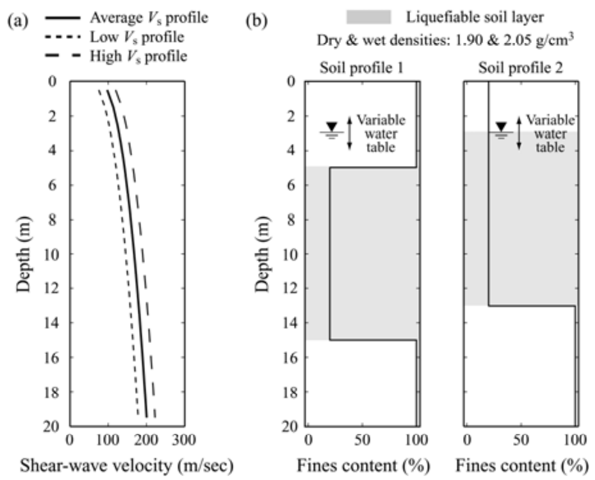


- Updated seismic hazard estimates for Vancouver lie between mean and median estimates based on the current GSC model.
- Soft soil condition increases seismic hazard estimates for longer vibration periods significantly.

V_s-based Liquefaction Potential Evaluation

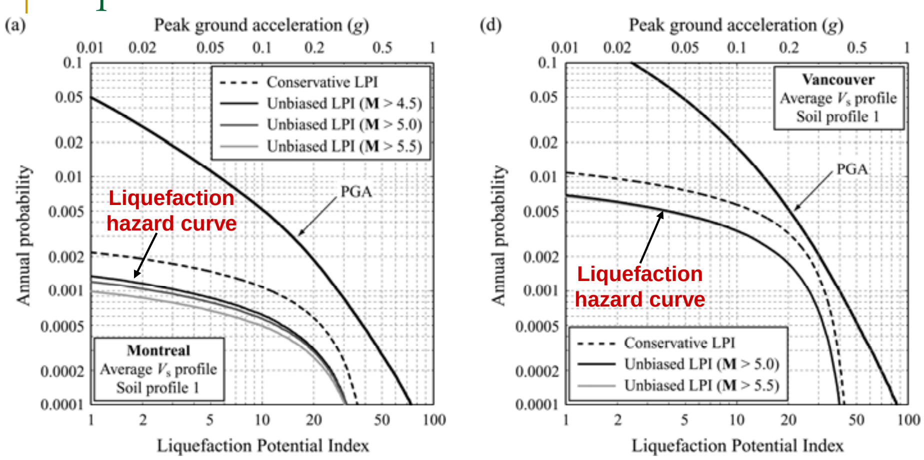
- stress-based liquefaction potential evaluation procedure (eg. Seed and Idriss) to compare cyclic stress ratio (CSR) with cyclic resistance ratio (CRR) (with adequate standardization).
- CSR model involves "peak ground acceleration (PGA)" and "moment magnitude (M)", which are inter-related – for probabilistic assessment, the joint distribution of PGA and M is necessary.
- CRR model can be expressed in terms of SPT data, CPT data, and V_s data – We adopt the V_s-based CRR model of Andrus and Stokoe.
- Recently, Juang et al. extended the conventional V_s-based CSR-CRR model into the probabilistic one using the first-order reliability method.
- As a measure of liquefaction potential, we consider the liquefaction potential index I_{LP} proposed by Iwasaki et al., but based on Juang et al.'s modification. Useful threshold values: moderate liquefaction hazard (sand boils) – $\boldsymbol{I_{LP} = 5}$, and severe liquefaction hazard (lateral spreading) – $\boldsymbol{I_{LP} = 15}$.

Representative Soil Profiles



- We consider 6 soil profiles with 3 V_s profiles over depth times 2 soil layer profiles.
- On average, both soil layer profiles have 10 m thickness of liquefiable sand layers with variable water table level between 2 and 4 m depth.

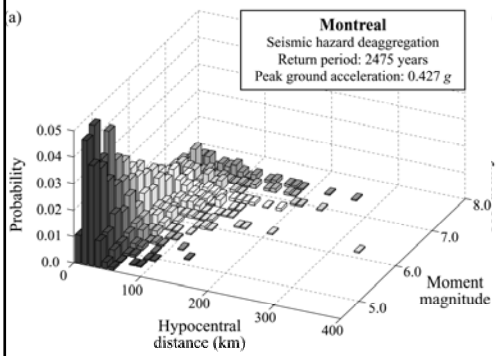
Liquefaction Hazard Curve



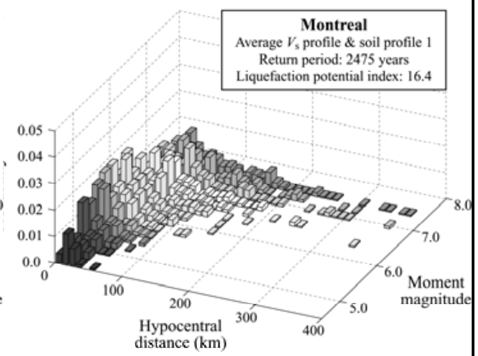
- By carrying out PLHA, a liquefaction hazard curve is obtained; this curve describes the extent of liquefaction severity as a function of annual probability.
- Liquefaction hazard curve for Vancouver is more severe than that for Montreal.

Deaggregation Analysis - Montreal

Seismic hazard deaggregation



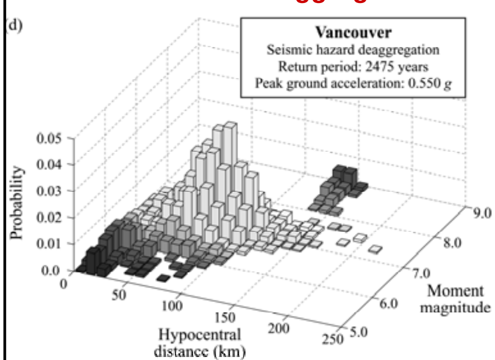
Liquefaction hazard deaggregation



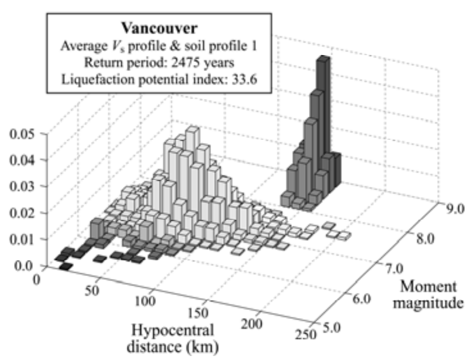
- Seismic/liquefaction hazard deaggregation shows the characteristics of contributing seismic events at a selected probability level.
- For Montreal, seismic hazard deaggregation tends to have higher contributions from smaller events – The application of the magnitude scaling factor reduces these contributions to liquefaction significantly.

Deaggregation Analysis - Vancouver

Seismic hazard deaggregation

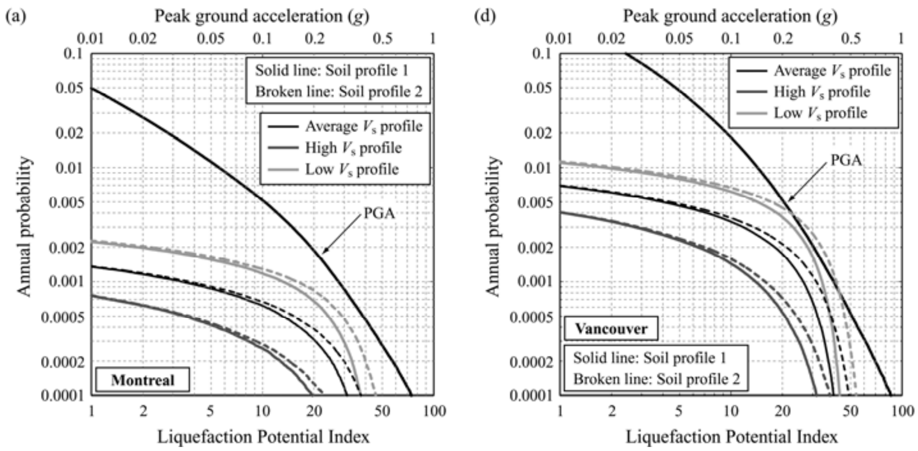


Liquefaction hazard deaggregation



- For Vancouver, liquefaction hazard deaggregation results highlight the impact of the Cascadia subduction events due to its large magnitude.
- In general, for the same seismic excitation level, more contributions due to larger magnitudes are observed for western cities than eastern cities – more significant liquefaction hazard in western cities for the same scenario than in eastern cities.

Effects of Different Soil Profiles



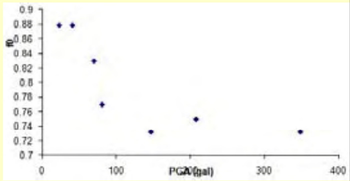
- Shear-wave velocity has significant impact on liquefaction hazard evaluation for both Montreal and Vancouver.

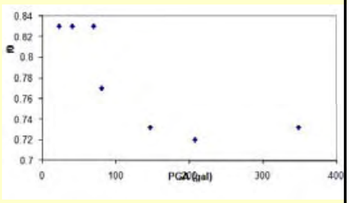
Summary and Conclusions

- Probabilistic liquefaction hazard analysis for generic eastern vs. western sites, based on a V_s-based liquefaction potential evaluation method and PSHA – joint distribution function of PGA and **M** is directly taken into account.
- Regional seismic hazard characteristics have significant impact on liquefaction hazard assessment: for the same seismic excitation level or earthquake scenario, liquefaction hazard is higher for western cities than eastern cities. This is because in western Canada, large earthquakes contribute more significantly to overall seismic hazard, in comparison with eastern Canada.

Issue #1 : calibration of T0 methods

- Is nonlinearity a factor?
  - Two more sites
  - The same trend



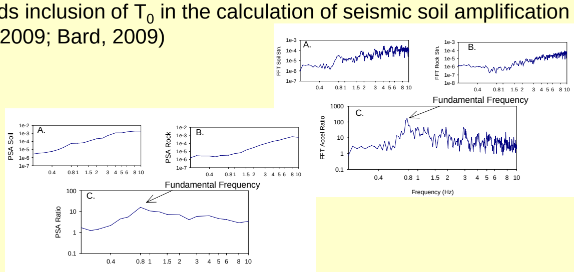


Issue #1 : calibration of T0 methods

- **Preliminary conclusion for this case :**
  - H/V method provides results closer to linear soil
  - 4H/ Vs_{ave} method provides results closer to nonlinear soil
  - Which way we should go as a team?
  - Suggestion
    - If UBC and McGill do some analysis we may get a reasonable answer to the question

Issue #2 : T0 amplification factors

- Amplification due to Vs gradient $A\,(Vs\ gradient) = \sqrt{\frac{(\rho V_s)_1}{(\rho V_s)_2}}$
- Amplification due to resonance $A' = A^2$ $A'(resonance) = \frac{(\rho V_s)_1}{(\rho V_s)_2}$
- Resonance amplification is strong when the Vs contrast between soil and rock is large
- It has been recognized that V_{s30} may not represent the entire seismic soil amplification phenomenon
- There is a trend towards inclusion of T_0 in the calculation of seismic soil amplification factors (Abrahamson, 2009; Bard, 2009)
- Observation is Ottawa
  - A soil site
  - B rock site
  - C soil to rock ratio



Issue #2 : T0 amplification factors

- Gail's question:
  - Should future approaches (including recommendations for building code applications) move towards including T0 as well as Vs30?
  - NBC soil amplification (Finn and Wightman, 2003) factors for Fa and Fv

Site Class	Values of F_a				
	$S_a(0.2) \le 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \ge 1.25$
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	(1)	(1)	(1)	(1)	(1)

Site Class	Values of F_v				
	$S_a(1.0) \le 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \ge 0.5$
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	(1)	(1)	(1)	(1)	(1)

  - **Should we be providing a similar table for T0 ?**
  - **Teamwork?**

Issue #3 : Q for higher level of starin

- **Teamwork for higher level of strain**

Location	Material	Depth Range	Method	% Strain	Velocity (m/s)	Damping (%)	Q	Source
Ottawa	silt (Leda Clay)	30-50m	in situ - monofreq spectral ratios	$<10^{-6}$	225	0.36%	139	MSc thesis of H.Crow, Carleton University, current
		30-70m	in situ - monofreq spectral ratios	$<10^{-6}$	248	0.20%	250	
		10m sample	lab - RC	10^{-6} - 10^{-5}	<100	1.81%	28	
		30m sample	lab - RC	10^{-6} - 10^{-5}	135	1.95%	26	
		70m sample	lab - RC	10^{-6} - 10^{-5}	220	1.40%	36	
Fraser Delta	silt	22-35m	in situ - spectral ratio from SCPT	small strain	202	0.30%	167	PhD thesis, Stewart, P., UBC, 1992
		18-25m	in situ - spectral ratio from SCPT	small strain	179	0.40%	125	
	clay	6-12m	in situ - spectral ratio from SCPT	small strain	101	0.80%	63	
		5-13m	in situ - spectral ratio from SCPT	small strain	102	0.70%	71	
		8-12m	in situ - spectral ratio from SCPT	small strain	119	0.80%	63	
Fraser Delta	clay		lab-RC (UBC)	10^{-3}		0.9% - 2.4%	21 - 56	MSc thesis, Zavoral, D., UBC, 1990
Other Soil Types, Higher Strains:								
U Berkley	cohesionless soils		lab	10^{-3} - 10^{-4}		0.5% - 2%	25 - 100	Seed et al, 1986
U Berkley	cohesive soils		lab	10^{-3}		1% - 5%	10 - 50	Sun et al, 1988
Monterey, Calif	sand		lab	10^{-3}		1.00%	10	Saxena and Reddy, 1989
	sand		lab	10^{-3}		1.50%	33	Ishihara, 1982

1

OVERVIEW OF RECENT LITERATURE ON SELECTING RECORDS FOR NL ANALYSIS

P. Léger – École Polytechnique de Montréal

GOAL – DEVELOP GUIDELINES TO SELECT (DEFINE) GROUND MOTION RECORDS FOR NL ANALYSIS OF BUILDING STRUCTURES – COMMENTARY TO NBCC / GUIDELINES

OBJECTIVES - EXAMINE WHAT OTHER GROUPS AND RESEARCHERS ARE CURRENTLY RECOMMANDING FOR SELECTING RECORDS FOR NL ANALYSIS

SOURCES OF INFORMATION

1. RESEARCH PAPERS - CONFERENCE PRESENTATIONS
2. RECENT GUIDELINES

2

NBC 05 - Commentary (Guidelines) - FACTUAL

- **Ground motion time-histories** having spectra which are **compatible** with the specified design spectral acceleration values
- A time-history is deemed to be "spectrum-compatible" if its response spectrum **equals or exceeds the target spectrum throughout the period range of interest**, i.e. the periods of the modes contributing to the response of the particular structure (Naeim and Lew 1995).
- There need to be **sufficient time-histories** used to enable uncertainties in ground motion parameters (e.g. durations) to be reflected in the dispersion of the resulting response parameters
- Spectrum-compatible time-histories may be obtained by **scaling and/or modifying actual recorded earthquake accelerograms** or by **creating artificial or synthetic time-histories.**
- If actual earthquake accelerograms are used, then these should be **scaled so that the spectral acceleration at the fundamental period of the structure corresponds to the design spectral response acceleration** for the particular site. The spectral acceleration ordinates at the **periods below** the fundamental period should also be equal to or greater than those of the design spectral response acceleration S(T) for those periods.

3

PDF – SUMMARY - FILE

Code Requirements for the Selection and Scaling of Ground Motion Records

COSMOS Annual Meeting
November 18, 2005

Charles Kircher, Ph. D., P.E.
Kircher & Associates
Palo Alto, California

Summary and Conclusion

- Seismic Codes, such as *ASCE 7-05*, have well established methods for selecting and scaling earthquake records (aka time histories) to match DBE and/or MCE design response spectra
  - Methods have evolved slightly, but are essentially the same as those first developed by SEAOC for base-isolated structures (as contained in Appendix 1L of the 1990 SEAOC *Blue Book*)
- Primary difficulty with (time-domain) scaling records is the requirement to envelop (within 10%) design response spectra over a broad range of periods (and frequency-domain scaling is not considered a desirable alternative to time-domain scaling)
  - Possible solution (when a sufficiently large number of records are used – e.g., at least 7 records) – Scale records to match a specific period of interest (e.g., S_{M1} or dominant period of structure) and use more liberal matching criteria at other periods

Soil Dynamics and Earthquake Engineering 30 (2010) 157–169

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

Selection of earthquake ground motion records: A state-of-the-art review from a structural engineering perspective

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ARTICLE INFO

Article history:
Received 19 March 2009
Received in revised form
14 October 2009
Accepted 15 October 2009

Keywords:
Ground accelerations
Strong ground motion
Selection of real records
Seismological parameters
Response spectrum
Spectral matching
Intensity measures
Inelastic response
Design codes

ABSTRACT

This paper reviews alternative selection procedures based on established methods for incorporating strong ground motion records within the framework of seismic design of structures. Given the fact that time history signals recorded at a given site constitute a random process which is practically impossible to reproduce, considerable effort has been expended in recent years on processing actual records so as to become 'representative' of future input histories to existing as well as planned construction in earthquake-prone regions. Moreover, considerable effort has been expended to ensure that dispersion in the structural response due to usage of different earthquake records is minimized. Along these lines, the aim of this paper is to present the most recent methods developed for selecting an 'appropriate' set of records that can be used for dynamic analysis of structural systems in the context of performance-based design. A comparative evaluation of the various alternatives available indicates that the current seismic code framework is rather simplified compared to what has actually been observed, thus highlighting both the uncertainties and challenges related to the selection of earthquake records.

Contents

1. Introduction

It is now well established that elastic analyses of structures subjected to seismic actions, typically in the form of response spectra, do not always predict the hierarchy of failure mechanisms. It is also not possible to quantify the energy absorption and

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doi:10.1016/j.soildyn.2009.10.005

Spectral matching with the conditional mean spectrum by utilizing ε (CMS-ε) may help widen the range of acceptable records for nonlinear dynamic analysis, because selected accelerograms may no longer have the appropriate (M, R, ε) values, but only posses a spectral shape that matches the mean spectrum with the casual event. Furthermore, the proposed CMS-ε measure seems better suited for use in design and probabilistic assessment of structures in comparison with the Uniform Hazard Spectrum (UHS), which is nowadays the most frequently used target spectrum in seismic structural analysis. It should be noted that many researchers [13,106,107] doubt that the UHS can be considered as a spectrum of a single earthquake event rather than an envelope of the spectra corresponding to different seismic sources. Therefore, use of UHS may result in designing for an unjustifiably conservative scenario of earthquakes occurring due to different seismic sources acting simultaneously [98]. In sum, the CMS-ε measure helps eliminate this conservatism.

Following the above line of thought, it is proposed to relax the prescribed period range from $(0.2T_1-2.0T_1)$ to $(T_L-1.5T_1)$, where T_L defined as previously, at least for structures designed for moderate ductility, in order to increase the number of records available for dynamic analyses and lessen the dominance of severe strong motion records on inelastic response and on the subsequent dispersion in the response quantities. Further investigation is certainly required until reaching a balance between earthquake record selection efficiency and design reliability.

4. Conclusions

This review presented various methodologies by which rational decisions can be made regarding the time-dependent earthquake input to be used for transient dynamic analysis of a structural system built in seismically prone regions. It can be concluded that there quite a few ways to achieve record selection, but it is still not possible to limit the bounds of the ensuing structural response dispersion uniformly. Moreover, despite much progress made, these record selection techniques have not yet been included in contemporary seismic code provisions. Because of that, seismic design codes used nowadays present a rather simplified version of the full picture when it comes to assessing seismically induced loads, which may or may not be commensurate with the detailed numerical modeling effort often expended in representing the structural system. In sum, seismic loading code provisions are adequate for a large class of conventional structures. This, however, may not be true for more complex situations which require sound engineering judgment, in addition to competence in setting up an adequate structural model, determining the seismic input and interpreting the response output.

Acknowledgement

Financial support to the first author through the Third EU-GR Framework program (EPEAK-MIS-91367) of the Hellenic Ministry for Education supporting graduate studies is acknowledged.

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Quantification of Building Seismic Performance Factors

ATC-63 Project Report - 90% Draft

FEMA P695 / April 2008

FEMA

nehrp

6.2 Input Ground Motions

Nonlinear response is evaluated for a set of pre-defined ground motions that are systematically scaled to increasing intensities until median collapse is established.

6.2.1 Ground Motion Hazard

Collapse safety is evaluated relative to ground motion intensity associated with Maximum Considered Earthquake (MCE), as defined in ASCE/SEI 7-05 (ASCE 2006a), and used as a basis for design. The MCE ground motion intensity is typically defined as rare ground motions (recurrence periods on the order of 1000 to 2500 years) that incorporate adjustment factors to account for local site conditions (F_a and F_v) and near field effects. As in ASCE/SEI 7-05, ground motion intensity is defined in terms of spectral acceleration.

For collapse assessment, ground motion levels correspond to maximum and minimum seismic criteria of the Seismic Design Category (SDC) for which a system is qualified. Figure 6-2 shows maximum and minimum MCE ground motion spectral intensities for Seismic Design Categories B, C and D. In all cases, site conditions are based on Site Class D (stiff soil). Table 6-1A and Table 6-1B provide specific values of short-period and 1-second spectral accelerations, respectively, for these categories.

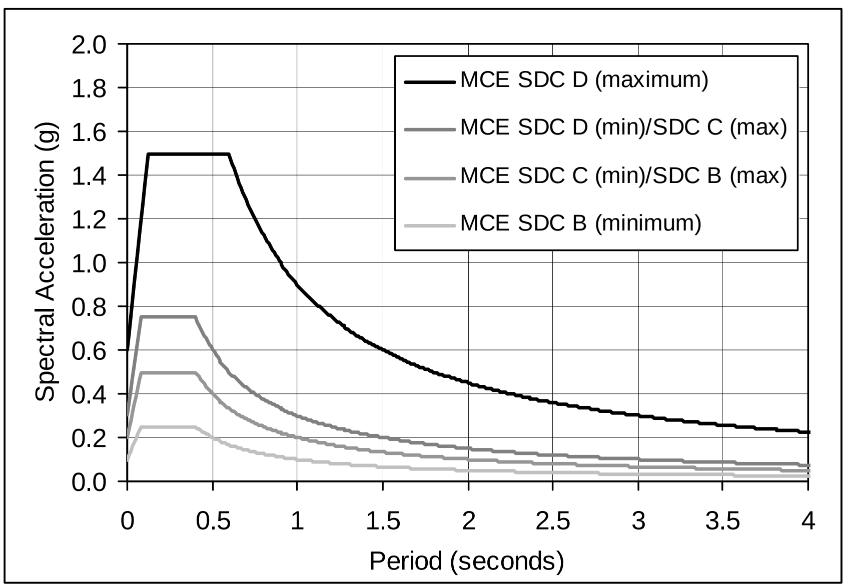

Figure 6-2 MCE response spectra for collapse evaluation of structure archetypes for Seismic Design Categories B through D.

Table 6-1A Summary of Short-Period Spectral Acceleration, Site Coefficients and Design Parameters Used for Collapse Evaluation of Seismic Design Category D, C and B Structure Archetypes, Respectively

Seismic Design Category		Maximum Considered Earthquake			Design
Maximum	Minimum	S_S (g)	F_a	S_{MS} (g)	S_{DS} (g)
D		1.5	1.0	1.5	1.0
C	D	0.55	1.36	0.75	0.50
B	C	0.33	1.53	0.50	0.33
	B	0.156	1.6	0.25	0.167

Table 6-1B Summary of 1-Second Spectral Acceleration, Site Coefficients and Design Parameters Used for Collapse Evaluation of Seismic Design Category D, C and B Structure Archetypes, Respectively

Seismic Design Category		Maximum Considered Earthquake			Design
Maximum	Minimum	S_1 (g)	F_v	S_{M1} (g)	S_{D1} (g)
D		0.60	1.50	0.90	0.60
C	D	0.132	2.28	0.30	0.20
B	C	0.083	2.4	0.20	0.133
	B	0.042	2.4	0.10	0.067

6.2.2 Ground Motion Record Sets

Two sets of ground motion records are provided for collapse assessment using nonlinear dynamic analysis. One set includes twenty-two ground motion record pairs from sites located greater than or equal to 10 km from fault rupture, referred to as the "Far-Field" record set. The other set includes twenty-eight pairs of ground motions recorded at sites less than 10 km from fault rupture, referred to as the "Near-Field" record set. While both Far-Field and Near-Field record sets are provided, only the Far-Field record set is required for collapse assessment. This is done for reasons of practicality, and in recognition of the fact that there are many unresolved issues concerning the characterization of near-fault hazard and ground motion effects. The Near-Field record set is provided as supplemental information to examine issues that arise due to near-fault directivity effects, if needed.

The ground motion record sets include records from all large-magnitude events in the PEER NGA database (PEER, 2006). Records were selected to meet a number of sometimes conflicting objectives. To avoid event bias, no more than two of the strongest records are taken from each earthquake, yet the record sets have a sufficient number of motions to permit statistical evaluation of record-to-record (RTR) variability and collapse fragility. Strong ground motions were not distinguished based on either site condition or source mechanism.

Due to inherent limitations in available data, no single set of records can fully meet all desired objectives. Large magnitude events are rare, and few existing earthquake ground motion records are strong enough to collapse large fractions of modern, code-compliant buildings. In the United States, strong-motion records date back to the 1933 Long Beach Earthquake, with only a few records obtained from each event until the 1971 San Fernando Earthquake.

Even with many instruments, existing strong motion instrumentation networks (e.g., Taiwan and California) provide coverage for only a small fraction of all regions of high seismicity. Considering the size of the earth and period of geologic time, the available sample of strong motion records from large-magnitude earthquakes is still quite limited, and potentially biased by records from more recent, relatively well-recorded events. Due to the limited number of very large earthquakes, and the frequency ranges of ground motion recording devices, the ground record sets are primarily intended for buildings with natural (first-mode) periods less than or equal to 4 seconds. Thus, the record set is not necessarily appropriate for tall buildings with long periods.

The record sets, and background information on their selection, are included in Appendix A.

6.2.3 Ground Motion Record Scaling

Ground motions are scaled to represent a range of earthquake intensities up to collapse level ground motions. Record scaling involves two steps. First, individual records in each set are "normalized" by their respective peak ground velocities, as described in Appendix A. This step is intended to remove unwarranted variability between records due to inherent differences in event magnitude, distance to source, source type and site conditions, without eliminating record-to-record variability. Second, normalized ground motions are collectively scaled (or "anchored") to a specific ground motion intensity such that the median spectral acceleration of the record set matches spectral acceleration at the fundamental period of the structure being analyzed.

The first step was performed as part of the ground motion selection process, so the record sets contained in Appendix A already reflect this normalization. The second step is performed as part of the analysis procedure. This two-

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2008

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by assuming that spectral acceleration is inversely proportional to period and anchoring spectral demand at a period of 3 or 4 seconds.

6. Geomean spectral demands can be substantially smaller than maximum spectral demands and substantially greater than minimum spectral demands. The ratio of maximum to geomean demands can exceed 1.3 in the long period range (Huang et al., 2008).

7. Near source effects can have a significant impact on spectral demands in the long period range. Care must be taken to adequately account for these effects in seismic hazard studies for sites situated within 15km of known active faults. Within 3km of active faults, maximum demands are generally oriented perpendicular to the strike of the fault for large magnitude earthquakes (Huang et al., 2008).

The mean geomean spectrum that is produced by PSHA should be adjusted for the maximum direction of shaking for response spectrum analysis using the procedures to be adopted by the United States Geological Survey in the 2009 seismic hazard maps for the United States. The short- and long-period multipliers on geomean spectral demands at 1.1 and 1.3, respectively, and are based on the studies reported in (Huang et al., 2008).

The site-specific spectrum for maximum shaking, which was developed for a reference site class, must be converted to a free-field or surface spectrum. The conversion is achieved using either short or long period site class modifiers (see ASCE 41-06) or site-response analysis, which is discussed in Section 3.3. If the site-class modifiers are to be used, the reference spectral values of bedrock motion are those of the mean geomean spectrum.

3.3 Site Response

For hard and soft rock sites, with shear wave velocities in the upper 30m of 760 m/sec or greater, site amplification of bedrock motion effects are generally small and are ignored in the hazard assessment. For firm soil and soft soil sites, a more robust procedure for establishing seismic demands is to conduct a site response study, wherein bedrock motions are transmitted upwards by vertically propagating shear waves through nonlinear soil layers. More sophisticated (and computationally intensive) 3-dimensional methods simulating the entire wave propagation process from fault to site are now beginning to emerge.

For the design of high-rise buildings on softer sites with deep and massive foundations and basements, one key issue is what motions are appropriate for the design of the building, given the variation of motions with depth in the ground. This is discussed further in section 4. These so-called foundation motions may be substantially different from the free-field surface motions predicted by a seismic hazard assessment.

A site response study should also identify the potential for liquefaction at depth, slope instabilities and other geo-seismic hazards.

3.4 Selection and Modification of Earthquake Histories for Response-History Analysis

Although acceleration response spectra can be used directly for elastic design using modal analysis, nonlinear response-history analysis requires the use of sets of ground motion records. Some modification of recorded real ground motions is generally necessary to assess the performance of a tall building because the spectral content of a given earthquake record is unlikely to be similar to that of the target spectrum.

There is no consensus on the best procedures for the selection and scaling of earthquake ground motion records (time series). The topic is the subject of significant study at this time and results will vary with the degree of inelastic response in the building for the chosen level of seismic hazard. Herein, it is assumed that the degree of inelastic response is limited and is less than that assumed for low and medium rise code compliant buildings subjected to maximum earthquake shaking.

The modification process typically generates a family of ground motion records that have similar response spectra to the target UHS over a wide range of natural periods. This process is conservative because a UHS is generally composed of spectral contributions from multiple sources, earthquake magnitudes, and site-to-source distances—no single combination of source, magnitude, and distance dominates the entire spectrum in most cases. Baker and Cornell (2006) developed the conditional mean spectrum to address this issue.

Alternate procedures may be used to select and scale ground motions for response-history analysis. The selected records must capture the distribution of spectral demand across the period range of interest in each principal horizontal direction, which will generally be between the period of the fourth translational mode and 1.5 times the fundamental translational mode.

Three acceptable procedures are presented below; other robust procedures may be used. For each of these procedures it is assumed that maximum, geomean and minimum spectra have been generated for the collapse-level assessment using the procedures presented in Section 3.3

Procedure 1: Matching to the maximum spectrum

Spectrally matched ground motion records should produce the same spectral response (+10%, -5%) as the maximum spectrum for all the important translational modes of the tall building. The ground motions should be matched in the time domain from a period of 0 second to a period of 1.5 times the fundamental translational period of the building.

The seed pairs of motions for spectral matching should be representative of the modal de-aggregation of the UHS at the fundamental period of the building. Each component in each pair shall be matched to the maximum spectrum.

Three pairs of motions should be matched to the maximum spectrum. Response-history analysis using this procedure will involve three analyses using simultaneous application of each component in the pair along the principal horizontal axes of the building.

Procedure 2: Matching to the maximum and minimum spectra

Spectrally matched ground motion records should produce the same spectral response (+10%, -10%) as the maximum and minimum spectra for all the important translational modes of the tall building. The ground motions should be matched in the time domain from a period of 0 second to a period of 1.5 times the fundamental translational period of the building. The seed pairs of motions for spectral matching should be representative of the modal de-aggregation of the UHS at the fundamental period of the building. One component in each pair shall be matched to the maximum spectrum; the other component shall be matched to the minimum spectrum. Three pairs of motions should be generated using this procedure. An additional three pairs should be then be developed by rotating the components 90 degrees.

Response-history analysis using this procedure will involve 6 analyses using the 6 pairs of ground motions. For each analysis, each component in the pair shall be applied simultaneously to the building model. The use of Procedure 2 will entail more computational effort than Procedure 1 but using less onerous earthquake demands.

Procedure 3: Matching to maximum and minimum conditional mean spectra

This procedure is more computationally intensive than Procedure 2 but recognizes that the conditional mean spectrum (CMS) as proposed by Baker and Cornell (2006) better characterizes recorded ground motions than the UHS, which is produced by PSHA.

Three CMS should be developed from the mean geomean UHS using the procedures of Baker and Cornell. In aggregate, the three CMS should envelope the UHS over the period range of 0 second to 1.5 times the fundamental translational period of the building. The ordinates of the long period CMS shall not fall below the UHS in the period range between 1.0 and 1.5 times the fundamental translational period of the building.

The ordinates of the three CMS so developed shall be increased and decreased by the Huang et al. (2008) factors relating maximum, geomean and minimum shaking to generate three sets of maximum and minimum CMS.
A total of nine pairs of ground motions will be generated using Procedure 3: three pairs for each CMS.

For each CMS, the seed pairs of motions for matching should be representative of the modal de-aggregation of the UHS at the anchor point for the CMS (e.g., the fundamental translation period of the building for the long period CMS). One component in each pair shall be matched to the maximum spectrum; the other component shall be matched to the minimum spectrum.

Response-history analysis using this procedure will involve 18 analyses using the nine pairs of CMS-compatible ground motions. The nine pairs of ground motions developed above shall be rotated 90 degrees to generate the second family of nine earthquake histories for response analysis. For each analysis, each component in the pair shall be applied simultaneously to the building model.

PACIFIC EARTHQUAKE ENGINEERING
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Evaluation of Ground Motion Selection and Modification Methods: Predicting Median Interstory Drift Response of Buildings

PEER Ground Motion Selection and Modification Working Group

Curt B. Haselton, Editor
California State University, Chico

PEER 2009/01
JUNE 2009

ABSTRACT

Nonlinear dynamic analysis of structures is becoming increasingly prevalent in code and regulatory documents prescribing design and analysis. A recurring challenge for both practicing engineers and developers of such documents is the selection and modification of ground motions for these nonlinear dynamic analyses. Nonlinear structural response is often highly sensitive to the selection and modification of input ground motions, and many ground motion selection and modification (GMSM) methods have been proposed. No systematic studies exist that provide impartial guidance to engineers regarding appropriate methods for use in a specific analysis application; thus engineers are left to make an important decision that is virtually uninformed.

The purpose of this report is to provide the engineering community with a foundation, backed by comprehensive research, for choosing appropriate ground motion selection and modification methods for predicting the median drift response of buildings. To this end, the approach taken in this report is (a) to select and scale ground motions using a wide variety of proposed methods, (b) to use these ground motions as inputs to nonlinear dynamic structural analyses, and then (c) to study differences in the resulting structural response predictions in order to identify what GMSM decisions are most crucial. By studying a large number of GMSM methods and analyzing a variety of structures, this report quantitatively compares many of the GMSM methods available to the engineering community.

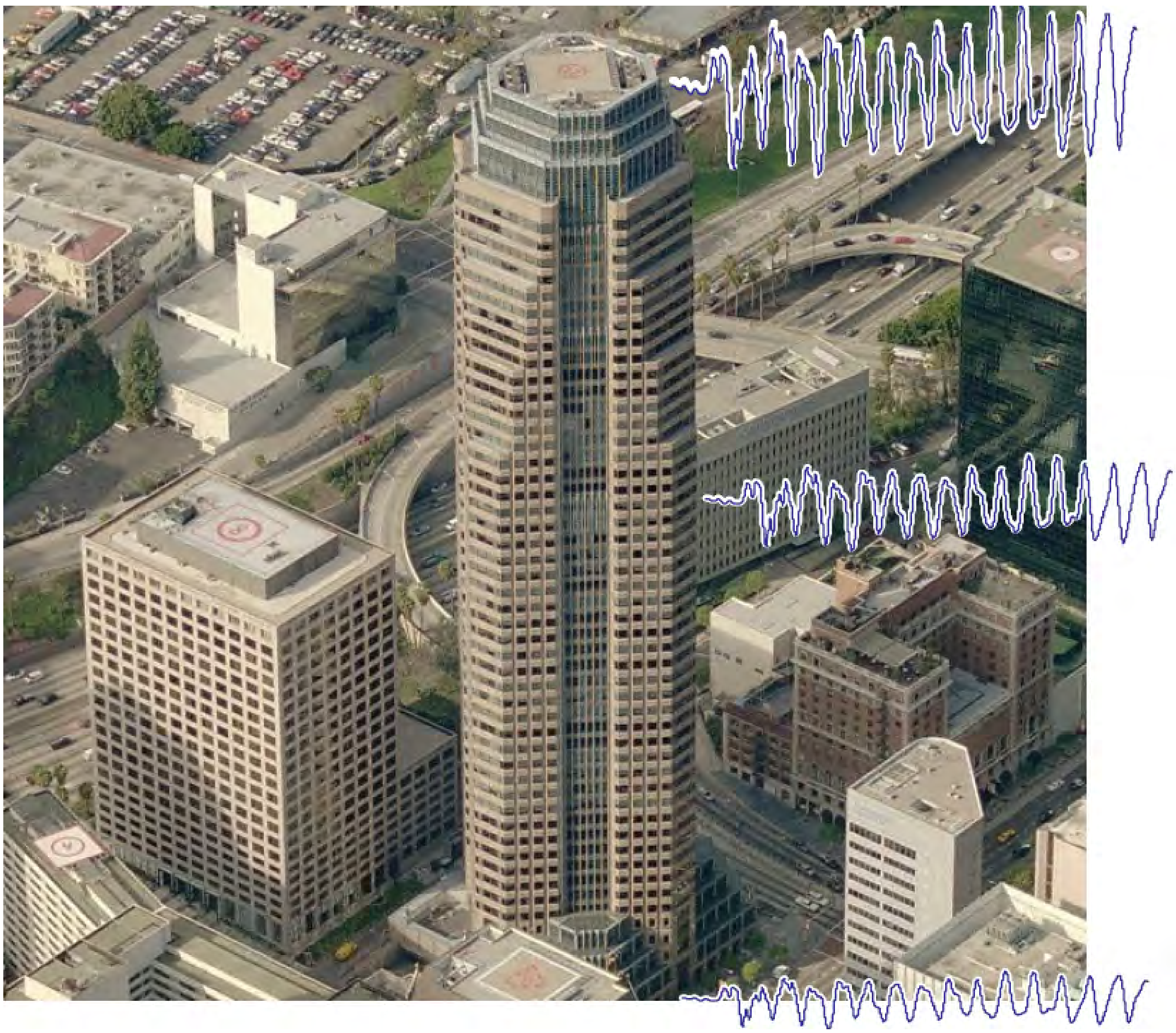
This report presents the methodology developed by the GMSM Program and the results obtained using 14 ground motion selection and modification techniques (25 if variations of those 14 are considered separately) to analyze four reinforced concrete frame and wall buildings. The results show that for the classes of buildings considered here, one can improve the prediction of structural response by appropriately taking into account higher-mode and nonlinear properties (in addition to elastic first-mode properties) of the buildings when selecting and scaling ground motion records. This is often accomplished through selection based on appropriate spectral shape, or through use of inelastic methods. The specific results of this report are intended to provide practical guidance for those selecting and scaling ground motions for buildings, and the overall methodology provides a general framework for future evaluation of other ground motion selection and scaling techniques and other classes of engineered structures.

The PEER Ground Motion Selection and Modification Program plans to continue these types of evaluations in order to bring further quantitative rigor to the use of ground motions for

the analysis of buildings, and also to initiate such research for a wider range of engineering problems (e.g., bridges, nuclear structures, earthen dams, site response). This report should thus be considered as an initial building block toward future studies that will grow increasingly comprehensive.

Practical Guidelines to Select and Scale Earthquake Records for Nonlinear Response History Analysis of Structures

By Erol Kalkan and Anil K. Chopra



Open-File Report 2010

U.S. Department of the Interior
U.S. Geological Survey

Abstract

Earthquake engineering practice is increasingly using nonlinear response history analysis (RHA) to demonstrate performance of structures. This rigorous method of analysis requires selection and scaling of ground motions appropriate to design hazard levels. Presented herein is a modal-pushover-based scaling (MPS) method to scale ground motions for use in nonlinear RHA of buildings and bridges. In the MPS method, the ground motions are scaled to match (to a specified tolerance) a target value of the inelastic deformation of the first-mode inelastic SDF system whose properties are determined by first-mode pushover analysis. Appropriate for first-mode dominated structures, this approach is extended for structures with significant contributions of higher modes by considering elastic deformation of higher-mode SDF systems in selecting a subset of the scaled ground motions. Based on results presented for two bridges and six actual buildings, covering low-, mid-, and high-rise building types in California, the accuracy and efficiency of the MPS procedure are established and its superiority over the ASCE 7-05 scaling procedure is demonstrated.

ASCE/SEI 43-05

Appendix - B

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Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities

This document uses both the International System of Units (SI) and customary units.

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TABLE 2-1. Design Response Spectrum Parameters

SDC	H_D	P_F	R_P	DF_1	α
3	4×10^{-4}	$\sim 1 \times 10^{-4}$	4	0.8	0.40
4	4×10^{-4}	$\sim 4 \times 10^{-5}$	10	1.0	0.80
5	1×10^{-4}	$\sim 1 \times 10^{-5}$	10	1.0	0.80

$$R_P = \frac{\text{Mean Annual Hazard Exceedance Frequency}}{P_F} = \frac{H_D}{P_F}$$

The DRS is defined at the same control location in the free field as that at which the hazard curve and the UHRS are defined. Provisions are given in Section 2.3 for defining this DRS at other locations in the site profile.

Minimum values of DRS Peak Ground Acceleration (PGA) at the foundation level are

0.06 g for SDC-3
0.08 g for SDC-4
0.10 g for SDC-5

2.2.2 Vertical Ground Motion

Vertical ground motion shall be developed following the provisions of ASCE 4.

SECTION 2.3 METHOD TO DEFINE THE DESIGN RESPONSE SPECTRA AT VARIOUS DEPTHS IN THE SITE PROFILE

This section presents provisions for defining the DRS at other locations in the site profile besides the control location. The control location is typically defined at the bedrock outcrop. The free surface at the top of the soil profile is the most common location at which the DRS is to be determined. The DRS may also be determined at other locations in the profile following these procedures. Prior to performing the site response evaluations, the characteristic earthquakes (magnitudes and distances) at frequencies of 1 Hz and 10 Hz associated with the UHRS at the control location shall be obtained. The selection of these bounding frequencies is considered appropriate for the relatively stiff structures typical of nuclear facilities. The spectral shapes associated with these characteristic events shall then be scaled to the UHRS at 1 Hz and 10 Hz, respectively. If the envelope spectrum associated with these two scaled spectra does not fall more than 10% below the UHRS at any frequency in the frequency range of interest, site response evaluations can be performed for these two bedrock spectra. If the envelope spectrum does fall below the UHRS at some intermediate frequency, a third intermediate spectral shape shall be determined using the characteristic event appropriate for that intermediate frequency. This spectral shape shall then be scaled to the UHRS at the intermediate frequency.

The approach for obtaining the DRS at the ground surface (or at some other intermediate depth) is summarized as follows:

(a) Convolve the UHRS at the hazard mean annual exceedance probability, H_D, at depth to obtain the corresponding UHRS at H_D at the ground surface (or other location in the soil column) using site-specific soil properties.
(b) Convolve the UHRS at $0.1H_D$ at depth to obtain the corresponding UHRS at $0.1H_D$ at the surface (or other location).
(c) Determine the slope factor, A_R, from the ratio of $UHRS_{0.1H_D}/UHRS_{H_D}$ at the ground surface, computed over the spectral frequency range, frequency by frequency, using Eq. (2-2).
(d) Use Eq. (2-3) to develop the Design Factor, DF, at each spectral frequency, at the ground surface.
(e) Modify the $UHRS_{H_D}$ at the surface with DF to obtain the DRS at the ground surface.

The number of convolution calculations performed must be sufficient to capture effects of the variability and uncertainty in soil properties on site response.

SECTION 2.4 CRITERIA FOR DEVELOPING SYNTHETIC OR MODIFIED RECORDED TIME HISTORIES

Ground motions that are generated to "match" or "envelop" given design response spectral shapes defined in Section 2.2 shall comply with steps (a) through (f) below. The general objective is to generate a modified recorded or synthetic accelerogram that achieves approximately a mean-based fit to the target spectrum; that is, the average ratio of the spectral acceleration calculated from the accelerogram to the target, where the ratio is calculated frequency by frequency, is only slightly greater than one. The aim is to achieve an accelerogram that does not have significant gaps in the Fourier amplitude spectrum, but which is not biased high with respect to the target. Records biased high with respect to a spectral target may overdrive (overestimate damping and stiffness reduction) a site soil column or structure when nonlinear effects are important.

(a) The time history shall have a sufficiently small time increment and sufficiently long duration. Records shall have a Nyquist frequency of at least 50 Hz (e.g., a time increment of at most 0.010 s) and a total duration of at least 20 s. If frequencies higher than 50 Hz are of interest, the time increment of the record must be suitably reduced to provide a Nyquist frequency ($N_y = 1/(2\ \Delta t)$, where Δt = time increment) above the maximum frequency of interest. The total duration of the record can be increased by zero packing to satisfy these frequency criteria.

(b) Spectral accelerations at 5% damping shall be computed at a minimum of 100 points per frequency decade, uniformly spaced over the log frequency scale from 0.1 Hz to 50 Hz or the Nyquist frequency. If the target response spectrum is defined in the frequency range from 0.2 Hz to 25 Hz, the comparison of the synthetic motion response spectrum with the target spectrum shall be made at each frequency computed in this frequency range.

(c) The computed 5% damped response spectrum of the accelerogram (if one synthetic motion is used for analysis) or of the average of all accelerograms (if a suite of motions is used for analysis) shall not fall more than 10% below the target spectrum at any one frequency. To prevent spectra in large frequency windows from falling below the target spectrum, the spectra within a frequency window of no larger than ±10% centered on the frequency shall be allowed to fall below the target spectrum. This corresponds to spectra at no more than nine adjacent frequency points defined in (b) above from falling below the target spectrum.

(d) In lieu of the power spectral density requirement of ASCE 4, the computed 5% damped response spectrum of the synthetic ground motion (if one synthetic motion is used for analysis) or the mean of the 5% damped response spectra (if a suite of motions is used for analysis) shall not exceed the target spectrum at any frequency by more than 30% (a factor of 1.3) in the frequency range between 0.2 Hz and 25 Hz. If the spectrum for the accelerogram exceeds the target spectrum by more than 30% at any frequency in this frequency range, the power spectral density of the accelerogram needs to be computed and shown to not have significant gaps in energy at any frequency over this frequency range.

(e) Because of the high variability in time domain characteristics of recorded earthquakes of similar magnitudes and at similar distances, strict time domain criteria are not recommended. However, synthetic motions defined as described above shall have strong motion durations (defined by the 5% to 75% Arias intensity), and ratios V/A and AD/V^2 (A, V, and D are the peak ground acceleration, ground velocity, and ground displacement, respectively), which are generally consistent with characteristic values for the magnitude and distance of the appropriate controlling events defined for the UHRS.

(f) To be considered statistically independent, the directional correlation coefficients between pairs of records shall not exceed a value of 0.30 (see Definitions in this Standard). Simply shifting the starting time of a given accelerogram does not constitute the establishment of a different accelerogram. If uncoupled response of the structure is expected, then only one time history is required. Then, the seismic analysis for each direction can be performed separately and then combined by the square root of the sum of the squares (SRSS).

Synthetic, recorded, or modified recorded earthquake ground motion time histories may be used for linear seismic analyses. Actual recorded earthquake ground motion or modified recorded ground motion shall be used for nonlinear seismic analyses. For nonlinear analyses, it is desirable to utilize actual recorded earthquake ground motion. However, to meet the requirements of steps (a) through (f) above, as many as 30 recorded earthquake motions would be required. As a result, it is acceptable to use modified recorded earthquake accelerograms that shall meet steps (a) through (f). A modified recorded accelerogram is a time-history record of acceleration versus time that has been produced from an actual recorded earthquake time history. However, the Fourier amplitudes are scaled such that the resulting response spectrum envelops the target response spectrum in the manner described above. The Fourier phasing from the recorded earthquake time history is preserved in a modified recorded earthquake accelerogram.

The selection of recorded or modified recorded accelerograms is based on the identification of dominant magnitude/distance pairs that impact the site DRS. The accelerograms used for nonlinear seismic calculations shall be selected from the appropriate magnitude/distance (M/D) bins. It may be necessary to produce different accelerograms that characterize the seismic hazard at appropriate low (about 1 Hz) and high (10 Hz) frequency. Alternatively, the accelerograms may be selected to match the dominant M/D pairs at the peak velocity and acceleration segments of the design spectrum. If behavior at the peak displacement frequency range is of interest, additional accelerograms may be selected whose controlling event is appropriate at these frequencies.

If recorded accelerograms are used directly as input to the nonlinear analyses, the suite of time histories shall meet the requirements of steps (a) through (f) above. If modified recorded time histories are generated to match the target spectrum, it is important to ensure that the phase spectra of the motions are generated from recorded motions in the appropriate *M/D* bins. In addition, the strong motion duration (as defined as the duration from the 5% to 75% Arias intensity) shall fall within the range appropriate for the *M/D* bin. In accepting the suite of motions, the range in variation in rise time of the Arias intensity shall be considered, such that all do not have the same rise time characteristics.

SECTION 3.0 EVALUATION OF SEISMIC DEMAND

SECTION 3.1 INTRODUCTION

Seismic demand shall be computed in accordance with the requirements of ASCE 4. Seismic demand shall be computed using linear equivalent static analysis, linear dynamic analysis, complex frequency response methods, or nonlinear analysis in accordance with the following sections and ASCE 4. Regardless of the procedure followed, it is important that

1. The input to the SSC be defined by either a DRS (Section 2.2) or a response spectrum compatible acceleration time history (Section 2.4).
2. The important natural frequencies of the SSC be estimated, or that the peak of the design spectrum, multiplied by an appropriate factor (Section 3.2.1),* be used as input. Soil–structure interaction and multimode effects shall be considered.
3. A load path evaluation for seismic induced inertial forces be performed. A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the foundation.
4. Seismic demand shall be obtained for the three orthogonal (two horizontal and one vertical) components of earthquake motion in accordance with ASCE 4. In general, the orthogonal axes shall be aligned with the principal axes of the structure.
5. All vertical load path elements shall be designed for the lateral displacements induced by seismic loads on the structure.

* From ASCE 4-98, Section 3.2.5.

SECTION 3.2 LINEAR ANALYSIS

3.2.1 Linear Equivalent-Static Analysis

An equivalent-static analysis may be used to evaluate single-point-of-attachment cantilever models with essentially uniform mass distribution, or other simple structures that can be idealized as a single-degree-of-freedom system. For cantilever models with essentially uniform mass distribution, the equivalent-static load base shear shall be determined by multiplying the cantilevered structure, equipment, or component masses by an acceleration equal to the peak of the input response spectrum. For these structures, the base moment shall be determined by using an acceleration equal to 1.1 times the peak of the applicable response spectrum. The resulting load shall be applied at the center of gravity of the structure.

For cantilevers with nonuniform mass distribution and other simple multiple–degree-of-freedom structures in which the predominant or fundamental mode shape of the structure has a curvature in one direction only (similar to a cantilever mode), the equivalent-static load shall be determined by multiplying the structure, equipment, or component masses by an acceleration equal to 1.5 times the peak acceleration of the applicable response spectrum. A smaller factor may be used, if justified.

Alternately, the spectral acceleration value at the fundamental frequency of the structure may be used if a modal solution has been obtained in accordance with ASCE 4. The use of the 1.1 or 1.5 factors defined above shall be applied to the spectral acceleration value determined at the fundamental frequency.

3.2.2 Linear Dynamic Analysis

Linear dynamic analysis may be used for any structure and may be performed using either response-spectrum or time-history approaches. Time-history approaches may use either direct integration or modal superposition methods in accordance with Section 3.2.2 of ASCE 4. P-Δ effects shall be included, if significant. If inclusion of P-Δ effects results in greater than a 10% increase in the imposed moment demand on a structural member, the effects shall be included; otherwise, they may be omitted.

SECTION 3.3 NONLINEAR ANALYSIS

Nonlinear seismic response analysis may need to be performed when significant nonlinear behavior is expected in some elements or when significant irregularities exist. This method requires definition of the load-deformation behavior of individual elements or the overall structural system. The nonlinear load-deformation curves used in analysis shall reflect behavior based on experimental data, which may be approximated by linear or curved segments. Nonlinear behavior shall be determined under monotonically increasing lateral deformation when nonlinear static analysis (pushover analysis) is performed. In the case of nonlinear dynamic analysis, appropriate load-deformation curves under multiple reversed deformation cycles shall be used.

3.3.1 Nonlinear Static Analysis

Structures whose response is dominated by a single mode may be evaluated using a nonlinear equivalent-static (pushover) analysis, provided that an effective frequency and damping are used to quantify the nonlinear response. Nonlinear equivalent-static methods of analysis shall follow the guidance provided in FEMA-356 for the target displacement method or in ATC-40 for the capacity spectrum method.

3.3.2 Nonlinear Dynamic Analysis

Nonlinear dynamic procedures shall follow the guidance provided in Section 3.2 of ASCE 4. Nonlinear dynamic analysis shall

- Have sufficient degrees of freedom to represent important responses of the structure. Single-degree-of-freedom models may be used for structures whose response is dominated by a single mode.
- Include P-Δ forces, if significant.
- Appropriately represent both the monotonic (backbone) and cyclic behavior of nonlinear elements. Members that exhibit pinched hysteretic behavior in laboratory tests shall be represented in the analysis with elements that represent similar pinching characteristics. Mean force-deflection properties shall be used.
- Approximate plastic hinge lengths for frame members by one beam depth, developed by rational analysis, or justified by comparison to test data.

When performing such nonlinear calculations, at least three different modified recorded accelerograms shall be used to determine potential nonlinear response. If less than five accelerograms are used, the largest response shall be used in making demand-to-capacity checks. If five or more accelerograms are used, the mean of the calculated responses may be used in making demand-to-capacity checks. If design spectrum matching is done separately for the low-frequency (about 1 Hz) and high-frequency (about 10 Hz) ranges, then at least three time histories are required for each frequency range.

SECTION 3.4 MODELING AND INPUT PARAMETERS

Modeling of SSCs for seismic analysis shall follow Section 3.1 of ASCE 4.

3.4.1 Effective Stiffness of Reinforced Concrete Members

In lieu of a detailed stiffness calculation, the effective stiffness of reinforced concrete members provided in Table 3-1 shall be used in linear elastic static or dynamic analysis. When finite element methods are used, the element stiffness shall be modified using the effective stiffness factor for the dominant response parameter.

3.4.2 Mass

The mathematical model used for determining seismic response shall include mass due to the following:

- Weight of the structure
- Weight of permanent equipment
- Expected live load, not less than 25% of the specified design live loads

Design snow loads of 30 psf or less need not be included. Where snow loads exceed 30 psf, the design snow load shall be included, but it may be reduced up to 75% where consideration of siting, configuration, and load duration warrant.

3.4.3 Damping Values for SSCs

Damping values to be used in linear elastic analyses for determining seismic design loads for SSCs are presented in Table 3-2 as a function of the average Response Level in the seismic load-resisting elements represented by the demand-to-capacity ratio (D_e/C). The D_e/C ratios are calculated on an element basis (C = code capacity, D_e = total elastic demand, including non-seismic loads). The appropriate Response Level can be estimated from Table 3-3.

Response Level 3 damping may be used for evaluating seismic-induced forces and moments in structural members by elastic analysis without consideration of the actual Response Level for Limit States A, B, or C. Response Level 2 damping may be used for Limit State D.

Consideration of the actual Response Level is required for generation of in-structure response spectra. In lieu of iterative analyses to determine the actual Response Level and associated damping value, Response Level 1 damping values may be used for generation of in-structure spectra. Response Level 1 damping values must be used if elastic buckling considerations control the design.

APPENDIX C

ASCE 4-98

American Society of Civil Engineers

Seismic Analysis of Safety-Related Nuclear Structures and Commentary

This document uses both Système International (SI) units and customary units.

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1801 Alexander Bell Drive
Reston, Virginia 20191-4400

N289.3-09

CANADIAN STANDARDS ASSOCIATION

Design procedures for seismic qualification of CANDU nuclear power plants

4.4 Time-histories of ground motion

4.4.1

Time-histories representing the design ground motion may be used for input in the dynamic analysis, provided they are compatible with the appropriate design ground response spectra.

4.4.2

A single set of three-component time-history accelerograms for design may be developed by taking an appropriate recording of a real or artificially synthesized ground motion and modifying its amplitudes and/or frequency content by mathematical suppression or raising techniques, such that the calculated spectrum for each time-history closely matches the design ground response spectrum as a minimum requirement. For examples see Tsai (1972) and Aziz and Biswas (1979). Other requirements for the time-history parameters are specified in Clause 4.4.4.

4.4.3

In lieu of the use of a single three-component time-history set, multiple artificial or real ground motion time-histories may be used. The calculated response spectrum for each time-history shall match a portion of the design ground response spectrum, such that the set of time-histories with appropriate combination of the results represents the effects of the broadband design ground response spectrum. Other time-history parameters are specified in Clause 4.4.4.

(2) *The selected duration should be consistent with the expected duration based on the dominant magnitude-distance contributions to hazard as determined from the investigations performed in accordance with CSA N289.2. As an exception, some near-field earthquake events may have a shorter duration.*

The time interval (Δt) shall be set to be a maximum of $\Delta t = 1/(2N_y)$, where N_y is Nyquist frequency in Hz, which represents the highest frequency of interest in the time-history analysis.

4.4.4.3 Damping value and frequency intervals

The 5% damped response spectrum shall be calculated from the time-histories at a minimum of 100 points at frequency intervals specified in Table 2.

4.4.4.4 Spectrum compatibility

No more than 6% of the total number of points used to generate the calculated response spectrum (CRS) from the time-history shall fall below the target response spectrum (TRS). No point on the CRS shall fall below the TRS by more than 10%.

Note: *The spectrum compatibility requirement may be applied to the average CRS from the 5 or more sets where at least 5 sets of records have been generated for*

(a) a broadband spectral match; or

(b) each of a series of sets of scenario events covering the entire frequency range of the TRS with appropriate combinations.

This permits spectral peaks and troughs of natural records to be preserved.

4.4.4.5 Power spectral density

The power spectral density (PSD) of each time-history shall be calculated and shown to not have any significant gaps in energy at any frequency over the frequency interval outlined in Table 2, as a minimum. The PSD computed from an accelerogram shall be defined in terms of Fourier amplitudes of the time-history, $F(\omega)$, by the relation

$$PSD(\omega) = 2|F(\omega)|^2 / (2\pi T_{sm})$$

where

T_{sm} = strong motion duration

4.4.4.6 Statistical independence

Simulated earthquake motion time-histories that are generated to be compatible with the TRS for the three directions (two horizontal and one vertical) shall be statistically independent. Two time-histories are considered statistically independent if the absolute value of the correlation coefficient does not exceed 0.3.

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Earthquake Analyses and Design of Dams

May 2005

FEMA

duration for the particular design earthquake. In addition, whenever possible, the acceleration time histories should be representative of the design or safety evaluation earthquake in all the following aspects: earthquake magnitude, distance from source-to-site, fault rupture mechanisms (fault type, focal depth), transmission path properties, and regional and geological conditions. Since it is not always possible to find empirical records that satisfy all of the above criteria, it is often necessary to modify existing records or develop synthetic records that meet most of these requirements.

2. Approaches to Developing Time Histories. There are two general approaches to developing acceleration time histories: selecting a suite of recorded motions and synthetically developing or modifying one or more motions. These approaches are discussed below. For either approach, when modeling near-source earthquake ground motions (i.e., minimum source-site distance less than 10 km), it is desirable that the motions include a strong intermediate- to long-period pulse to model this particular characteristic of ground motion often observed in the near field and generally accepted to be responsible for significant damage. Of specific importance at distances less than 10 km are the effects of directivity in developing fault normal and fault parallel components (Somerville et al 1997).

a. Selecting Recorded Motions

(1) Typically, in selecting recorded motions, it is necessary to select a suite of time histories (typically 3 or more) such that, in aggregate, valleys of individual spectra that fall below the design (or "target") response spectrum are compensated by peaks of other spectra and the exceedance of the design response spectrum by individual spectral peaks is not excessive (preferably at least within the bandwidth of interest for structures specific analysis). For nonlinear analyses, it is desirable to have additional time histories because of the importance of phasing (pulse sequencing) to nonlinear response. In the past, when using selected recorded motions, simple scaling of acceleration time histories was frequently performed to enhance spectral fit. However, scaling should be done with caution. The ramifications of significant scaling of acceleration time-histories on velocity, displacement, and energy can be profound.

(2) The advantage of selecting recorded motions is that each accelerogram is an actual recording; thus, the structure is analyzed for motions that are presumably most representative of what the structure could experience. The disadvantages are: multiple dynamic analyses are needed for the suite of accelerograms selected; although a suite of accelerograms is selected, there will typically be some exceedances of the smooth design spectrum by individual spectrum peaks; and although a reasonably good spectral fit may be achieved for one horizontal component, when the same simple scaling factors are applied to the other horizontal components and the vertical components for the records selected, the spectral fit is usually not as good for the other components.

b. Synthetically Developing or Modifying Motions

(1) Techniques. A number of techniques and computer programs have been developed to either completely synthesize an accelerogram or modify a recorded accelerogram so that the response spectrum of the resultant waveform closely matches the design or target spectrum. Recent advances have used either (a) frequency-domain techniques with an amplitude spectrum based upon band-limited white noise and a simple, idealized source spectrum combined with the phase spectra of an existing record; or (b) kinematic models that produce three components of motion using complex source and propagation characteristics. Such motions have the character of recorded motions since the modeling procedures are intended to simulate the earthquake rupture and wave propagation process. Recent research suggests dynamic and three-dimensional models may be important in estimating engineering ground motions in the future.

(2) Comments. The natural appearance and duration of strong motion can be maintained using these techniques. A good fit to the target spectrum may or may not be possible with a single component of motion. However, for non-linear applications, it is particularly desirable to have multiple accelerograms because different accelerograms may have different phasing (pulse sequencing) characteristics of importance to nonlinear response yet have essentially identical response spectra. For near-field situations, the characteristics of the motions should reproduce the coherent velocity pulses ("fling") commonly observed in near-field recordings.

(3) Advantages and Disadvantages. The advantages of synthetic techniques for developing time-histories are: the natural appearance and strong motion duration can be maintained in the accelerograms; three component motions (two horizontal and one vertical) each providing a good spectral match can be developed; and the process is relatively efficient. The disadvantage is that the motions are not "real" motions. Real motions generally do not exhibit smooth spectra. Although a good fit to a design spectrum can be attained with a single accelerogram, it may be desirable to fit the spectrum using more than one accelerogram. Such motions have the character of recorded motions since the modeling procedures are intended to simulate the earthquake rupture and wave propagation process.

3. Application. Ground motion parameters should be specified in a manner that is consistent with the analyses to be performed. Where ground motions are specified at one location (e.g., a rock outcrop) and are used in the analysis at a different location (e.g., at the base of a soil layer), the motions need to be adjusted accordingly. Where magnitude and distance are used in empirical procedures, it is important to verify that distance-attenuation definitions in the procedure are consistent with those inferred for the site of interest.

Conditional Mean Spectrum – account for inter-period correlations in record selection to "match" UHS. Defn:

$$\mu_{\ln S_a(T_i)|\ln S_a(T_n)} = \mu_{\ln S_a}(\overline{\mathbf{M}}, \overline{R}, T_i) + \rho(T_i, T_n)\overline{\varepsilon}(T_n)\sigma_{\ln S_a}(T_i)$$

Baker [14] proposed a CMS-based record selection procedure for the seismic performance evaluation of a structure. The procedure begins by specifying a target seismic intensity level, in terms of $S_a(T_n)$, and representative scenario(s), in terms of $\overline{\mathbf{M}}$, $\overline{R}$, and $\overline{\varepsilon}$ (see Figure 3 and Table I). By adopting an adequate GMPE for the considered scenario, one can evaluate the mean and standard deviation of natural logarithm of the spectral acceleration at the vibration period T_i, denoted by $\mu_{\ln Sa}(\overline{\mathbf{M}}, \overline{R}, T_i)$ and $\sigma_{\ln Sa}(T_i)$. Then, the CMS, in natural logarithmic space, is given by:

$$\mu_{\ln S_a(T_i)|\ln S_a(T_n)} = \mu_{\ln S_a}(\overline{\mathbf{M}}, \overline{R}, T_i) + \rho(T_i, T_n)\overline{\varepsilon}(T_n)\sigma_{\ln S_a}(T_i), \quad (1)$$

ε is the number of σ from median

where $\rho(T_i, T_n)$ is the inter-period correlation of spectral accelerations at vibration periods T_i and T_n. Baker and Cornell [24] carried out empirical analysis of the inter-period correlation using California records, and proposed the following prediction equation:

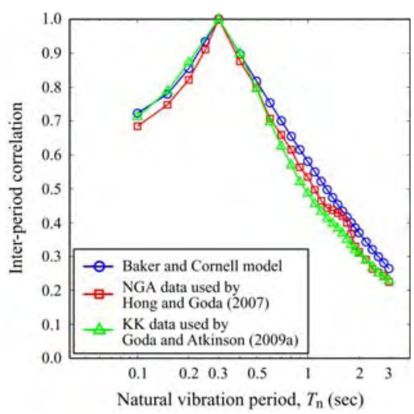
$$\rho(T_{n1}, T_{n2}) = 1 - \cos\left(\pi/2 - [0.359 + 0.163 I_{T_{min}<0.189} \ln(T_{min}/0.189)] \ln(T_{max}/T_{min})\right), \quad (2)$$

where T_{max} and T_{min} are the larger and the smaller of T_{n1} and T_{n2}, respectively, and $I_{T_{min}<0.189}$ is the indicator function that equals one if T_{min} is less than 0.189 sec and equals zero otherwise. We note that

Conditional Mean Spectrum

$$\mu_{\ln S_a(T_i)|\ln S_a(T_n)} = \mu_{\ln S_a}(\overline{\mathbf{M}}, \overline{R}, T_i) + \rho(T_i, T_n)\overline{\varepsilon}(T_n)\sigma_{\ln S_a}(T_i)$$

CMS = UHS + correlationCoef *residuals



- Conditional Mean Spectrum takes inter-period correlation of spectral accelerations at different vibration periods.
- Useful when the target response spectrum is defined in tandem with UHS (because UHS ordinates at different vibration periods do not represent spectral characteristics of a single record)

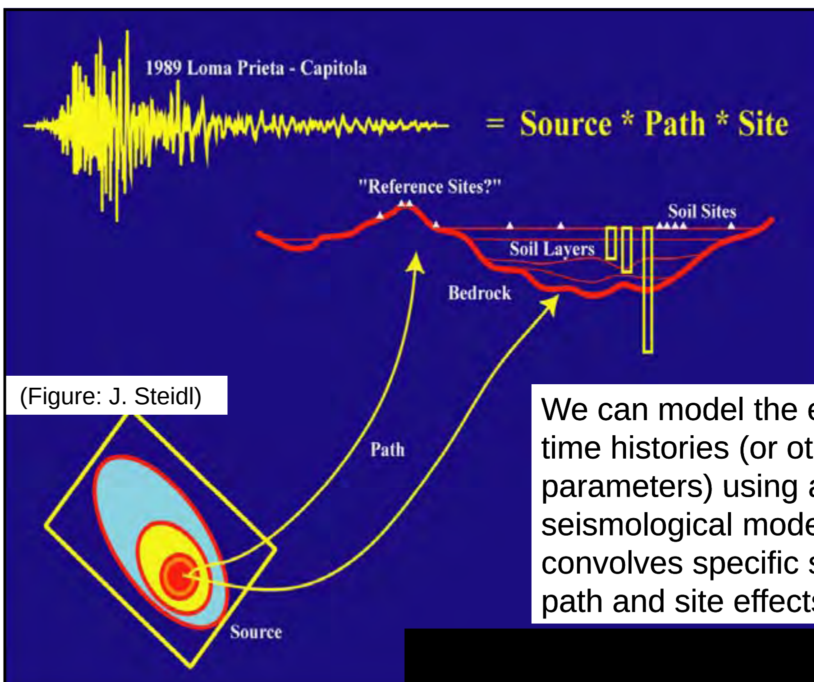
Simulated time histories compatible with 2005 (or 2010) NBCC UHS

Gail M. Atkinson

CSRN meeting

April 2010

(paper published in CJCE, 2009)



(Figure: J. Steidl)

We can model the expected time histories (or other parameters) using a seismological model that convolves specific source, path and site effects

[redacted]

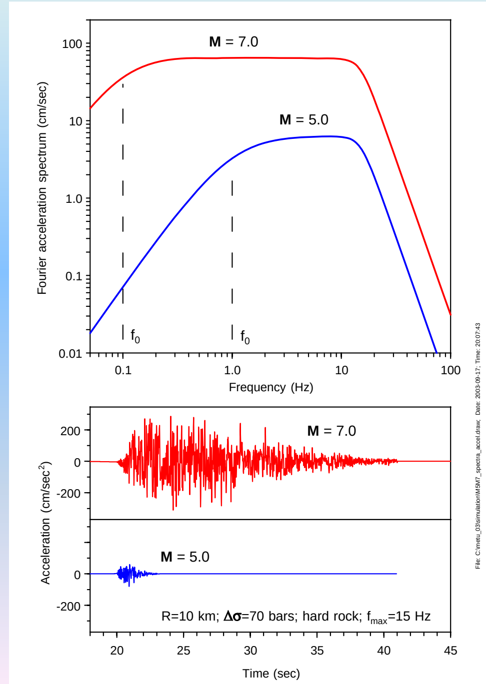
Stochastic method

•Assume we have a target spectrum (such as top graph) that describes the event. The spectrum is given by seismological models

•Radiated energy for the target spectra is assumed to be distributed randomly over a duration that depends on magnitude and distance

Advantages:
•Complex physics is encapsulated into simple functional forms
•Empirical findings can be easily incorporated

Example generated with SMSIM (Boore)



Steps in simulating time series for a simple point source

• Generate Gaussian or uniformly distributed random white noise

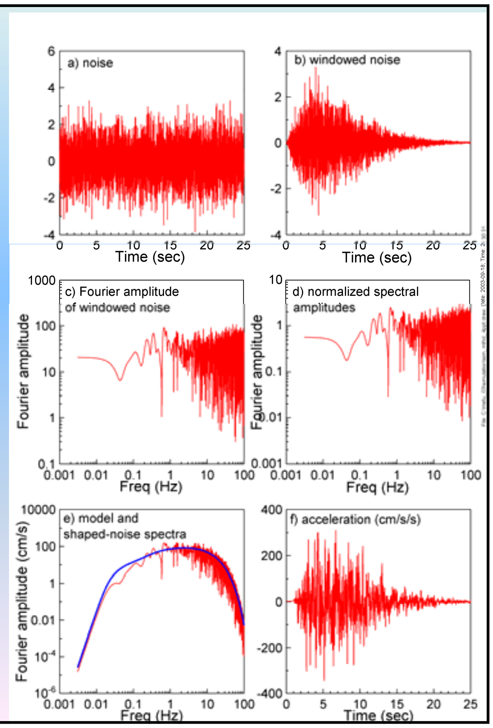
• Apply a shaping window in the time domain

• Compute Fourier transform of the windowed time series

• Normalize so that the average squared amplitude is unity

• Multiply by the spectral amplitude and shape of the ground motion

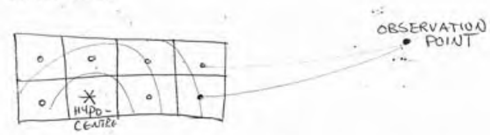
• Transform back to the time domain



Extension of stochastic model to finite faults (Silva; Beresnev and Atkinson; Motazedian and Atkinson)

STOCHASTIC FINITE-FAULT MODEL
(Beresnev and Atkinson, 1997, 1998)

TREAT FINITE FAULT PLANE AS AN ARRAY OF SUBFAULTS



MODEL EACH SUBFAULT AS A STOCHASTIC POINT SOURCE, WITH A BRUNE (ω^2) SOURCE SPECTRUM

RUPTURE STARTS AT A SPECIFIED SUBFAULT (HYPOCENTRE), AND PROPAGATES IN ALL DIRECTIONS WITH SPECIFIED RUPTURE PROPAGATION VELOCITY (SAY 0.8 TIMES SHEAR WAVE VELOCITY).

SUBFAULT RADIATION IS 'TRIGGERED' WHEN RUPTURE REACHES THE CENTRE OF THE SUBFAULT

CONTRIBUTIONS TO RADIATION AT OBSERVATION POINT ARE SUMMED OVER ALL SUBFAULTS.

Basics of Atkinson (2009 CJCE) simulations

- Realistic records for the typical magnitudes/distances that contribute to 2005 NBCC UHS for Canadian cities, for several generic site conditions (A, C, D, E)
- Simple finite-fault stochastic model encapsulates basic seismological parameters for east, west Canada
- User picks records from time history library and scales/matches as per study needs

What is generated: East

East: For each site condition (A, C, D, E)

- M6 Set 1: 3 random components at 15 random locations about 10 to 15 km from fault (=45 records)
- M6 Set 2: 45 records about 20 to 30 km from fault
- M7 Set 1: 45 records about 15 to 25 km from fault
- M7 Set 2: 45 records about 50 to 100 km from fault

Download from www.seismotoolbox.ca

What is generated: West

West: For each site condition (A, C, D, E)

For Crustal/Inslab Events:

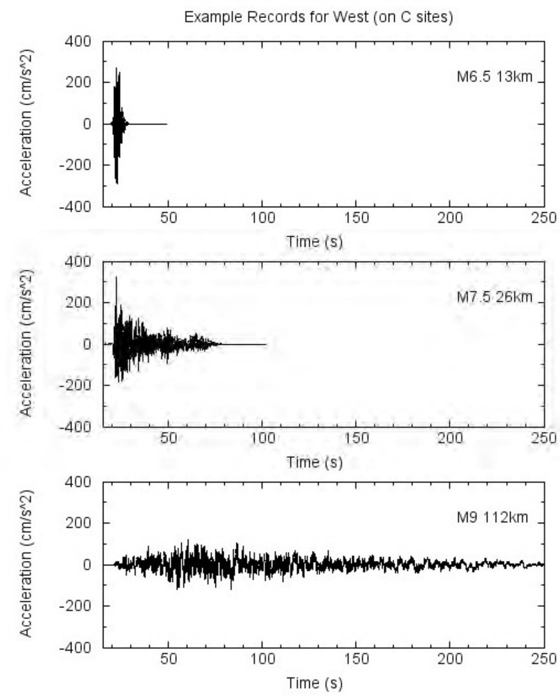
- M6.5 Set 1: 3 random components at 15 random locations about 10 to 15 km from fault (=45 records)
- M6.5 Set 2: 45 records about 20 to 30 km from fault
- M7.5 Set 1: 45 records about 15 to 25 km from fault
- M7.5 Set 2: 45 records about 50 to 100 km from fault

For Interface Events:

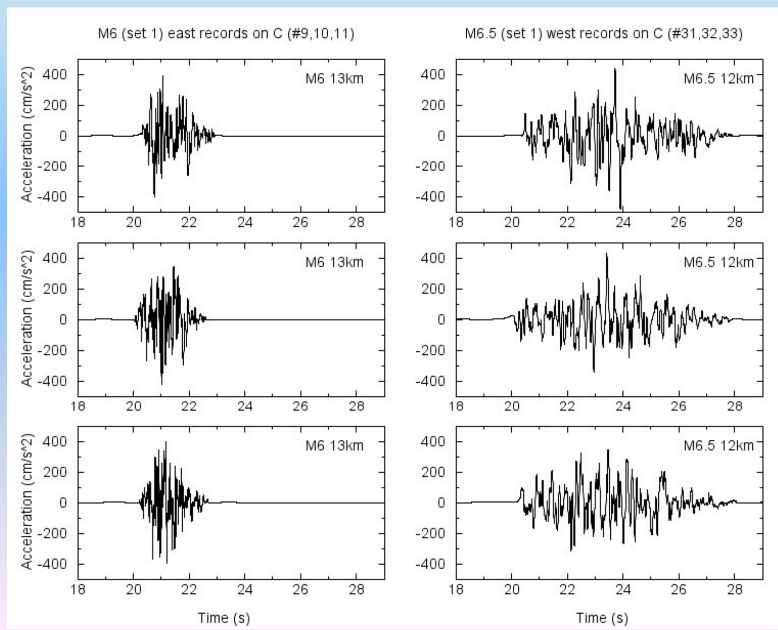
- M9 Scenario (Atkinson and Macias, 2009 BSSA for details): 45 records at distances 100 to 200 km from fault (eg. Victoria is at about 100 km)

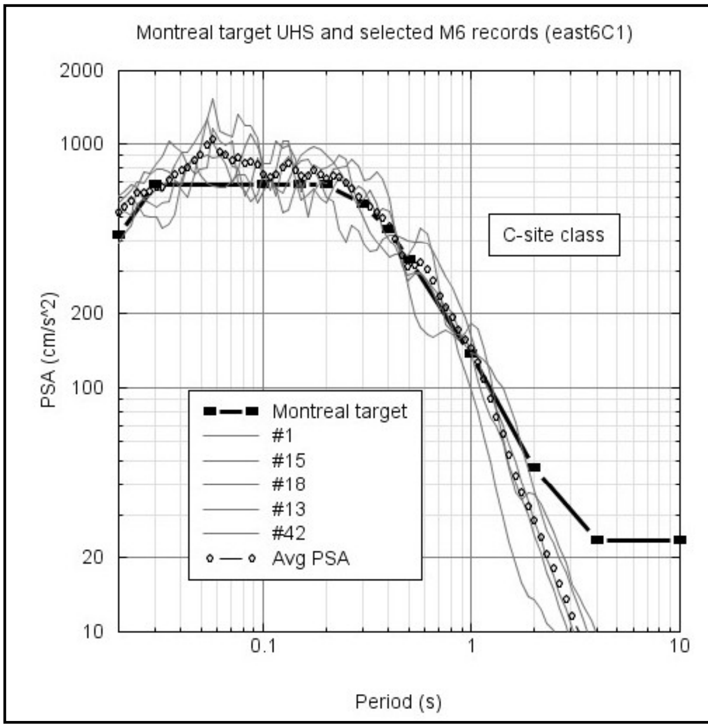
Download from www.seismotoolbox.ca

Example western records: note low PGA but long duration for M9 Cascadia

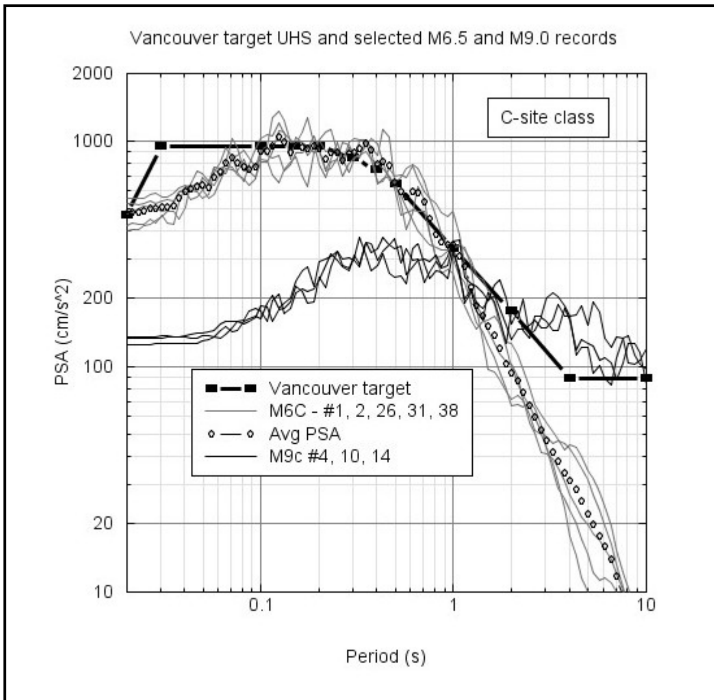


Example of east vs. west records (higher frequency content in east)





Example of 5 records of M6 selected/scaled to approximately match Montreal UHS on C-class site, 0.1-1 sec



Example of 5 M6.5 records selected/scaled to match Vancouver UHS at 0.1-1 sec

+ 3 M9 records (unscaled) selected to approximately match UHS at 1 to 2 sec.

1

COMMENTS ON USING SIMULATED RECORDS FOR NL ANALYSIS

P. Léger (R. Tremblay) – École Polytechnique de Montréal

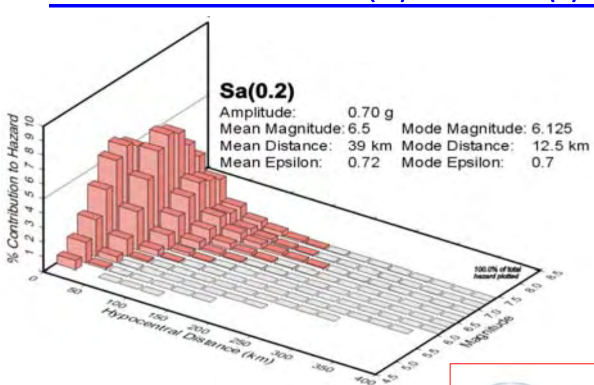
GOAL – DEVELOP GUIDELINES TO USE SIMULATED RECORDS FOR 3D NL ANALYSIS OF BUILDING STRUCTURES COMMENTARY TO NBCC / GUIDELINES

OBJECTIVES – Present typical results, identify some needs

SOURCES OF INFORMATION

1. SIMQKE – FILTERED WITH NOISE
2. SPECIFIC BARRIER MODEL (SOFTWARE SUNY BUFFALO)
3. G. ATKINSON SEISMO-TECTONIC MODELS (GA WEB SITE)

DEAGGREGATION OF SEISMIC HAZARD MAGNITUDE (M) – DISTANCE (R) SCENARIOS



Montreal
$S_a(0.2)$ 2%-50 yrs
Return period = 2500 yrs

13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2470

DEAGGREGATION OF SEISMIC HAZARD FOR SELECTED CANADIAN CITIES

Stephen HALCHUK [1] and John ADAMS [1]

2

SIMULATED GROUND MOTIONS

Stochastic approach with a physical representation of the source (Specific Barrier Model)– MCEER Buffalo NY, USA

Computer program used for M-R scenarios (on rock)

SGMSv5 ("Strong Ground Motion Simulation")

→Generation of two independent horizontal components

RSCTH ("Response spectrum Compatible Time Histories")

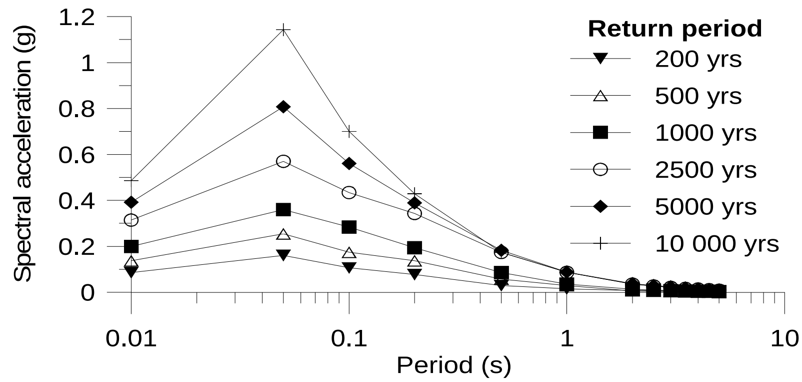
→ Generation of the vertical component

SPECTR (Frequency domain iterative modifications to obtain spectra compatibility – EPM)

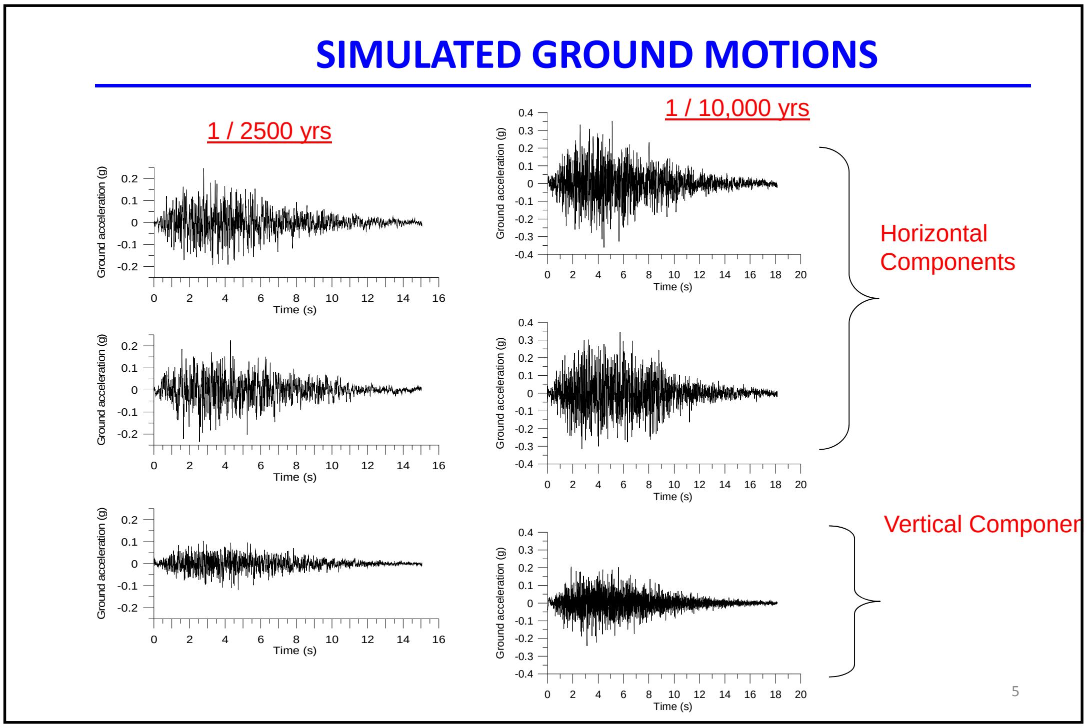
3

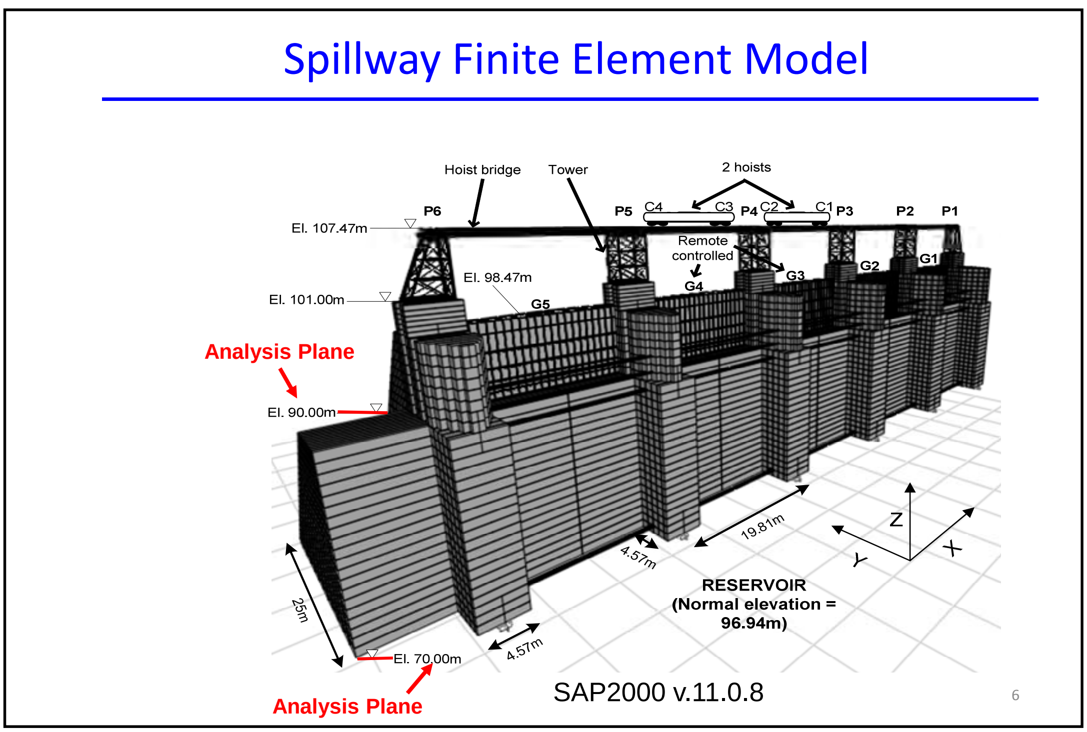
HORIZONTAL DESIGN SPECTRA

ADAPTED FROM ONTARIO POWER GENERATION (OPG)



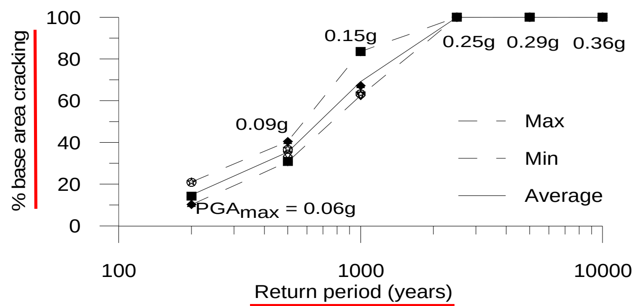
4


SIMULATED GROUND MOTIONS
1 / 2500 yrs
1 / 10,000 yrs
Ground acceleration (g)
Time (s)
Horizontal Components
Vertical Componen
5


Spillway Finite Element Model
Hoist bridge
Tower
2 hoists
P6
P5
C4
C3
P4
C2
C1
P3
P2
P1
El. 107.47m
Remote controlled
G4
G3
G2
G1
El. 98.47m
G5
El. 101.00m
Analysis Plane
El. 90.00m
19.81m
4.57m
Z
Y
X
RESERVOIR
(Normal elevation = 96.94m)
25m
El. 70.00m
4.57m
Analysis Plane
SAP2000 v.11.0.8
6

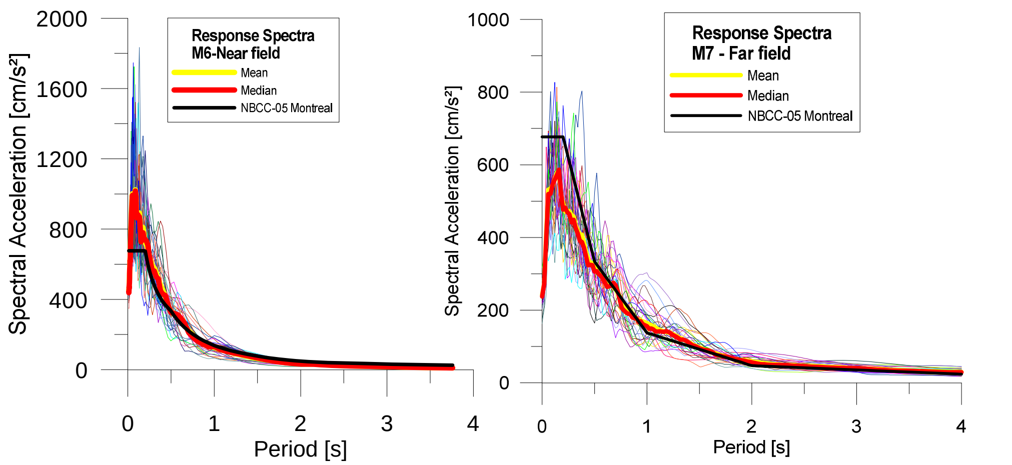
INCREMENTAL DYNAMIC ANALYSIS (IDA)

- Results = Evolution of a performance parameter as a function of an earthquake intensity parameter
- Subject the structure to a series of accelerograms of increasing intensity (same record or different records) – Damage Response –



7

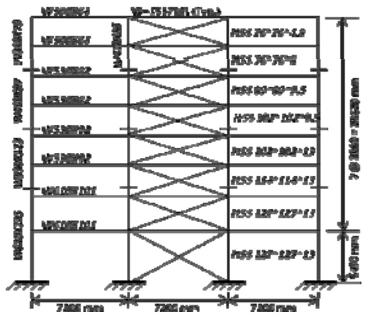
G. ATKINSON – 2010 WEB SITE 2010
30 ACCELEROGRAMS IN EACH BIN (M6 vs M7) Montreal

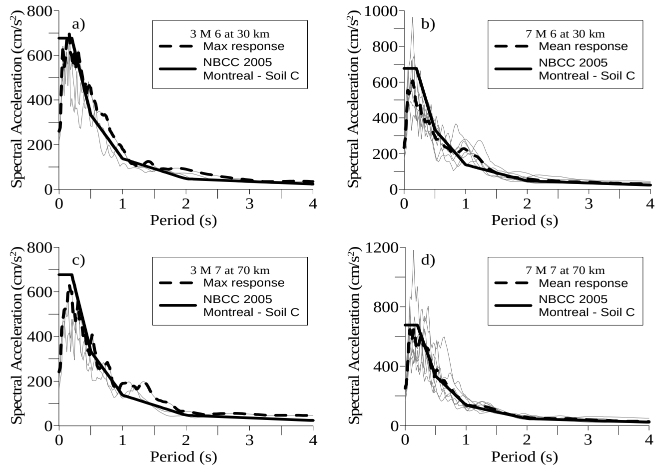


8

NBCC 05 - NUMBER OF GROUND MOTION RECORDS

INPUT MOTIONS FROM GA - WEB DATABASE

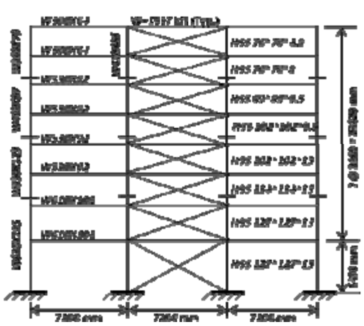


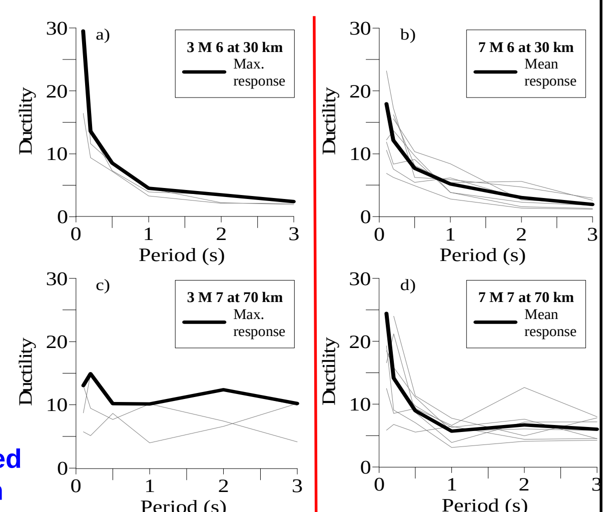


8 storey Bldg – Montreal
3 M6 – 30 km ; 3 M7 - 70 km
7 M6 – 30 km; 7 M7 – 70 km

9

NBCC 05 - NUMBER OF GROUND MOTION RECORDS; μ demand





8 storey Bldg – Montreal
Max. response 3 records or
Mean response of 7 records

Envelope ductility demand computed from 3 accelerograms >> the mean ductility value computed from 7 accelerogram sets.

10

USE OF SYNTHETIC RECORDS - SOME NEEDS

1D ANALYSES

•Guidelines for selecting period range for scalar scaling to target spectra (NL behaviour = period elongation)
• Different damping ratios (G. Atkinson, J.R. Pierre)
• Required number of signals (current guidelines 3 env., 7 max.)

2D – 3D ANALYSES
• Statistical independence of each realisation
• Cross correlation coefficients
• Principal directions ….
• H1 vs H2 spectral intensities (use of 0.8 or not)
• Vertical records

11

CONCLUSIONS

- Ideally use a series of historical (simulated) ground motion records – perform statistical (Dispersion; input vs output) analyses (median, mean, 95% confidence level)
- Not practical in several cases – use reduced sets of 3D spectrum compatible earthquake records (FD or TD modifications) - …. To COME … FOR DISCUSSION….
- **GUIDELINES**

12

Selection and scaling of NBCC 2005 compatible simulated ground motions for nonlinear seismic analysis of building structures

Sanda Koboevic, Kim Guilini-Charrette, Pierre Castongay and Robert Tremblay

(submitted to Canadian Journal of Civil Engineering)

Group for Research in STRUCTURAL ENGINEERING

Department of Civil, Geological and Mining Engineering

ÉCOLE POLYTECHNIQUE MONTRÉAL

OBJECTIVES:

- **Evaluate different approaches for selection and scaling of ground motions for use in nonlinear time-history analysis (NBCC 2005 UHS)**
- **Examine the influence of site conditions on ground motion amplification**

Historical versus simulated records ?

Impact on nonlinear structural response ?

Vancouver Class C site

- **Simulated records: Atkinson (2008)**

8 azimuths, two magnitudes, 9 distances from the center of the fault (5, 10, 15, 20, 25, 30, 40, 50 and 100 km)
432 simulated records

In Atkinson (2009) 180 records

Initial selection (dominant M-R scenarios):

M 6.5 R 10 and 20 km (3 trials x 8 azimuts)
M 7.5 R 20, 30 and 50 km (3 trials x 8 azimuths)

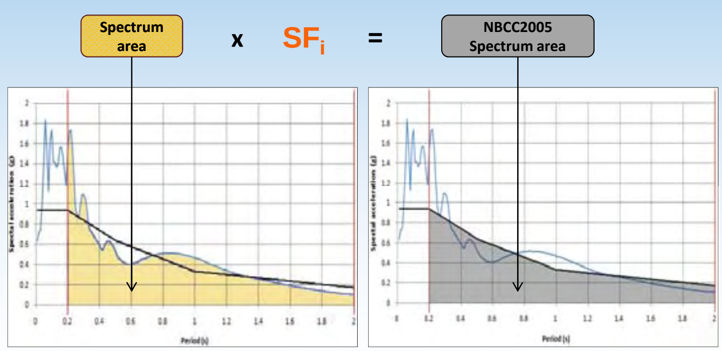
Total:120 simulated records

Atkinson, G. February 2008. Private Communication

*Atkinson, G. M. 2009. Earthquake time histories compatible with the 2005 National Building Code of Canada uniform hazard spectrum. Canadian Journal of Civil Engineering, **36** (991–1000).*

Two scaling methods (based on spectral intensity):

- **Method IND** (scaling of each individual record; 0.2s to 2s)



1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

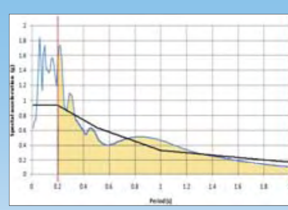
- **Method ATC (*similar to ATC-63 procedure*)**

$$SF_i = SF1_i \times SF2$$

$$SF1_i = \frac{\text{Median record PGV}}{PGV_i}$$

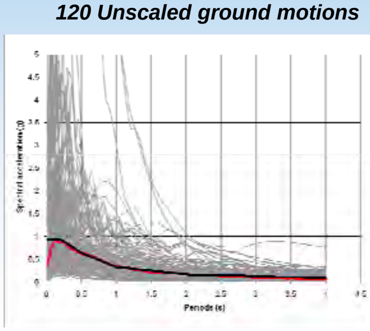
$$\text{Spectrum Area}\big|_{0.2}^{2.0} \times SF2 - \text{NBCC05 spectrum Area}\big|_{0.2}^{2.0}$$

1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

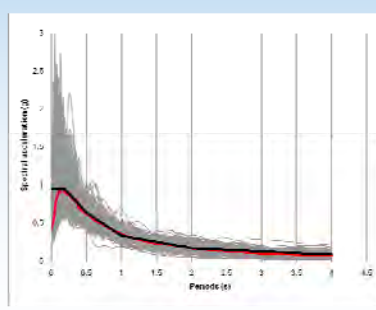


Equal Area (IND)

120 Unscaled ground motions



120 Scaled ground motions



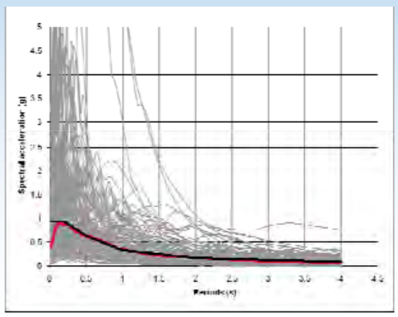
1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

$SF1_i$ = Median PGV / PGV_i

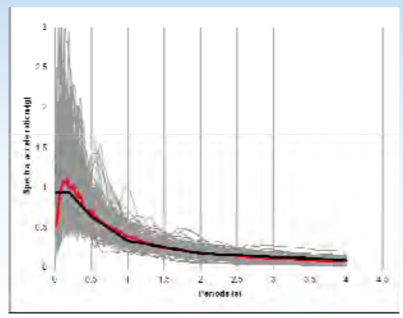
SF2 = Median Spectrum = NBCC Spectrum

(ATC-63)

120 Unscaled ground motions



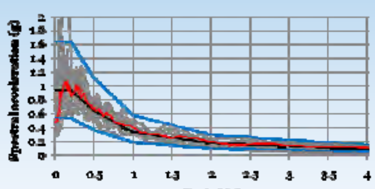
120 Scaled ground motions



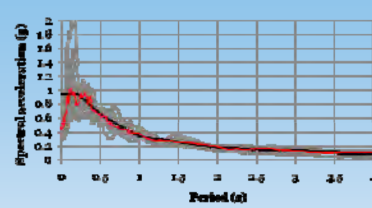
1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

FIT

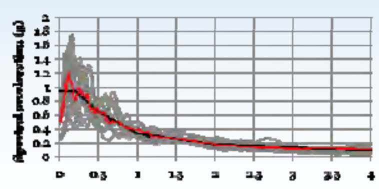
10 / 120 Unscaled ground motions that best fit the NBCC sectrum



+ Equal Area Scaling



+ ACT-63 Scaling



1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

- **Reduce ground motion database (10 records from 120):**

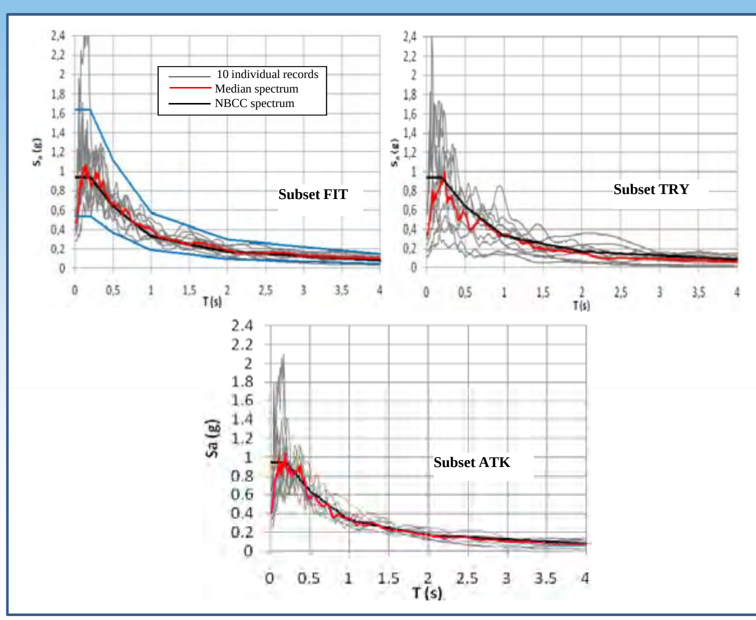
<u>Subset **FIT**</u>: best "natural" fit between the spectrum of a candidate record and the target UHS NBCC 2005

<u>Subset **TRY**</u>: two acceleration records from each M-R scenario (closest IND scaling factor to mean IND scaling factor obtained for 24 records from one M-R scenario)

<u>Subset **ATK**</u>: procedure described in Atkinson (2009)

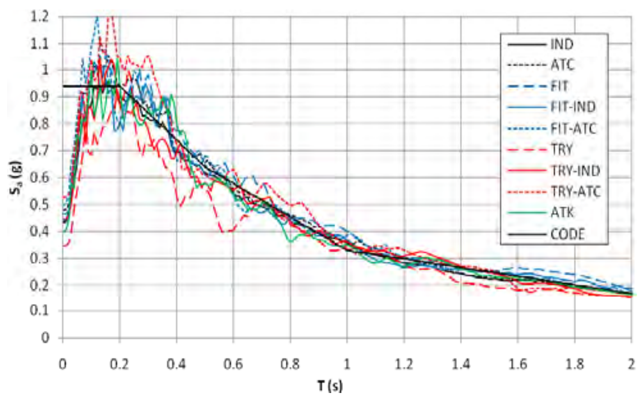
•SA_{targ} NBCC 2005/SA_{sim}, at every characteristic T (0.2 s to 2.0 s)
•determine the mean and standard deviation
•select records with minimum STD
•scale with mean (SA_{targ}/SA_{sim})

1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.



1.SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

Vancouver – Class C site

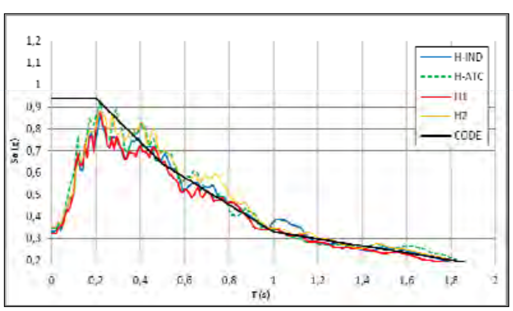


Comparison between the median spectra of the simulated ground motion sets and the target NBCC (CODE)

1.SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

- **Historical records: PEER database (10 records)**

ID	Event	M	R (km)	Station	Comp. (º)	PGA (g)	PGV (m/s)
V11	Jan. 17, 1994 Northridge	6.7	44	Castaic, Old Ridge Rd	90	0.568	0.53
V12	Jan. 17, 1994 Northridge	6.7	30	Santa Monica City Hall	360	0.369	0.251
V13	Jan. 17, 1994 Northridge	6.7	34	Los Angeles Baldwin Hills	360	0.167	0.176
V14	Fev. 9, 1971 San Fernando	6.6	31	Castaic, Old Ridge Rd	291	0.268	0.259
V15	Jan. 17, 1994 Northridge	6.7	26	Pacific Palisades-Sunset	280	0.197	0.149
V16	Avr. 25, 1992 Cape Mendocino	7.0	52	Eureka - Myrtle & West	90	0.178	0.283
V17	Oct. 18, 1989 Loma Prieta	7.0	54	Stanford Univ.	360	0.29	0.28
V18	Oct. 18, 1989 Loma Prieta	7.0	100	Presidio	90	0.200	0.34
V19	Avr. 13, 1949 West.Wash.	7.1	76	Olympia, Test Lab	86	0.28	0.17
V20	Juin 28, 1992 Landers	7.3	93	Barstow	90	0.135	0.258



H1, H2 alternative scaling procedures based on the compatibility of spectral intensity

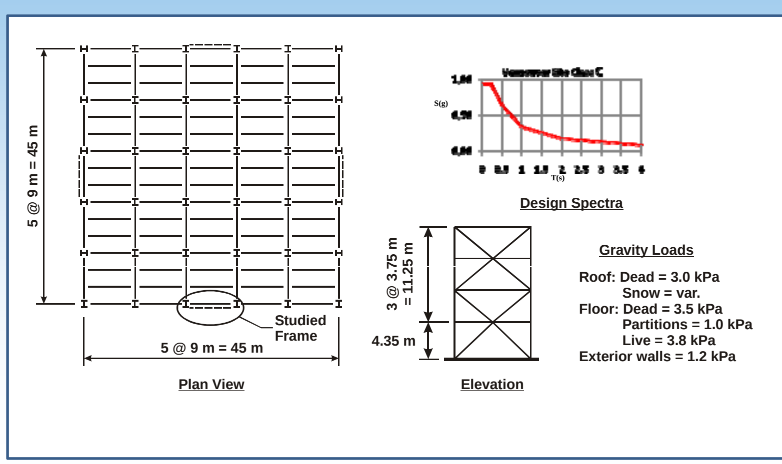
1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

Comparaison of ground motion parameters:

	Simulated records							Historical records			
	FIT	FIT-IND	FIT-ATC	TRY	TRY-IND	TRY-ATC	ATK	H1	H2	H-IND	H-ATC
PGA (g)	0.47	0.43	0.44	0.60	1.01	1.22	0.44	0.32	0.35	0.34	0.35
PGV (m/s)	0.45	0.41	0.41	0.45	0.63	0.74	0.40	0.33	0.36	0.36	0.34
PGA/PGV	1.1	1.1	1.1	1.3	1.3	1.3	1.2	1.0	1.0	1.0	1.0
t_d (s)	16	16	16	16	16	16	17	14	14	14	14
I_A (m/s)	2.81	2.26	2.55	3.32	10.03	15.17	2.22	1.34	1.69	1.50	1.73
NZC	226	226	226	157	157	157	202	82	82	82	82
v_{incr} (m/s)	0.21	0.19	0.20	0.28	0.44	0.54	0.21	0.23	0.25	0.25	0.23

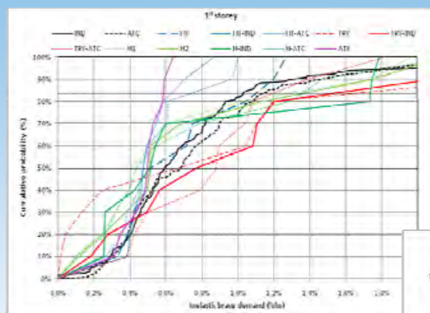
1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

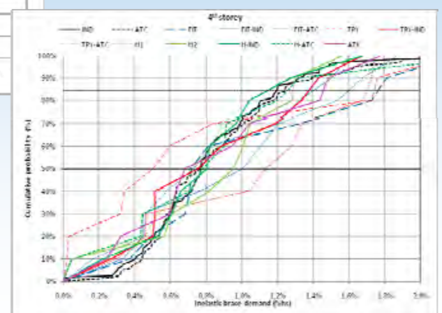
Comparaison of inelastic structural response:



1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

Comparaison of inelastic structural response:





β_{RTR} (record-to-record variability factor)

> 0.4 (ATC-63)

particularly for TRY, H2, and H-ATC

inelastic deformation response: more sensitive to the characteristics of a particular acceleration record

1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

Table 4. Median and 84th percentile values of the normalized brace inelastic deformations (% h_s)

		Simulated records									Historical records			
		IND	ATC	FIT	FIT-IND	FIT-ATC	TRY	TRY-IND	TRY-ATC	ATK	H1	H2	H-IND	H-ATC
4th storey	50th	0.77	0.77	0.76	1.06	0.78	0.55	0.82	1.20	0.81	0.78	0.98	0.81	0.84
	84th	1.19	1.24	1.78	1.43	1.62	1.75	1.39	1.71	1.46	1.18	1.29	1.17	1.27
	β_{RTR}	0.46	0.45	0.60	0.56	0.75	1.62	0.59	0.64	0.66	0.97	1.02	0.56	1.04
3rd storey	50th	0.64	0.58	0.77	0.79	0.51	0.53	0.75	0.96	0.47	0.51	0.70	0.55	0.72
	84th	1.03	1.09	1.36	0.96	1.16	1.63	1.15	1.29	1.01	0.98	1.06	0.95	1.25
	β_{RTR}	0.56	0.53	0.52	0.41	0.67	0.90	0.68	0.54	0.53	0.75	0.76	0.60	0.79
2nd storey	50th	0.51	0.49	0.43	0.41	0.48	0.42	0.49	0.64	0.40	0.52	0.64	0.56	0.64
	84th	0.81	0.82	0.79	0.84	0.71	0.81	0.94	0.75	0.67	1.06	1.06	1.07	1.01
	β_{RTR}	0.56	0.52	0.46	0.45	0.52	2.57	0.57	0.45	0.44	0.80	0.88	0.48	0.79
1st storey	50th	0.60	0.69	0.61	0.48	0.49	0.85	0.93	0.89	0.50	0.47	0.51	0.53	0.49
	84th	1.03	1.16	1.13	0.65	0.81	1.90	1.69	1.26	0.59	1.42	1.47	1.74	1.39
	β_{RTR}	0.59	0.62	0.49	0.26	0.42	1.79	0.83	0.51	0.23	0.88	0.91	0.80	0.90

1. SELECTION AND SCALING OF GROUND MOTION RECORDS (Western Canada) cont.

CONCLUSIONS:

- **Historical records and large ensemble of simulated records induced similar inelastic structural response in spite of differences in ground motion characteristics;**
- **Reduction of the number of simulated records does not significantly impact the response provided that the records are adequately selected and scaled;**
- **Best concordance with historical records obtained for simulated records with response spectra that fit the best NBCC UHS without scaling (0.2s to 2 s);**
- **Records selected and scaled using method described in Atkinson (2009) induce generally similar structural response as historical records (some underestimation of brace deformations);**
- **Confidence to use simulated records when historical records are rare or unavailable.**

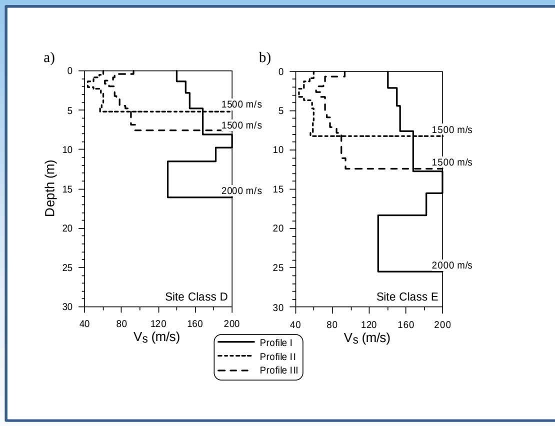
2. GROUND MOTION AMPLIFICATION DUE TO SITE EFFECTS (Eastern Canada)

Montreal Class D and E sites

- **Dynamic soil response analysis (ProShake) for 3 realistic soil profiles for Class D and Class E sites;**
- **Compare spectra of surface ground motions obtained by Proshake (simulated versus historical);**
- **Compare spectra of surface ground motions obtained by Proshake to spectra of simulated ground motions generated for D and E class sites directly;**
- **Compare ground motion characteristics;**
- **Compare induced inelastic structural response.**

2. GROUND MOTION AMPLIFICATION DUE TO SITE EFFECTS (Eastern Canada) cont.

Soil profiles studied

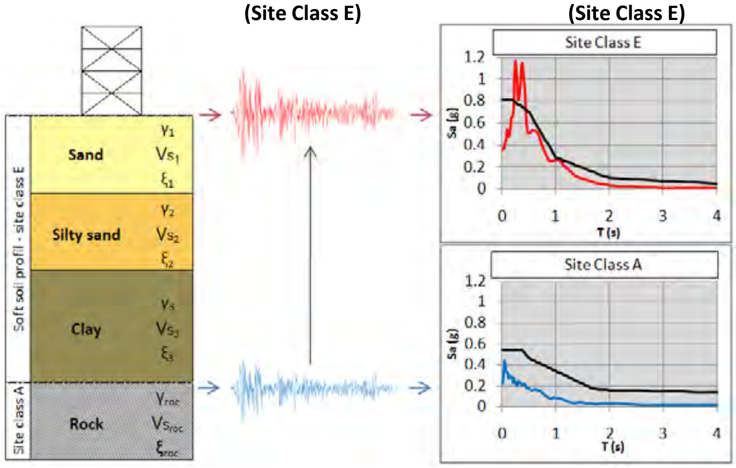


Shear wave velocity profiles I to III assumed in Montreal for:
a) Class D site; b) Class E site.

2. GROUND MOTION AMPLIFICATION DUE TO SITE EFFECTS (Eastern Canada) cont.

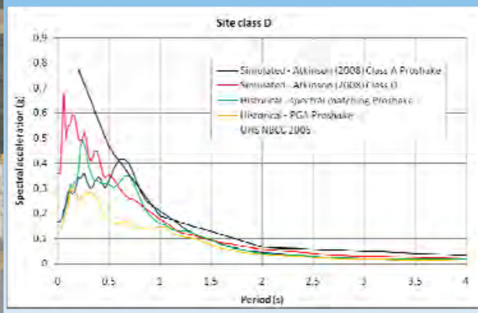
ProShake analysis

Structure | **Final ground motion (Site Class E)** → **Corresponding spectrum (Site Class E)**



Soil Profile | **Initial ground motion (Site Class A)** → **Corresponding spectrum (Site Class A)**

2. GROUND MOTION AMPLIFICATION DUE TO SITE EFFECTS (Eastern Canada) cont.

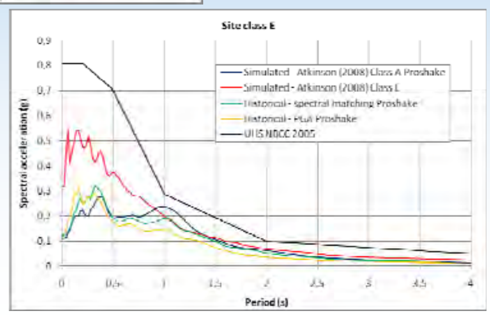


Comparison of mean response spectra at the soil surface:

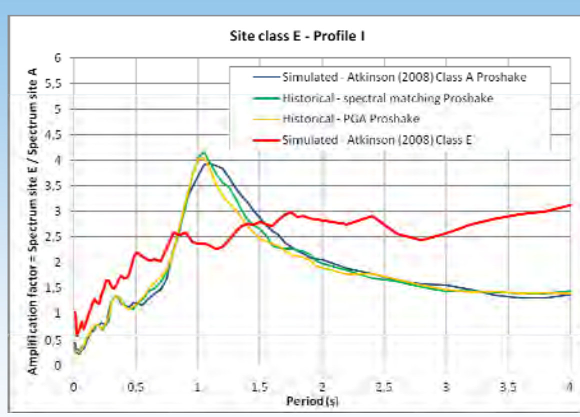
ProShake analysis (simulated and historical; A to D, A to E)

Atkinson (2008) simulated directly for D and E site classes

NBCC UHS

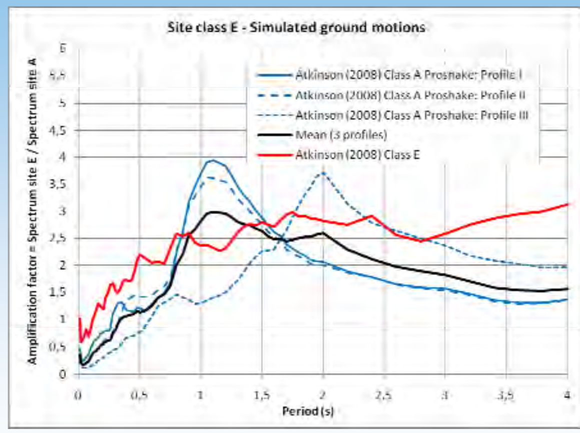


2. GROUND MOTION AMPLIFICATION DUE TO SITE EFFECTS (Eastern Canada) cont.



Comparison of the amplification factors for Soil Profile I and Class E site

2. GROUND MOTION AMPLIFICATION DUE TO SITE EFFECTS (Eastern Canada) cont.



Comparison of the amplification factors for all soil profiles and Class E site

2. GROUND MOTION AMPLIFICATION DUE TO SITE EFFECTS (Eastern Canada) cont.

- **Comparison of ground motion characteristics and inelastic structural response is done for Class E site**

  (i) surface ground motions from ProShake analysis using class A site simulated motions as input at the base of the soil profile

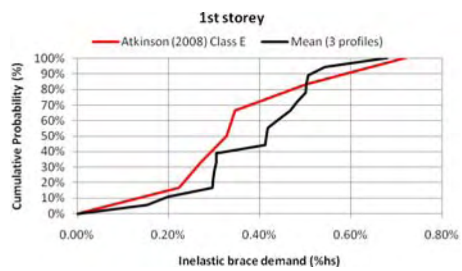
  (ii) simulated motions generated directly for class E site.

The most significant differences between median values:

- peak ground acceleration (0.20 m/s^2 *vs* 0.46 m/s^2)
- number of zero crossings (13 *vs* 64)
- Arias intensity (0.4 m/s *vs* 1,1 m/s)

for sets (i) *vs* (ii), respectively

2. GROUND MOTION AMPLIFICATION DUE TO SITE EFFECTS (Eastern Canada) cont.



2. GROUND MOTION AMPLIFICATION DUE TO SITE EFFECTS (Eastern Canada) cont.

CONCLUSIONS:

- **Site class A simulated and spectrally matched historical ground motions applied at the base of the soil profiles in dynamic soil analysis resulted in ground motions at the soil surface with similar acceleration response spectra;**
- **Amplification was more pronounced at periods close to the natural periods of the soil profile;**
- **The spectra of simulated records that were generated directly for class D and E sites have more uniform amplification over a wider period range, with significantly higher values for short periods compared to results obtained by dynamic soil analysis;**
- **Both group of records induced similar inelastic structural response but local soil conditions or structural characteristics may lead to unconservative results if ground motions simulated directly for D an E sites are used;**
- **Values of F_a and F_v factors may be too conservative for Eastern Canada.**

1

TIME HISTORY SPECTRAL MODIFICATION METHODS FOR EQ RECORDS FOR NL ANALYSIS

P. Léger, C. Combescure, R. Tremblay – ÉPM

GOAL – ASSESS THE ADEQUACY OF TIME DOMAIN WAVELET TRANSFORMS TO OBTAIN SPECTRALLY MATCHED RECORD FOR NL ANALYSES

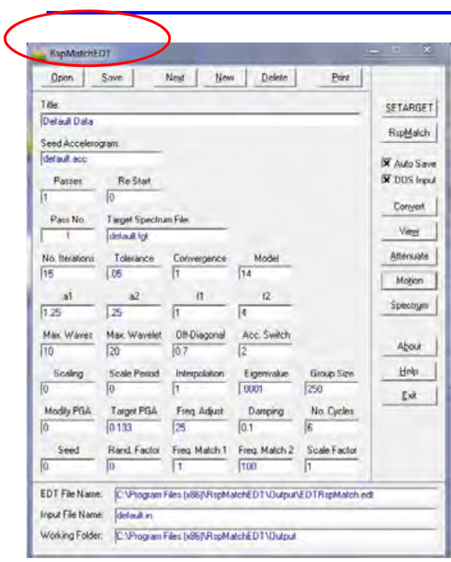
OBJECTIVES - COMPARE FREQUENCY DOMAIN (FD – FFT) AND TIME DOMAIN (TD –WAVELET) METHODS

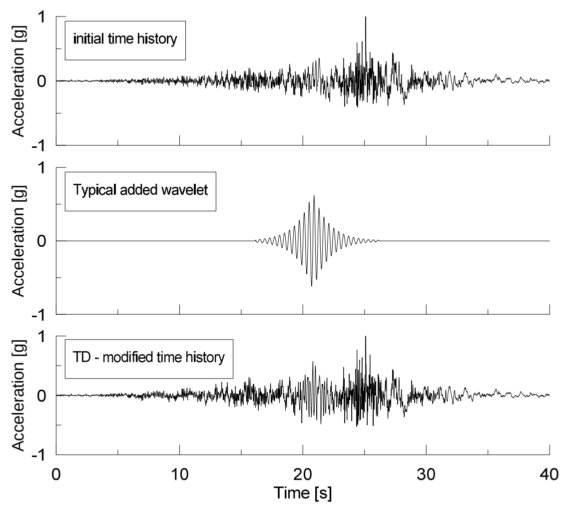
- Ground motion characteristics (PGA, AI, NZC …)
- Elastic response SDOF
- Inelastic response SDOF

TOOLS - SPECTR (FD), RSPMATCH-EDT (TD)

2

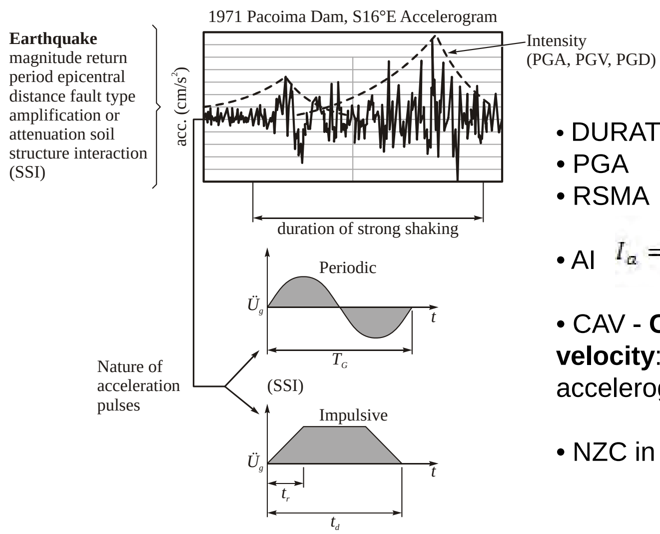
Time Domain Wavelets – Spectrum Compatibility





3

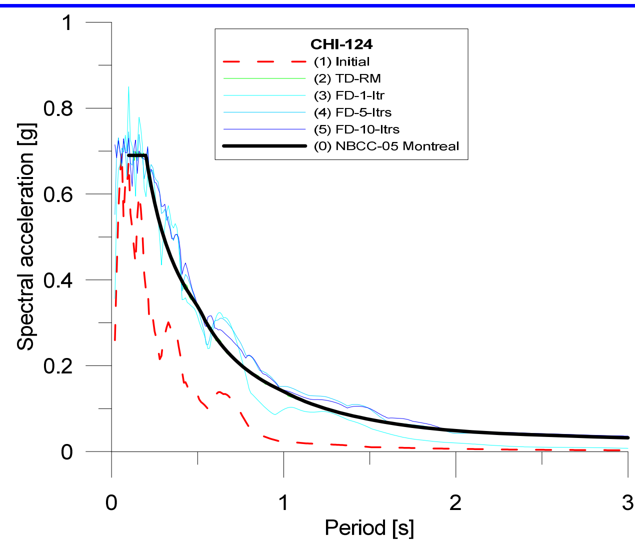
GROUND MOTION CHARACTERISTICS



- DURATION (D3, AI (5%-95%)
- PGA
- RSMA
- AI $I_a = \int_0^{t_d} \ddot{u}_g^2 dt$
- CAV - **Cumulative absolute velocity**: area under the absolute accelerogram $CAV = \int_0^{T_d} |a(t)| dt$
- NZC in D3

4

1988 SAGUENAY, QC, CANADA, STATION CHICOUTIMI 52 KM, 124°, M 5.8, *CHI-124 – RECORD (rock)* *HIGH FREQ.*



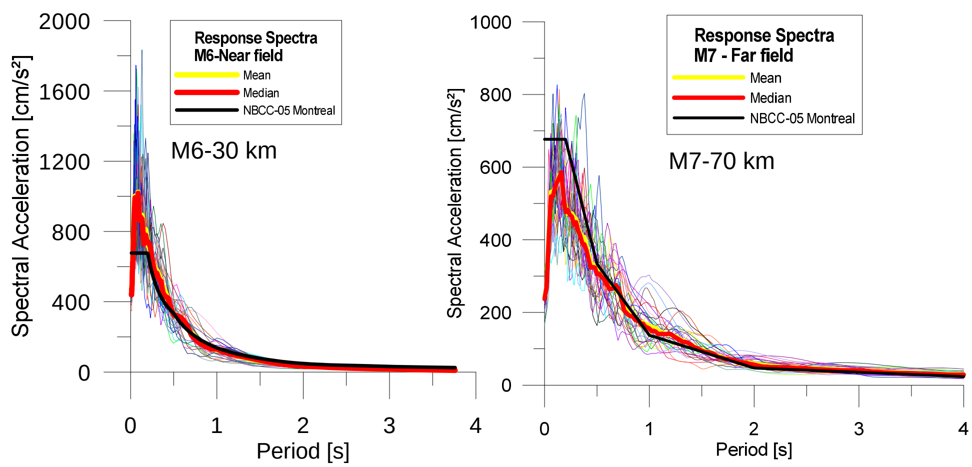
1988 SAGUENAY, *CHI-124 – RECORD (rock)*
HIGH frequency – SMALL periods

5

		D3 [s]	PGA [g]	RMSA [g]	AI [m/s]	CAV [cm/s]	NZC in D3
Original	CHI-124	15.85	0.249	0.035	0.623	719.46	522
FD	CHI-124-10 Itrs	17.04	0.346	0.055	1.535	1194.10	1450
	CHI-124-5 Itrs	19.770	0.350	0.052	1.331	1115.00	778
	CHI-124- 1-Itr	14.890	0.323	0.045	0.991	889.350	483
TD-RM	CHI-124	14.415	0.312	0.041	0.848	819.829	490

BENCHMARK – REFERENCE SOLUTION 30 ACCELEROGRAMS

6



G. ATKINSON – 2010 WEB SITE 2010
30 ACCELEROGRAMS IN EACH BIN (M6 vs M7) Montreal (60 acc. Total)

7

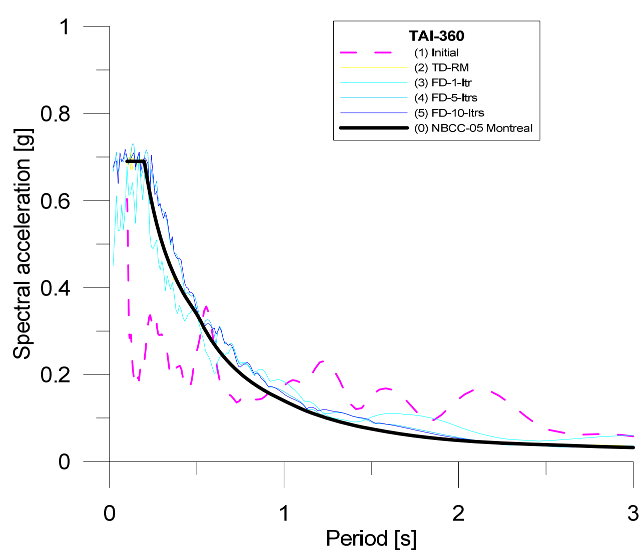
1988 SAGUENAY, *CHI-124 – RECORD (rock)*: *HIGH frequency – SMALL periods*

ELASTIC MODERATE DUCTILITY DUCTILE

PRELIMINARY ANALYSIS – FINAL ANALYSIS … to be developed

8

1999 Chi Chi, Taiwan station TCU047, soil type S_c, 62.2 km, 360°, M 7.6, *TAI-360:* *LOW FREQ – LONG PERIOD*



9

1999 Chi Chi, Taiwan, *TAI-360, LOW FREQ, LONG PERIOD*

		D3 [s]	PGA [g]	RMSA [g]	AI [m/s]	CAV [cm/s]	NZC in D3
Original	TAI-360	16.79	0.266	0.013	0.381	724.51	281
FD	TAI-360-10 Itrs	19.380	0.385	0.026	1.607	1625.69	271
	TAI-360-5 Itrs	18.730	0.401	0.026	1.563	1546.14	263
	TAI-360-1 Itr	17.680	0.402	0.022	1.081	1220.61	239
TD-RM	TAI-360	12.940	0.365	0.017	0.660	863.24	205

10

1999 Chi Chi, Taiwan, *TAI-360, LONG PERIOD*

ELASTIC

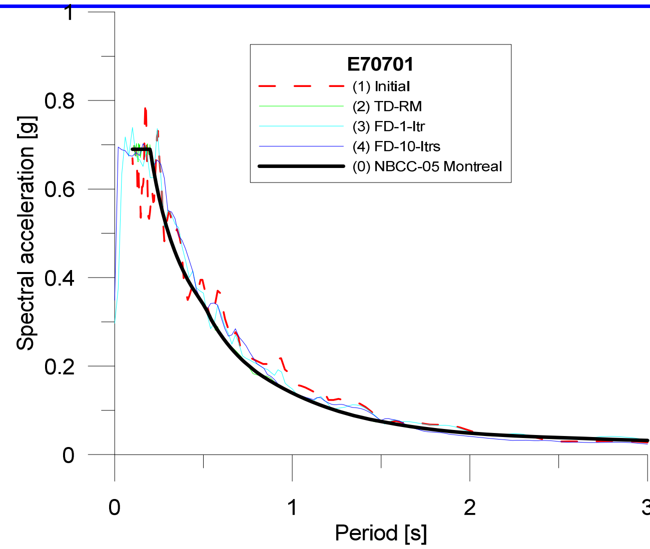
MODERATE DUCTILITY

DUCTILE

PRELIMINARY ANALYSIS – FINAL ANALYSIS … to be developed

11

SYNTHETIC RECORD – M7 AT R=70 KM – MONTREAL (GA)
Far field – Low Freq, long period



12

SYNTHETIC RECORD – M7 AT R=70 KM – MONTREAL (GA)

		D3 [s]	PGA [g]	RMSA [g]	AI [m/s]	CAV [cm/s]	NZC in D3
Original	E70701	16.95	0.271	0.070	1.802	1199.11	400
FD	E70701- 10 Itrs	17.41	0.349	0.093	3.099	1611.52	634
	E70701- 1Itr	16.93	0.297	0.076	2.065	1291.98	473
TD-RM	E70701	16.77	0.275	0.069	1.773	1190.16	408

13

SYNTHETIC RECORD – M7 AT R=70 KM – MONTREAL (GA)

ELASTIC MODERATE DUCTILITY DUCTILE

PRELIMINARY ANALYSIS – FINAL ANALYSIS … to be developed

CONCLUSIONS

- Time domain spectral matching give a better preservation of initial ground motion characteristics than Frequency domain techniques

- Time domain spectral matching using a single (few) record appears to be an adequate substitute to multiple analysis using synthetic records with scalar multiplication to achieve spectrum compatibility

- Extension to real buildings – 3D analyses Comparisons of TD, FD, Push-over …(literature)

- Technical papers – Guidelines (1 MScA, May 10)

14

Time History modifications – frequency domain approach

Gail M. Atkinson

CSRN meeting April 2010

Spectral matching approach to time history generation

- Attempt to combine advantages of real recordings while mitigating some of their limitations
- These approaches often make it easier to reasonably comply with various code or engineering analysis requirements
- The idea is to start with an actual recording and modify it to better fit the target spectrum

Some practical issues that the spectral-matching approach aims to address

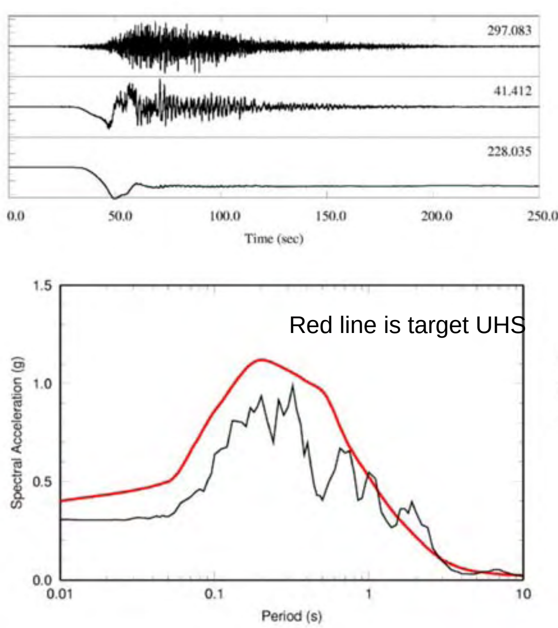
- Difficult to adequately match a smooth UHS over a broad period range with a limited number of records, given their variability (peaks and troughs), with just amplitude scaling
- Use of many records in engineering analyses is expensive and time-consuming – engineers typically want to limit consideration to 1 to 7 records
- Spectral matching techniques often suggested to reduce variability and thereby obtain stable response estimates with fewer records

Goals of record selection need to be defined

- Need to know if we are after the average response, or want to characterize the variability of response
- Average response best achieved by spectrally-matched records, although these can cause biases in response relative to real records
- Variability best characterized by using a larger number of records

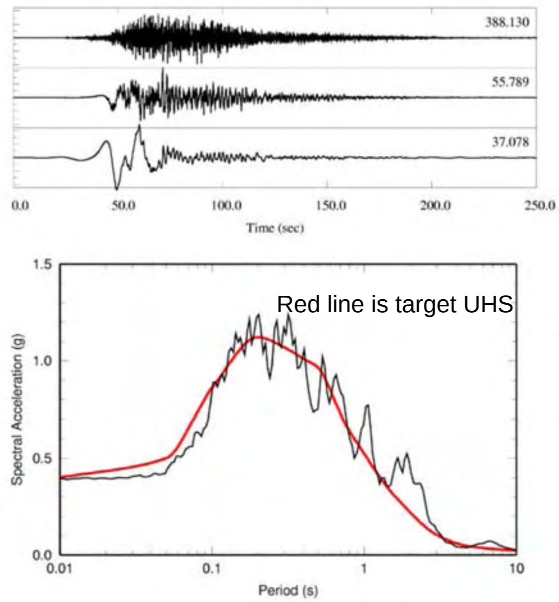
Some alternatives in spectral matching

- No spectral matching (scaled records only) – could leave critical peaks and troughs that strongly determine nonlinear response – OK if using many records
- Some spectral matching to make spectrum approximately follow a smooth target, but leaves peaks and troughs – OK if using a few records
- Tight spectral matching, which make a smooth spectrum without peaks and troughs – OK if using only 1 record, but may produce biased response



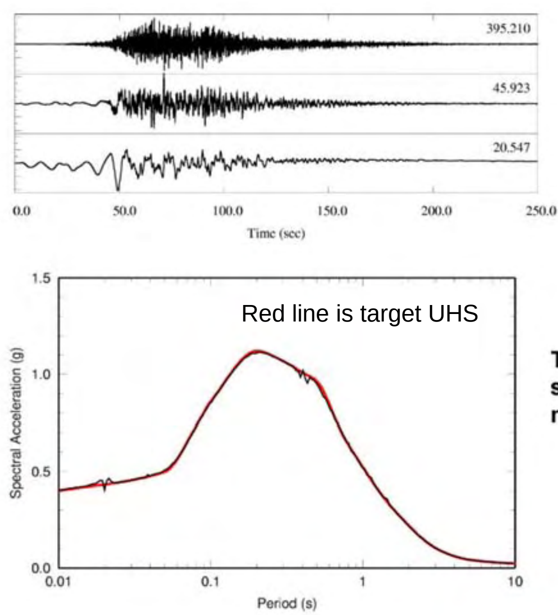
No spectral matching

(from Somerville, COSMOS, 2005)



Loose spectral matching

(from Somerville, COSMOS, 2005)



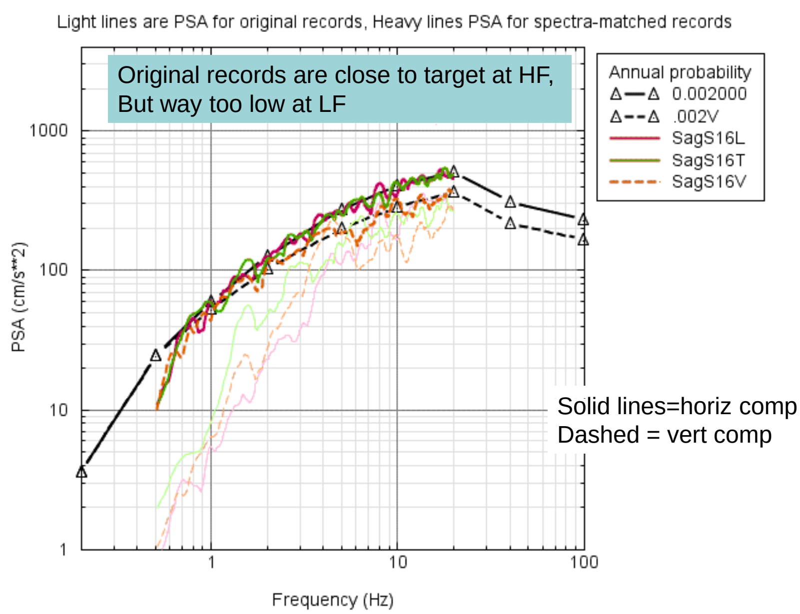
Tight spectral matching

(from Somerville, COSMOS, 2005)

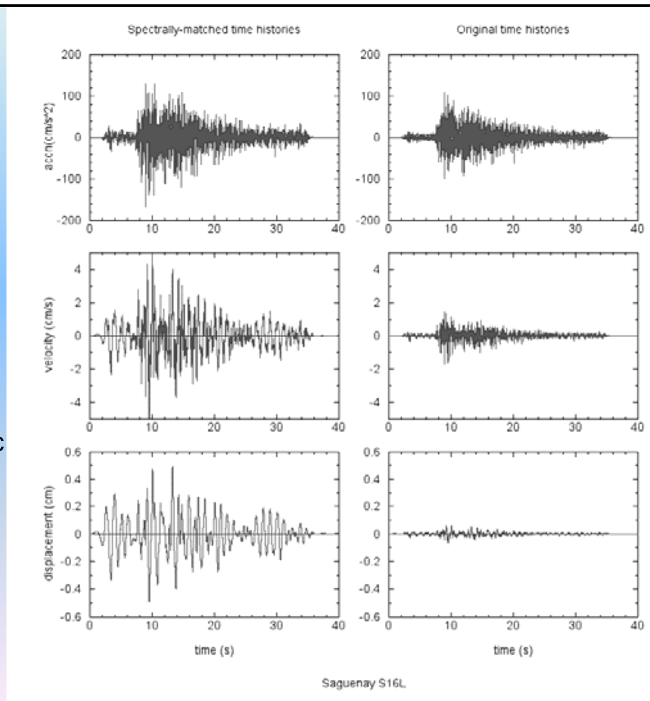
Note on how we do spectral matching –
the frequency-domain approach is highlighted here
Can also be done in the time domain (preferred by some; eg. Abrahamson's RSPMATCH program)

- Take the Fourier transform (FFT) of the input selected record, FA(input). Also compute its response spectrum, PSA(input).
- Compare PSA(input) to the UHS PSA, PSA(targ), for the selected probability level (as function of frequency).
- Multiply the Fourier amplitude spectrum FA(input) by the ratio [PSA(targ)/PSA(input)] at each frequency (leaves phase unchanged).
- Reverse FFT to get a modified time history.
- Iterate a few times, since PSA does not equal FA (correction is approximate, but will converge in a few iterations).
- Baseline correct the modified record.

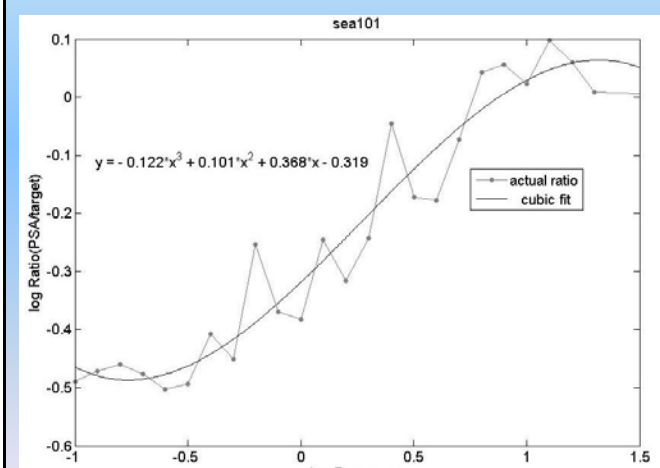
Example:
Matching Saguenay records to a UHS for a site near Charlevoix (10%/50yrs)



Comparison of spectrally matched time histories to original records: Saguenay, Quebec (M5.8 at 52 km)



A spectral-matching approach that preserves peaks/troughs but improves match and thus reduces number of records



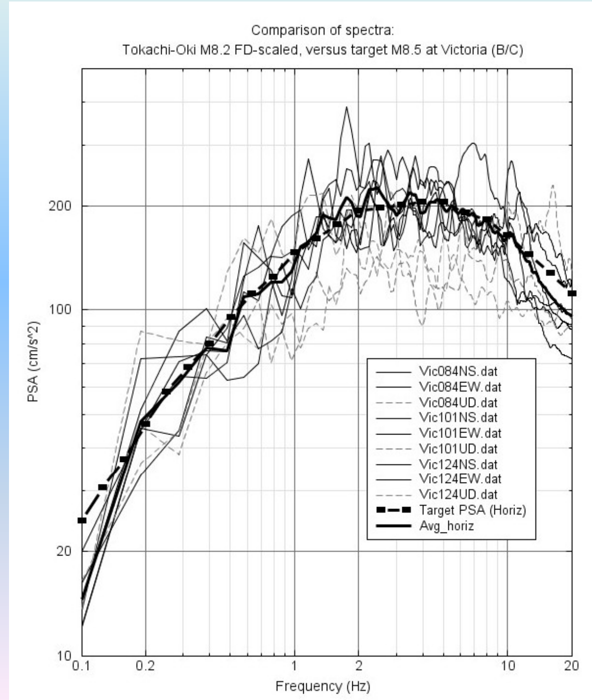
- "Frequency dependent" scaling approach of Atkinson and Macias (BSSA, 2010).

For the input "real record" determine the ratio between the recorded spectrum and the target spectrum (eg. the scenario spectrum or the UHS or the CMS).

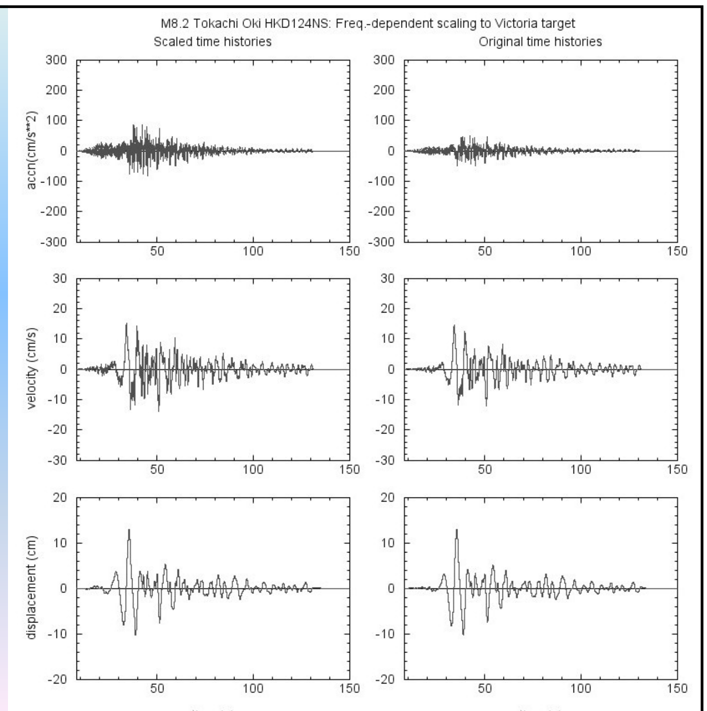
Fit with a polynomial: this becomes a frequency-dependent scaling factor.

Apply FD scaling factor (polynomial) as a single iteration in the frequency-domain scaling approach, then reverse FFT to get lightly-modified time history.

Example: Match records from the **M**8.2 Tokachi Oki subduction earthquake in Japan (3 records, each 3 components) to the spectrum expected for a **M**8.5 Cascadia mega-thrust event at Victoria on B/C site conditions



Example of lightly-modified time history: original and scaled records very similar in acceleration, velocity, displacement.



Some concluding remarks

- Spectral matching is a useful technique to reduce the number of records needed to match a target
- Can use loose or tight spectral matching depending on objectives (but tight matching will not capture variability in response)
- Matching can be done in either time or frequency domains – the key is to check that reasonable acceleration, velocity and displacement time series are obtained

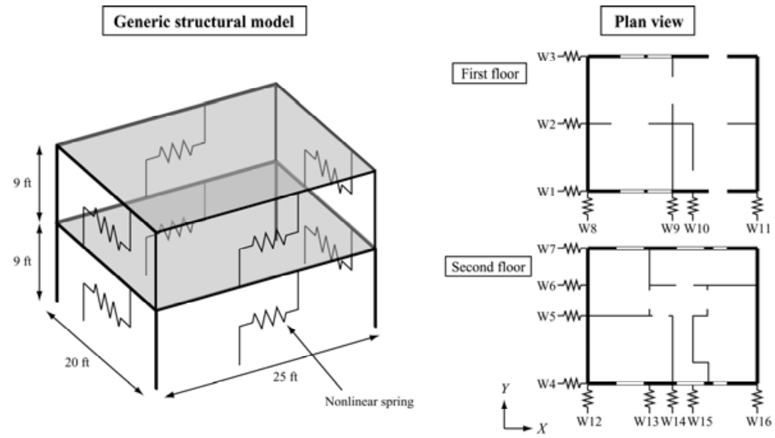
Impact of Record Selection Procedures on Seismic Performance of wood-frame houses in Southwestern British Columbia

Katsu Goda & Gail Atkinson
University of Western Ontario

Objectives

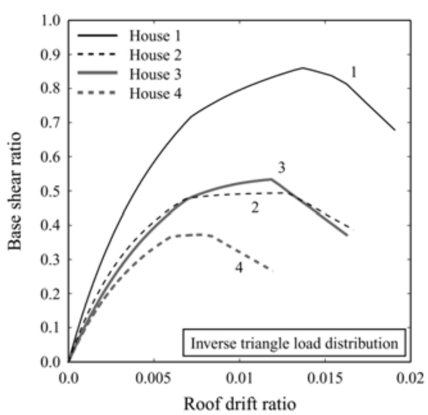
- Investigate the seismic performance of conventional wood-frame houses in south-western British Columbia.
- Utilize available structural models – UBC-SAWS model developed by Prof. C. Ventura
- Utilize up-to-date tools: Uniform Hazard Spectra (UHS), Conditional Mean Spectrum (CMS), and Incremental Dynamic Analysis (IDA).
- Take into account seismic hazard characteristics due to different earthquake types (crustal, inslab, and interface events)
- Focus on "impact of record selection" and "impact of shear-wall types"

UBC-SAWS Models



- The UBC-SAWS models were developed/calibrated by researchers at the University of British Columbia (lead by Prof. C. Ventura) based on experimental results of various shear-walls and two-story house models.

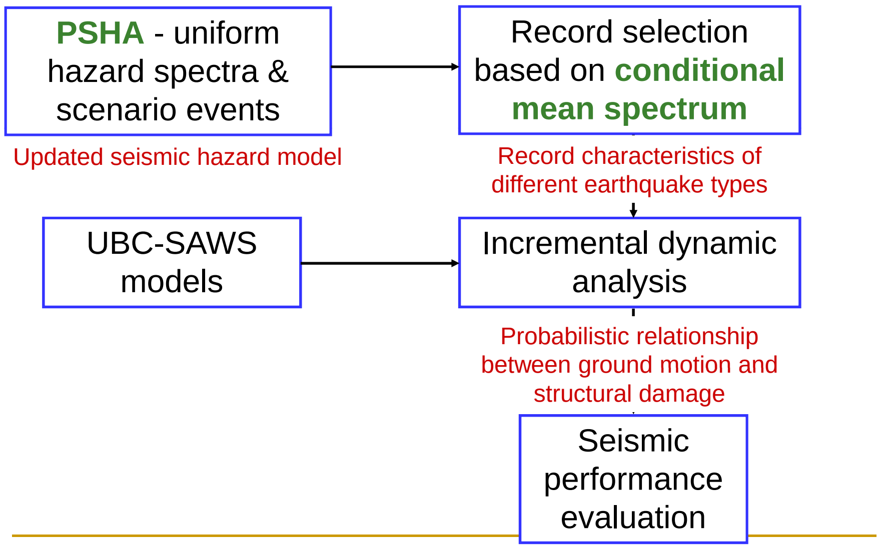
Pushover Curves of UBC-SAWS Models



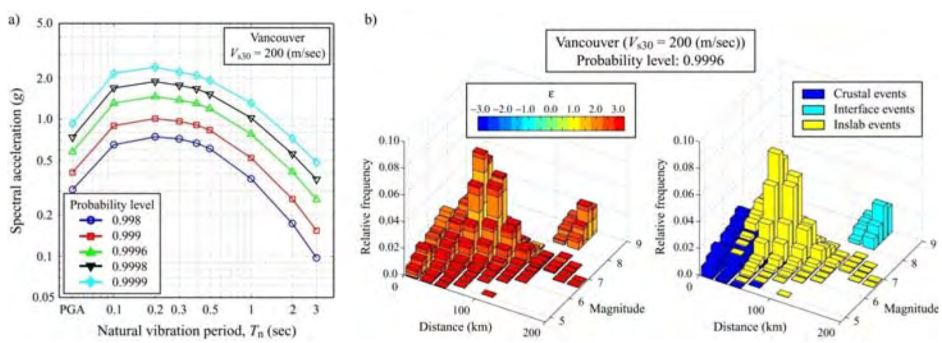
- Four UBC-SAWS models are available: House 1 - stucco/engineered OSB/GWB; House 2 - engineered OSB/GWB; House 3 - non-engineered OSB/GWB; and House 4 - horizontal boards/GWB
- House 1 with stuccos has a high seismic resistance in comparison with other houses.

OSB=oriented strandboard (plywood)
GWB=gypsum wallboard

Seismic Performance Evaluation



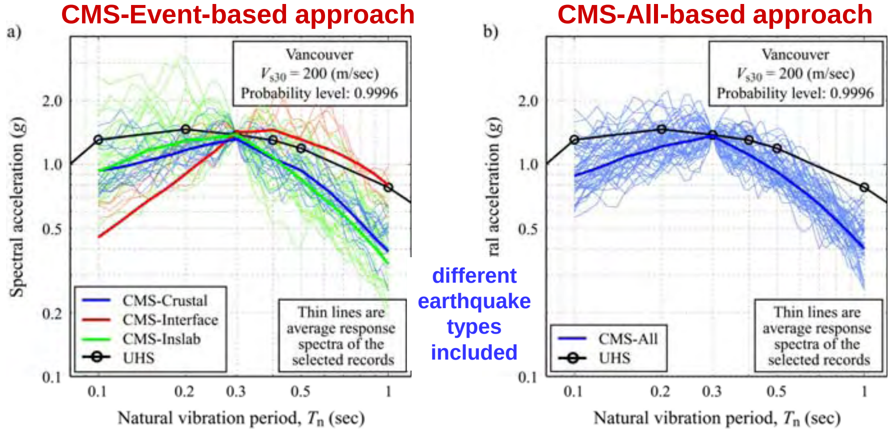
PSHA – UHS and Scenarios



- In western Canada, three types of earthquakes contribute to overall seismic hazard significantly: shallow crustal earthquakes, deep inslab earthquakes, and interface Cascadia earthquakes.
- As the probability level (of non-exceedence) increases, contributions of inslab events gradually increase.

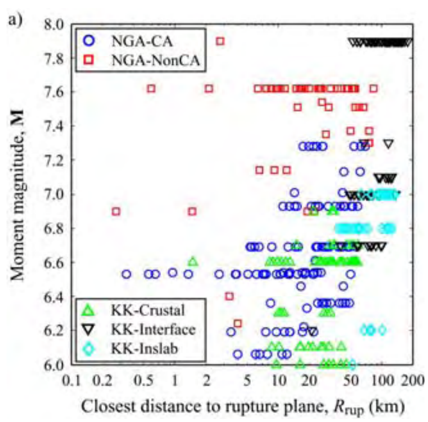
Target Response Spectrum and CMS

CMS-Event-based approach **CMS-All-based approach**



- The Conditional Mean Spectrum (CMS) is a useful tool to define a target response spectrum, consistent with uniform hazard spectra by taking adequate correlation of spectral accelerations at different periods - Avoid "overestimation" of ground motions and structural responses!

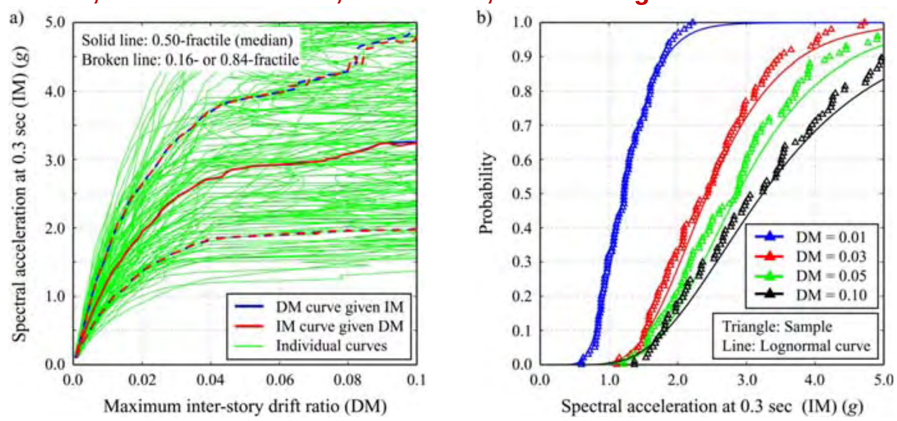
Ground motion records and nonlinear dynamic analysis



- We constructed a large pool of ground motion records suitable for IDA from PEER-NGA and K-NET/KiK-NET databases.
- The pool includes records with relatively large PGA and PGV values – 368 records from 51 earthquakes are selected.
- We carry out nonlinear dynamic analysis of Houses 1-4 by varying seismic intensity measure (IM) (= spectral acceleration at 0.3 sec) from 0.1 to 8.0 *g*.
- We obtained the set of seismic demand measures (DM) (= maximum inter-story drift ratio at the first story level).

Typical IDA Results

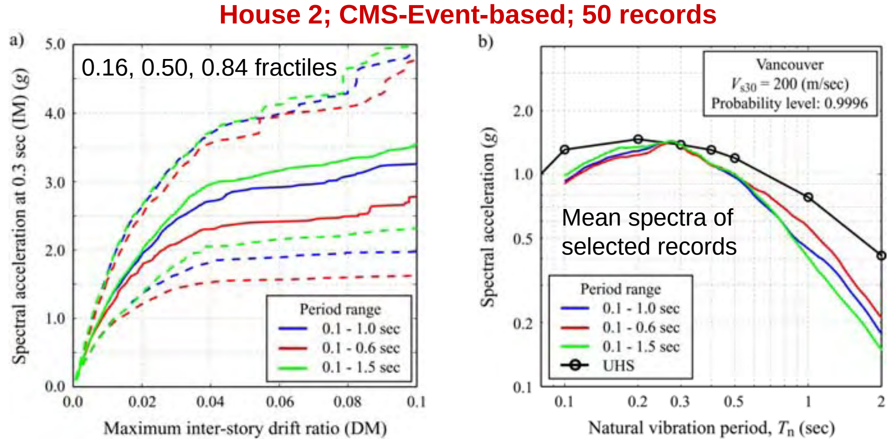
House 2; CMS-Event-based; 50 records; Period range from 0.1 to 1.0 sec



- IDA analysis produces a probabilistic relationship between Intensity measure (IM) and Damage Measure (DM) – useful for seismic performance evaluation, such as the calculation of probability of reaching a specific DM level given an IM level.

Effects of vibration period range (wide vs. narrow band)

House 2; CMS-Event-based; 50 records

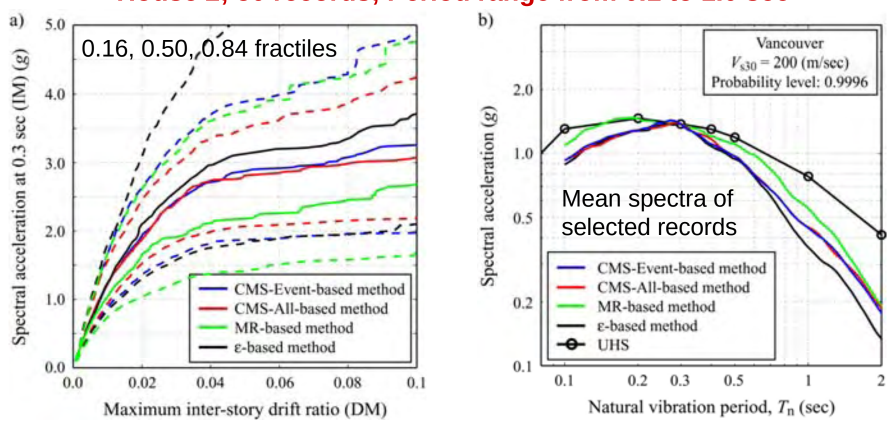


- The vibration period range (wide vs. narrow band) for which response spectrum of a record is matched with the target spectrum has impact on nonlinear response potential – this is related to the extent of induced nonlinearity of the selected records.

Effects of record selection criteria

(for a single period band criterion)

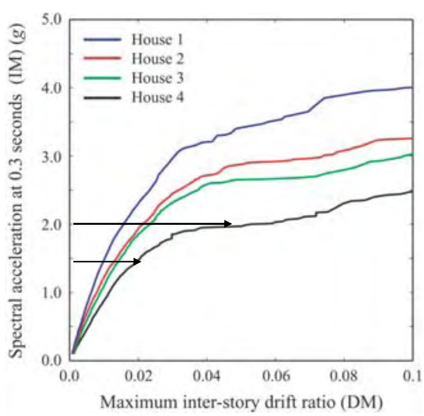
House 2; 50 records; Period range from 0.1 to 1.0 sec



- Different record selection criteria lead to different IM-DM curves.
- "CMS-Event" has more variability than "CMS-All".
- Nonlinear response potential: "MR-method" > "CMS-Event" > "MR-ε-method" - this can be explained by inspecting "response spectral shape"

Effects of Shear-wall Types

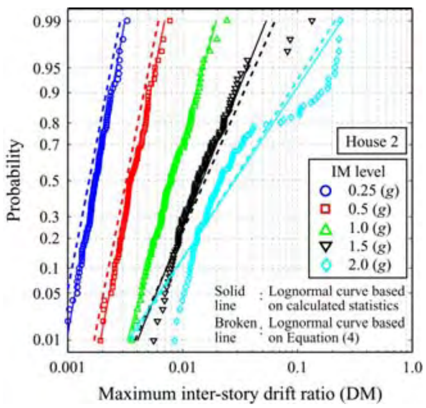
CMS-Event-based; 50 records; Period range from 0.1 to 1.0 sec (<u>median curves only</u>)



- The shear-wall types have significant impact on expected damage levels for a given seismic hazard level.
- For example, House 1 will experience DM equal to about 0.007 and 0.014 (which are minor damage) given IM = 1.5 and 2.0 *g*.
- On the other hand, House 4 will experience DM equal to about 0.02 and 0.05 (moderate to extensive damage) given IM = 1.5 and 2.0 *g*.

Statistical Model for Max Inter-story Drift

House 2; CMS-Event-based; 50 records; Period range from 0.1 to 1.0 sec



- We developed simple statistical models of the maximum inter-story drift ratio given seismic intensity level for Houses 1-4 using the lognormal distribution.
- Median and logarithmic standard deviation are characterized.
- Such statistical models can be useful to facilitate the seismic risk analysis in performance-based earthquake engineering applications.

Summary and Conclusions

- Seismic performance of conventional wood-frame houses was evaluated using UHS, CMS, and IDA and considering seismic characteristics of different earthquake types.
- IDA constructs a probabilistic relationship between seismic hazard and structural response, which is particularly valuable for performance-based earthquake engineering – We developed a simple statistical model for seismic risk analysis and seismic loss estimation.
- The results indicate that House 1 (as well as Houses 2 and 3) is associated with minor seismic risk (at the seismic hazard level specified in building codes), whereas House 4 may be subjected to extensive seismic damage.

Conditional Mean Spectrum – account for inter-period correlations in record selection to "match" UHS. Defn:

$$\mu_{\ln S_a(T_i)|\ln S_a(T_n)} = \mu_{\ln S_a}(\overline{\mathbf{M}}, \overline{R}, T_i) + \rho(T_i, T_n)\overline{\varepsilon}(T_n)\sigma_{\ln S_a}(T_i)$$

Baker [14] proposed a CMS-based record selection procedure for the seismic performance evaluation of a structure. The procedure begins by specifying a target seismic intensity level, in terms of $S_a(T_n)$, and representative scenario(s), in terms of $\overline{\mathbf{M}}$, $\overline{R}$, and $\overline{\varepsilon}$ (see Figure 3 and Table I). By adopting an adequate GMPE for the considered scenario, one can evaluate the mean and standard deviation of natural logarithm of the spectral acceleration at the vibration period T_i, denoted by $\mu_{\ln Sa}(\overline{\mathbf{M}}, \overline{R}, T_i)$ and $\sigma_{\ln Sa}(T_i)$. Then, the CMS, in natural logarithmic space, is given by:

$$\mu_{\ln S_a(T_i)|\ln S_a(T_n)} = \mu_{\ln S_a}(\overline{\mathbf{M}}, \overline{R}, T_i) + \rho(T_i, T_n)\overline{\varepsilon}(T_n)\sigma_{\ln S_a}(T_i), \quad (1)$$

ε is the number of σ from median

where $\rho(T_i, T_n)$ is the inter-period correlation of spectral accelerations at vibration periods T_i and T_n. Baker and Cornell [24] carried out empirical analysis of the inter-period correlation using California records, and proposed the following prediction equation:

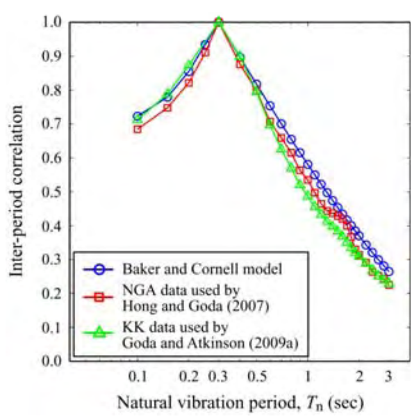
$$\rho(T_{n1}, T_{n2}) = 1 - \cos\left(\pi/2 - [0.359 + 0.163 I_{T_{min}<0.189} \ln(T_{min}/0.189)] \ln(T_{max}/T_{min})\right), \quad (2)$$

where T_{max} and T_{min} are the larger and the smaller of T_{n1} and T_{n2}, respectively, and $I_{T_{min}<0.189}$ is the indicator function that equals one if T_{min} is less than 0.189 sec and equals zero otherwise. We note that

Conditional Mean Spectrum

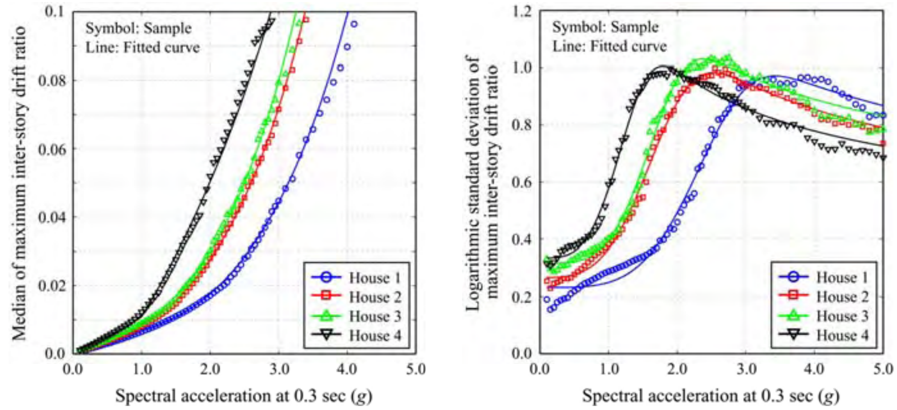
$$\mu_{\ln S_a(T_i)|\ln S_a(T_n)} = \mu_{\ln S_a}(\overline{\mathbf{M}}, \overline{R}, T_i) + \rho(T_i, T_n)\overline{\varepsilon}(T_n)\sigma_{\ln S_a}(T_i)$$

CMS = UHS + correlationCoef *residuals



- Conditional Mean Spectrum takes inter-period correlation of spectral accelerations at different vibration periods.
- Useful when the target response spectrum is defined in tandem with UHS (because UHS ordinates at different vibration periods do not represent spectral characteristics of a single record)

Statistics of Max Inter-story Drift



- The model parameters for the lognormal distribution, median and logarithmic standard deviation, are characterized in terms of seismic intensity levels for Houses 1-4.

Impact of new estimates of seismic hazard for eastern vs. western Canada

Gail Atkinson & Katsu Goda
University of Western Ontario

Objectives

- Provide updated seismic hazard models for eastern and western Canada
- These are long overdue as NBCC 2005-2010 estimates are actually based on calculations/technology/information as of 1995 – thus 15 years out of date.
- Difficulties arise because current seismic hazard estimates, as used in site-specific and industry-type studies over the last decade or so, may differ markedly from the “NBCC standard”.

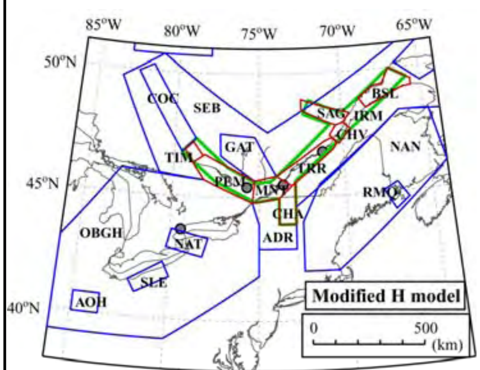
Updated Hazard Analysis Aspects

Updated seismic hazard models

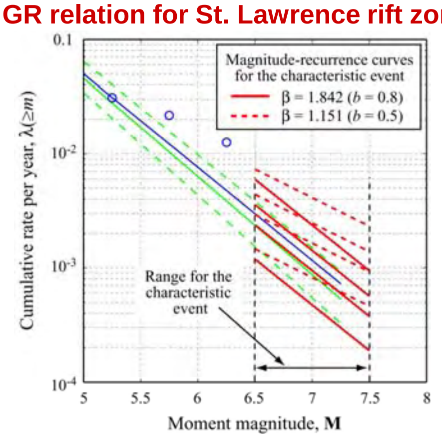
- **New seismicity rates (low impact)**
- **Proper conversion of magnitude statistics to moment M scale (moderate impact)**
- **New seismic source models (important only in east)**
- **New ground motion prediction equations (important all areas) – a suite of GMPEs from last decade are used**
- **Correct implementation of finite-fault measures in western GMPEs (moderate importance in west)**
- **Probabilistic inclusion of Cascadia subduction events (important for long periods in the west)**

Updated Seismic Hazard Model – changes in seismic source zone characterization in East

Modified seismic source zones



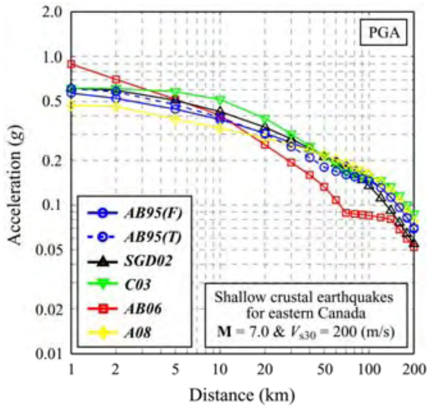
GR relation for St. Lawrence rift zone



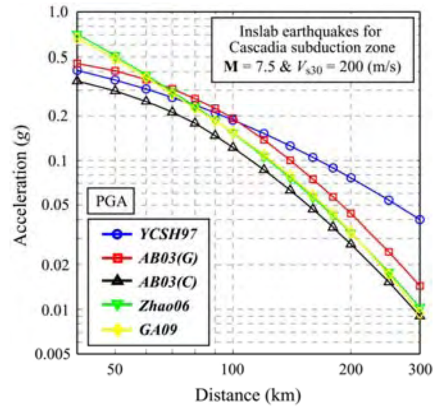
- Seismic rates are reevaluated using a longer and homogeneous *CCSC09* earthquake catalog compiled by Macias et al.
- For the **St. Lawrence rift region** (IRM, **green color**), small-to-moderate events are characterized by several GR relations for smaller zones, whereas large events are characterized by a semi-characteristic model.

Updated Seismic Hazard Model – changes in GMPEs

Eastern Canada



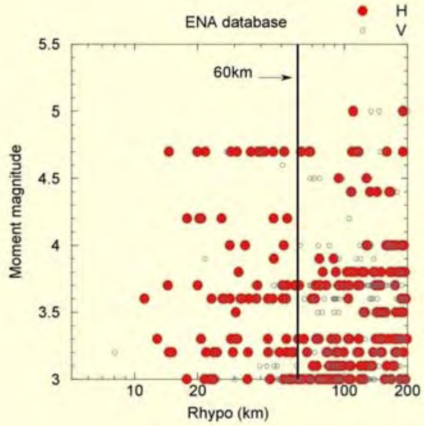
Western Canada (inslab events)



- Ground motion prediction equations have most significant impact on seismic hazard estimates.
- We consider multiple recent ground motion prediction equations to account for epistemic uncertainty regarding their selection.

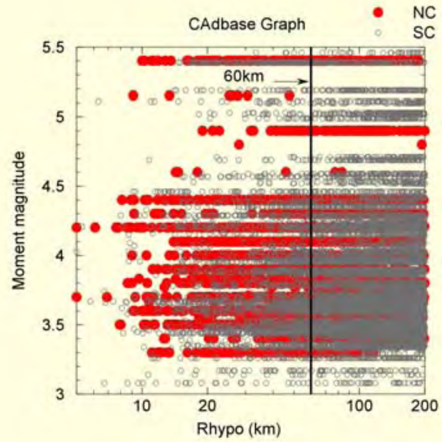
Some notes on the GMPEs

- At recent SCED meeting much was made over the fact that AB06 is the "lowest" of recent GMPEs for ENA – it has been questioned by many for this reason
- Note that we use a range of GMPEs, not just AB06 in the seismic hazard analysis; this follows standard practice (same approach used by USGS, and for site-specific analyses for facilities in the east over the past 5 years)
- Re "fit" of AB06 to data: AB06 was compared against the existing ground-motion data in the east to a much greater extent than most other GMPEs
- many popular GMPEs (Frankel, 1996; Campbell, 2003) did not contain ANY comparisons to data



Recent work on comparing ENA ground-motion data to California data

ENA vs. California (northern CA, southern CA) ground-motion data for moderate events

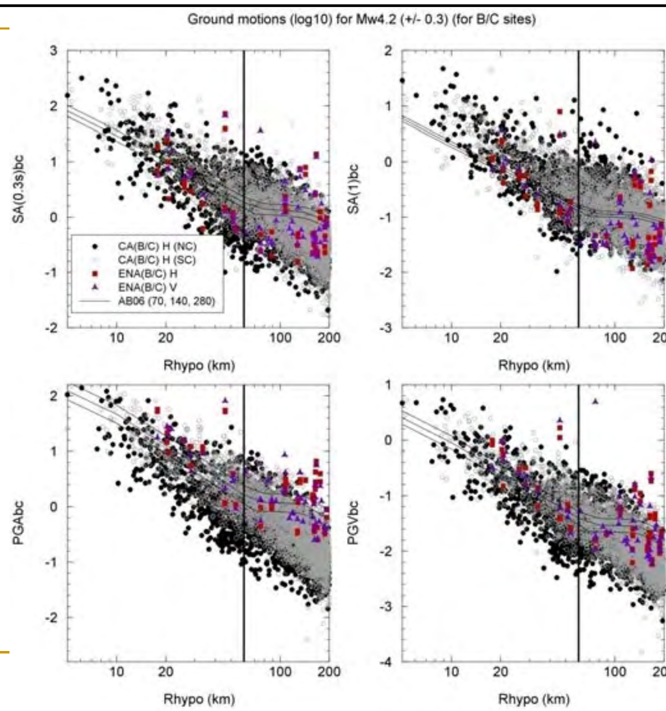


Moment M=4.2 (+/- 0.3)

All data corrected to equivalent amplitudes for B/C (760 m/s) using BA08 linear amplification factors

Note main differences are at R>60km

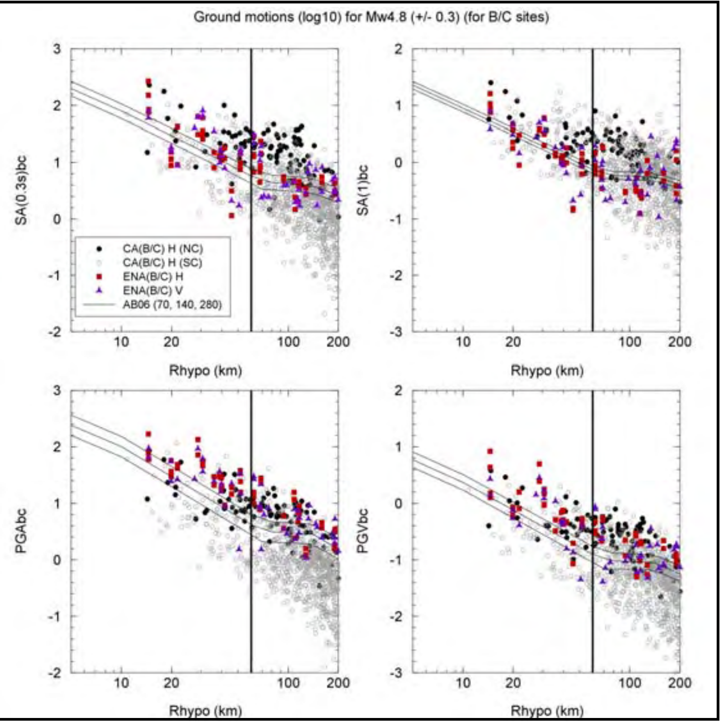
(all ENA data processed by Karen Assatourians)



Moment M=4.8 (+/- 0.3)

All data corrected to equivalent amplitudes for B/C (760 m/s) using BA08 linear amplification factors

ENA lower than NC at 0.3s to 1s for R<60km? (but PGA higher)

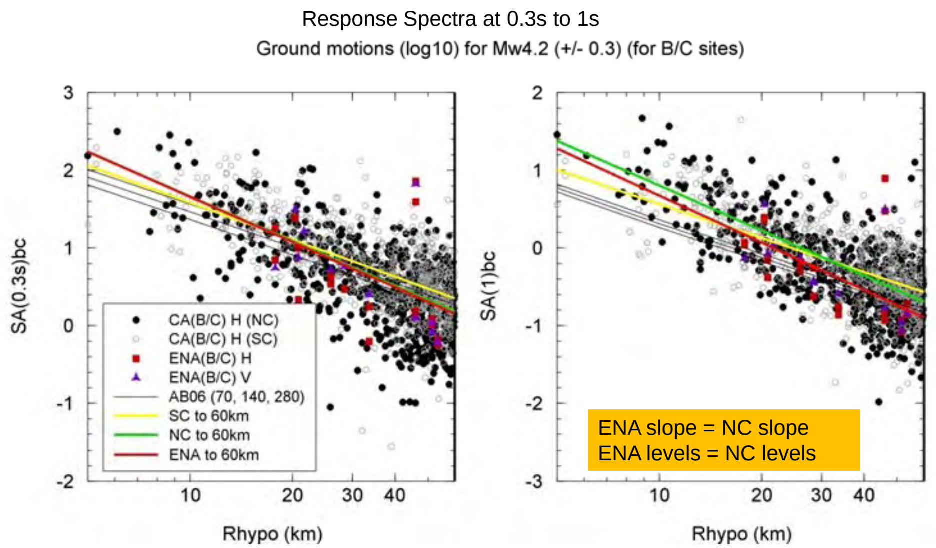


Regress logY for M3.2 to M5.0, for Rhypo<60km SC, NC, ENA

To incorporate observed dependence of slope on magnitude:

$$\log Y = c1 + c2\,M + c3 \log R + 0.1(M-4) \log R$$

Compare simple regression lines (for data to 60 km) for SC, NC, ENA: M4.2



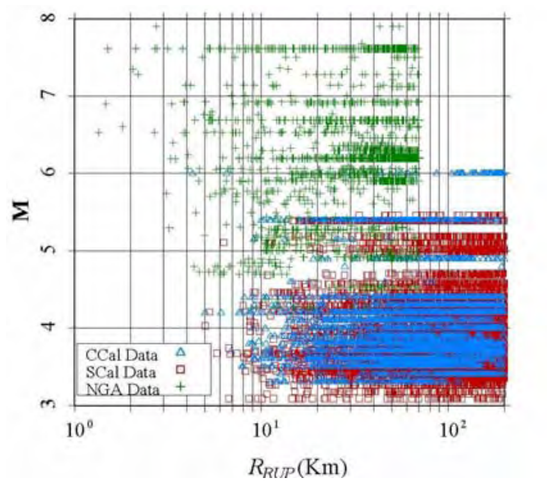
Lessons from small-to-moderate events

- *ENA spectral amplitudes at T~0.3s to T~1s approximately equal to those in Northern CA for R<60km*
- *Attenuation rates for R<60 km in ENA very similar to those in Northern CA*
- *PGA in ENA markedly higher than in CA*
- *We should expect the same general trends at larger magnitudes……*

Compare ENA vs. CA predictions for magnitudes that contribute most to hazard

- ShakeMap data constrain GMPEs for M<5
- NGA equations shown to be applicable for M>=6 (but biased high for M<6; see Atkinson and Morrison, 2009; Chiou et al., 2010)

(figure from Chiou et al., 2010)

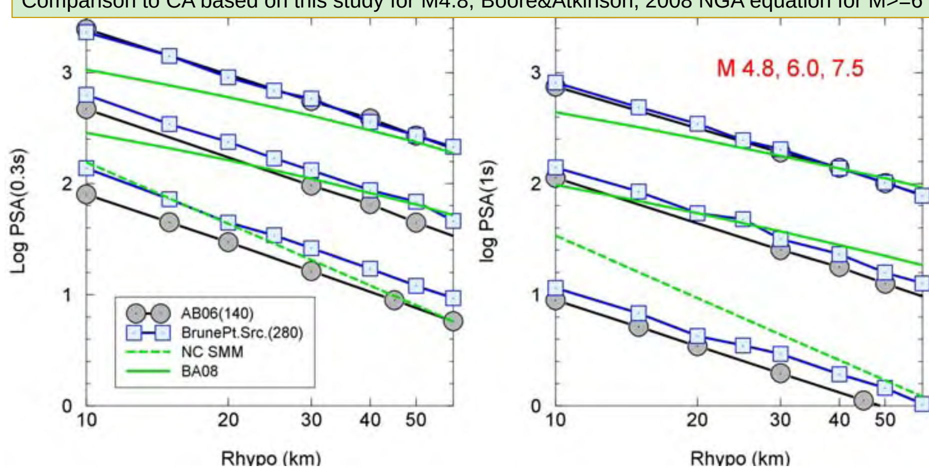


Comparing ENA to CA GMPEs – full magnitude range of interest

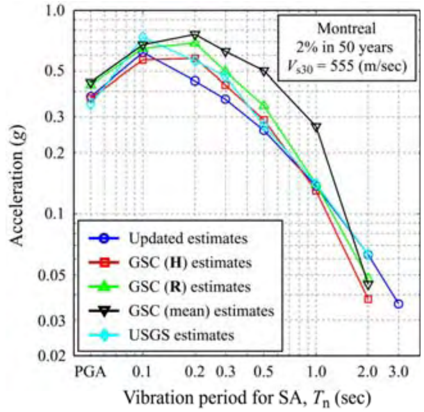
For PSA (0.3 to 1s) ENA model-based GMPE of AB06 may tend to underpredict moderate events R<50 km, but overpredict M>6 at R<25 km (lack of near-source saturation)

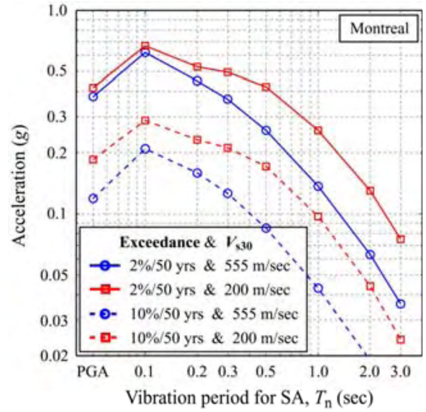
Ground motions (log10 cgs units) for B/C sites: M4.8, 6.0, 7.5

Comparison to CA based on this study for M4.8, Boore&Atkinson, 2008 NGA equation for M>=6



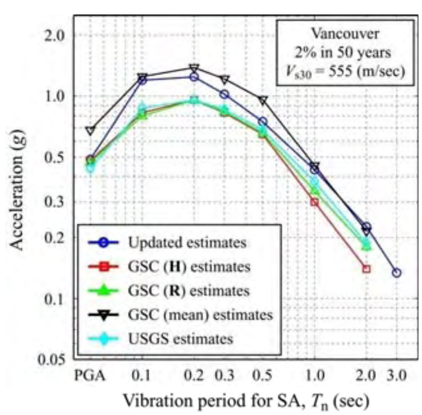
Seismic Hazard Assessment - Montreal

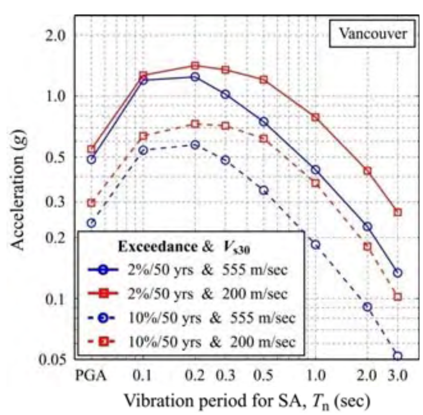




- Updated seismic hazard estimates for Montreal are lower than mean and median estimates based on the current GSC model.
- Our mean-hazard UHS for Montreal is lower than USGS (2008) by ~20% at short periods
- Soft soil condition increases seismic hazard estimates for longer vibration periods significantly.

Seismic Hazard Assessment - Vancouver

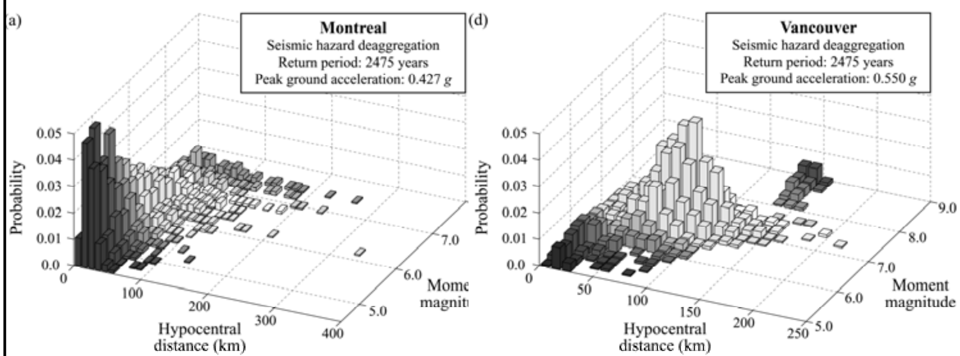




- Updated seismic hazard estimates for Vancouver lie between mean and median estimates based on the current GSC model
- Our mean-hazard UHS for Vancouver is higher than USGS (2008) by ~30% at short periods
- Soft soil condition increases seismic hazard estimates for longer vibration periods significantly.

Deaggregation Analysis -

Montreal **Vancouver**



- Seismic hazard deaggregation shows the characteristics of contributing seismic events at a selected probability level – PGA shown here
- For Montreal, seismic hazard (short periods and PGA) is dominated by M~5 to 6.5 at R<50 km
- For Vancouver in-slab events dominate for short periods (and PGA)

Laura Kreykenbohm, James Traber, and Yan Yang
Department of Civil Engineering
University of British Columbia

UBC Earthquake Engineering Research Facility

Regional Seismic Risk Assessment in British Columbia

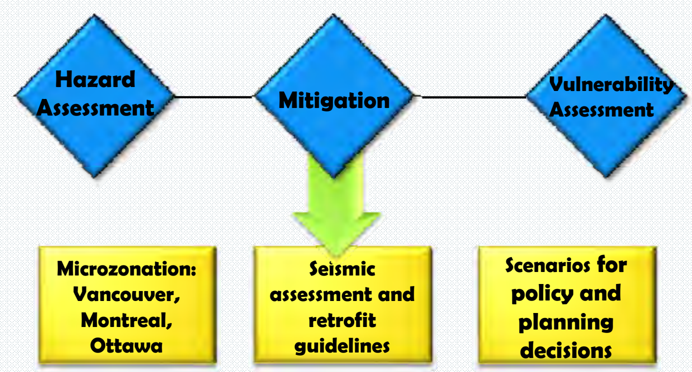
Building Inventory Analysis and Microtremor Testing

Overview

1. Background of research
2. Building Inventory Analysis
3. Microtremor Testing

CSRN

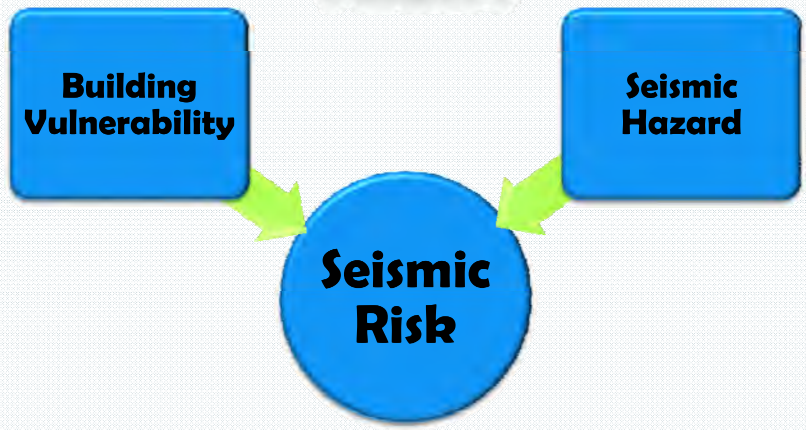
- Canadian Seismic Research Network (CSRN) involves 8 Canadian Universities
- Research focused on Hazard and Vulnerability Assessment, and Mitigation for major Canadian cities.



Project Outline

- Complete a seismic risk assessment for major population centers of British Columbia
  - Estimate economical cost
  - Approximate causality rate
- Data collection
  - Micro-tremor testing
  - Building inventory
- Calculate damage matrix for British Columbia

What is Seismic Risk?



Seismic Hazard

- The intensity measures the destructiveness of the earthquake
- Intensity Scale is different from the Richter Magnitude Scale in that the effects of any one earthquake change from place to place
- Depends on:
  - proximity of sources
  - path
  - site conditions

MMI. description of effects
VI. Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.
VII. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.
VIII. Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.
IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.
X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.
XI. Few, if any (masonry) structures remain standing. Bridges destroyed. Rails bent greatly.
XII. Damage total. Lines of sight and level are distorted. Objects thrown into the air.
Source: U.S. Geological Survey - [illegible]

Obtaining Data

- Micro-Tremor Testing
- Building Inventory

Micro-Tremor Testing

Equipment used:

- Pinocchio Velocity Transducers
  - Measures the velocity of ground movement
  - From the test our team can determine the dominant frequency of the soil at a location as well as the amplification factor between horizontal and vertical components.

Procedure

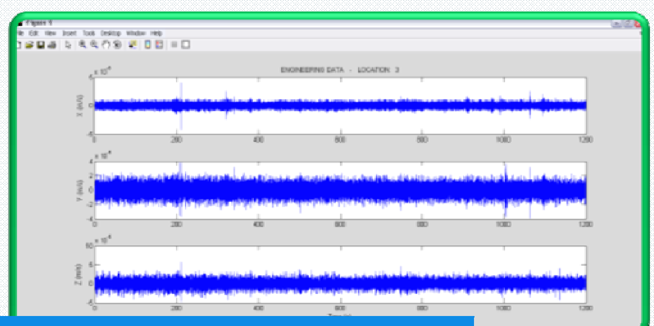
In The Field:

1. Program the Pinocchios
2. Run the test for 10-20 minutes
3. Retrieve data, complete field check

Data Analysis:

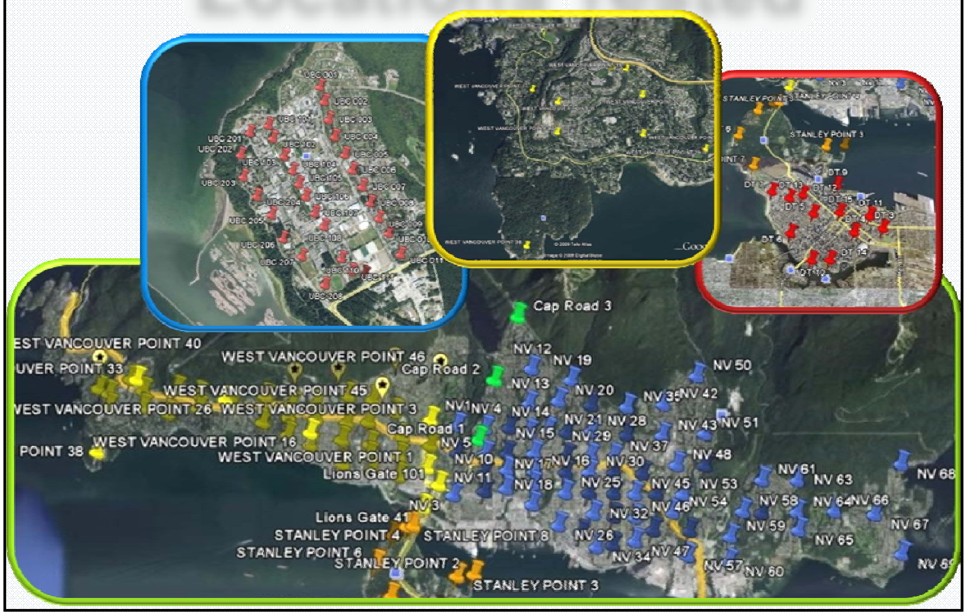
4. Analyse in Matlab
5. Export to Artemis
6. Compare type of peak in Artemis to location of dominant peaks

Results



Location	Time into test	Peak Value	Type of Peak
North Van 1	1.92	0.06	Horizontal
North Van 2	2.24	0.11	Horizontal

Locations Tested



Building Inventory

Building Inventory Form			Reviewer:	HT & KD
Address:	138 Esplanade	Postal Code: V7M 1A2	Date:	June 18th, 2009
Building Name:	Premiere		Zone:	CD-393
Primary Use:	Residential		Photo:	32-34
No. of Storeys:	12	Year Built:	Footprint Area:	680, podium 1450
Shape:	Rect.		Storefront:	Y
Prototype: (Bold Appropriate, and Enter here)		CFHR	Pounding:	N
Wood	WLFR	Wood Light Frame, Residential	Retrofit:	N
	WLFCI	Wood Light Frame, Commercial/Inst.	Adj. Building Types:	
	WLFLR	Wood Light Frame, Low Rise	Soil Type:	
	WLFCR	Wood Light Frame, Commercial/Residential	Photo/Sketch:	
	WPB	Wood Post and Beam		
Steel	LMF	Light Metal Frame		
	SMFLR	Steel Moment Frame, Low Rise		
	SMFMR	Steel Moment Frame, Mid Rise		
	SMFHR	Steel Moment Frame, High Rise		
	SBFLR	Steel Braced Frame, Low Rise		
	SBFMR	Steel Braced Frame, Mid Rise		
	SBFHR	Steel Braced Frame, High Rise		
	SFCWLR	Steel Frame Concrete Walls, Low Rise		
	SFCWM	Steel Frame Concrete Walls, Mid Rise		
	SFCWHR	Steel Frame Concrete Walls, High Rise		
	SFCI	Steel Frame with Concrete Infill Walls		
	SFMI	Steel Frame with Masonry Infill Walls		
Concrete	CFLR	Concrete Frame with Concrete Walls, Low Rise		
	CFMR	Concrete Frame with Concrete Walls, Mid Rise		
	CFHR	**Concrete Frame with Concrete Walls, High Rise**		
	RCMFLR	Reinforced Concrete Moment Frame, Low Rise		
	RCMFMR	Reinforced Concrete Moment Frame, Mid Rise		
	RCMFHR	Reinforced Concrete Moment Frame, High Rise		
	RCFIW	Reinforced Concrete Frame with Infill Walls		
Masonry	RMLR	Reinforced Masonry Shear Wall, Low Rise		
	RMMR	Reinforced Masonry Shear Wall, Mid Rise		
	URMLR	Unreinforced Masonry Bearing Walls, Low Rise		
	URMMR	Unreinforced Masonry Bearing Walls, Mid Rise	Notes:	
Tilt Up	TU	Tilt Up		
Precast	PCLR	Precast Concrete, Low Rise		
	PCMR	Precast Concrete, Mid Rise		
Mobile	MH	Mobile Homes		

- **Obtained to create a damage matrix for every city block**
- **Has 32 different prototype classifications each with their own damage assessment**

Computer Survey

Preliminary Analysis:

- Retrieve data provided by municipalities
- Fill in blanks using different web sources
- Quick overview of Area
  - Primarily of use in deciding which areas to further investiga t e

Taken from Bing.com

Main Programs:

- Excel, Access
- Google Earth
- Bing Maps
- Batch Geocode
- Municipality GIS sites

CNV GIS

Foot Survey

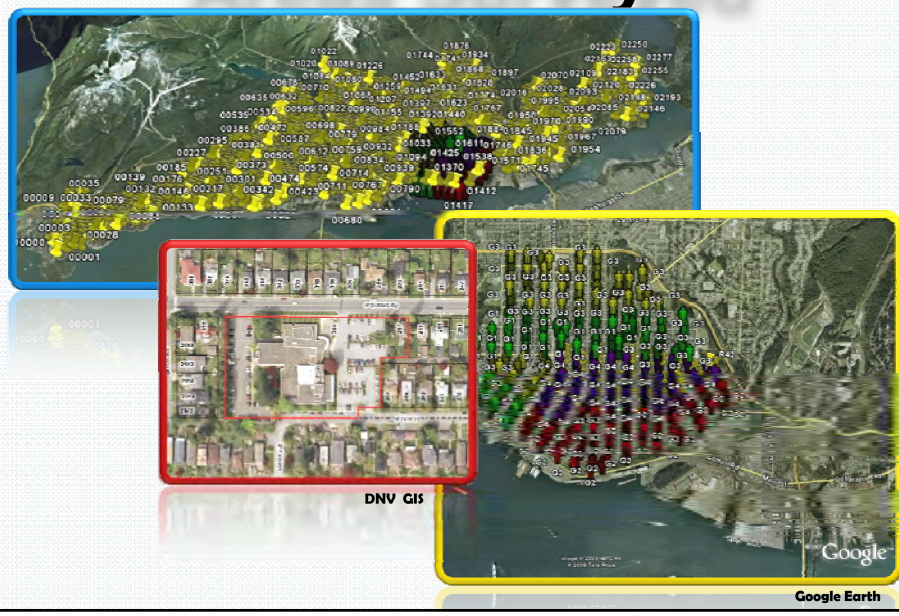
- Majority completed during June
- Most of the City of North Vancouver was surveyed by foot, especially along Lonsdale Avenue, and the two blocks to either side

Drive-by's

- Done after computer Building Inventory to fill in details where needed, such as prototypes , building use, and condition of building
- Although no photos taken during this portion, it was almost as detailed as foot survey and much faster

Photos courtesy of LK and JM

Areas Surveyed



DNV GIS

Google Earth

Analysis:

Damage Matrices

- All buildings are classified by prototypes
- Different mean damage factors used for each type
- Percentage of dollar losses are calculated on building by building and block by block basis at different earthquake intensities
- Gives data for GIS map
- Same info used for casualty and functionality losses

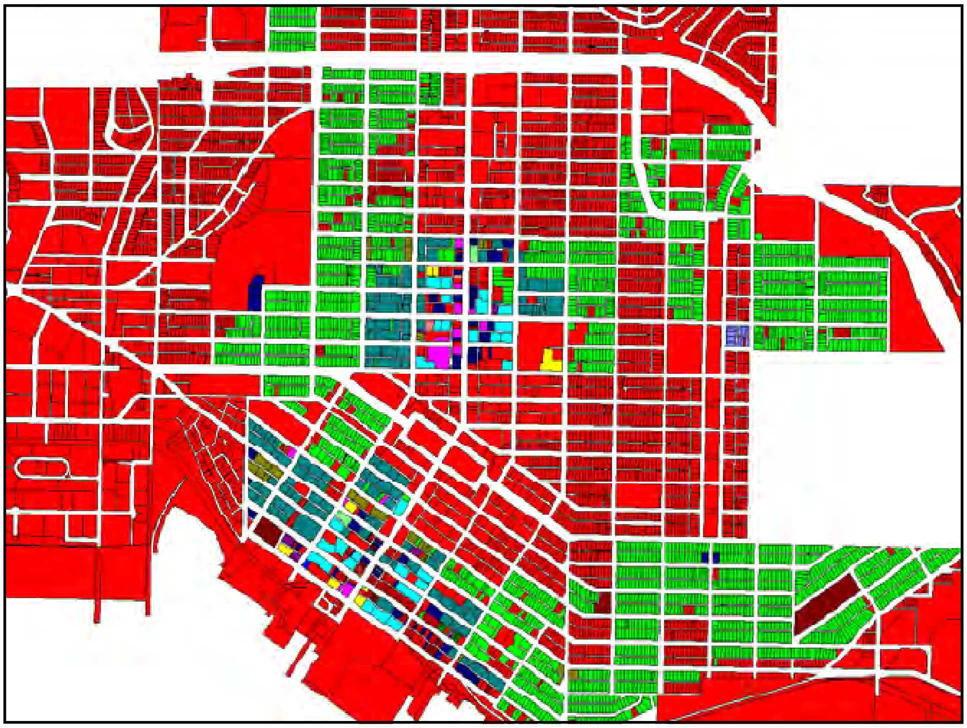
Damage Matrix for WLFR

Description	This prototype includes one or two-storey single family detached homes and attached townhouses. The vast majority of the buildings in southwestern BC are of this prototype.						
CDF	**VI**	**VII**	**VIII**	**IX**	**X**	**XI**	**XII**
0.0	8.0	4.0	1.0	***	***	***	***
0.5	75.0	28.0	6.0	1.0	***	***	***
5.0	17.0	64.0	86.0	69.0	10.0	2.0	***
20.0	***	4.0	5.0	20.0	76.0	69.0	42.0
45.0	***	***	2.0	10.0	12.0	25.0	50.0
80.0	***	***	***	***	2.0	4.0	6.0
100.0	***	***	***	***	***	***	2.0

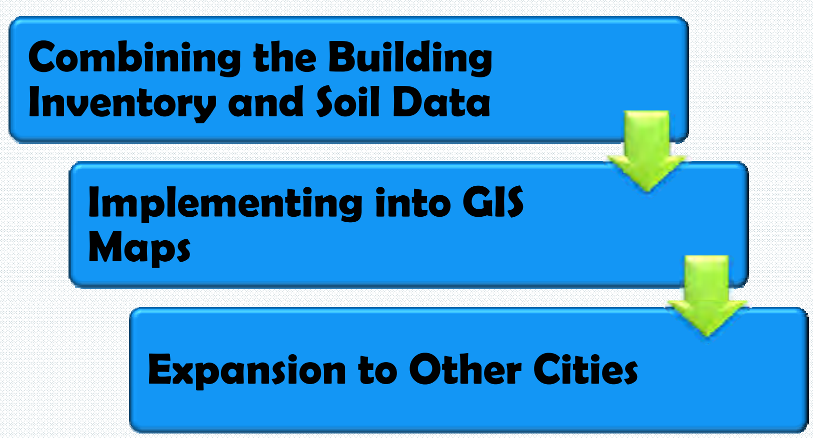
MDF = 6.23%

GIS map

- **Takes Mean Damage Factors and map out distribution over different locations, colour coded for different values**
- **Level of economic, casualty, functionality losses will be mapped out using the same approach**



Project Future



Thank You